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Fleet management of rural timber bridges

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by

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1 INTRODUCTION

1.1 GENERAL INTRODUCTION

There are roughly 600,000 bridges in the United States' National Bridge Inventory (NBI), a compilation of bridge data supplied to the Federal Highway Administration by individual states. Of the roughly 600,000 bridges, an average of 28 percent [32] is 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 [36].

Pontis [36] was first developed through the FHWA, six state DOT's, and a private consultant during the late 1980's. 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 DOT's 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 it is conceivable that 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 [15]. 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 the 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. Sensitivity to weather conditions and de-icing agents is 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 United States 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 Federal Highway Administration 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.

Though 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. Often times, local jurisdictions control these bridges with very little funding for inspection, maintenance, and 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. Under separate covers are 15 individual reports that summarize the visual inspection and static load testing that were performed during the summer of 2006 as part of the development of the fleet management strategy.

1.4 Objective and Scope

The objectives associated with this work are:

1. To 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 to adopt and implement 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. By performing a literature review one can adapt conclusions drawn from previous research, avoid duplication, and gain 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 that 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. Chapter 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, an overview of the California Bridge Health Index is explained, which is partially adapted to this research. Chapter 3 provides the fleet management of timber bridges concept and the evaluation methodology used outlines the methods for obtaining the information necessary to develop the bridge management concept. Next, the bridge management concept development by creating a four-level performance metric is explained. Chapter 4 discusses some applications and limitations of the management concept and conclusions and recommendations from the work are presented.

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” [13]. Stated simply, fleet management is the overseeing of a fleet. According to the article by Wyrick and Storhaug entitled Benchmarking Fleet Management,

> “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. These topics include 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 creating a 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 [15]. 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 the 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 impacting fleet maintenance costs are fleet age and condition. Advanced age for a fleet drives up both direct maintenance and repair costs. Establishing the fleet condition is essential to fleet operations. This task may be started while determining the fleet size. If the initial condition is not properly determined, the starting point for the fleet operations will be incorrect. Because all factors are not objective, a fleet manager may be required to use subjective judgment when determining a fleet size [2].

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 break downs 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 chapter.

2.2.4 Benchmarking

Benchmarking is simply a method of comparing the 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" [20]. Organizations that do not practice benchmarking are only able to compare a fleet to the past performance of that same fleet. Utilizing the costs 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 maintenance or inspection allows one to identify specific areas of excellence or needed improvement. When addressing bus maintenance, Maze [29] 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 [29]. 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.

Table 1. Performance Measurement Areas [29]

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.

A six-step benchmarking process is described in *Benchmarking Procedure for Fleet Management* [20].

1. Determine the focus of the benchmarking study.
2. Understand the organization in order to understand the process to be benchmarked.
3. Determine what to measure and determine the measures of performance.
4. Determine what to benchmark against.
5. Benchmark.
6. Improve performance.

The six-step process listed 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

In order 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 what is capable of a singular 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 a fleet manager has, in effect, numerous fleets under his 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. Two examples of established information sharing programs in existence are described in the subsequent paragraphs.

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 [2]. 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 General Services Administration and the 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, while specific data is 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" [18]. It is well known that preventive maintenance is beneficial to the life expectancy of bridges. The problem is, however, when and to what extent preventive maintenance should take place in order to maximize cost efficiency. A vast number of bridges are under the control of small jurisdictions and bridge managers certainly do not have unlimited funds. Often times, even necessary maintenance is 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 funds 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 [23] follows.

- To maximize the useful life and reliability of the fleet while minimizing total life cycle costs.
- To assure that preventive maintenance work is scheduled at appropriate intervals.
- To assure 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 that of scheduled repairs, provided that a proper preventive maintenance program exists [18]. One needs to be aware of excessive preventive maintenance as 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.

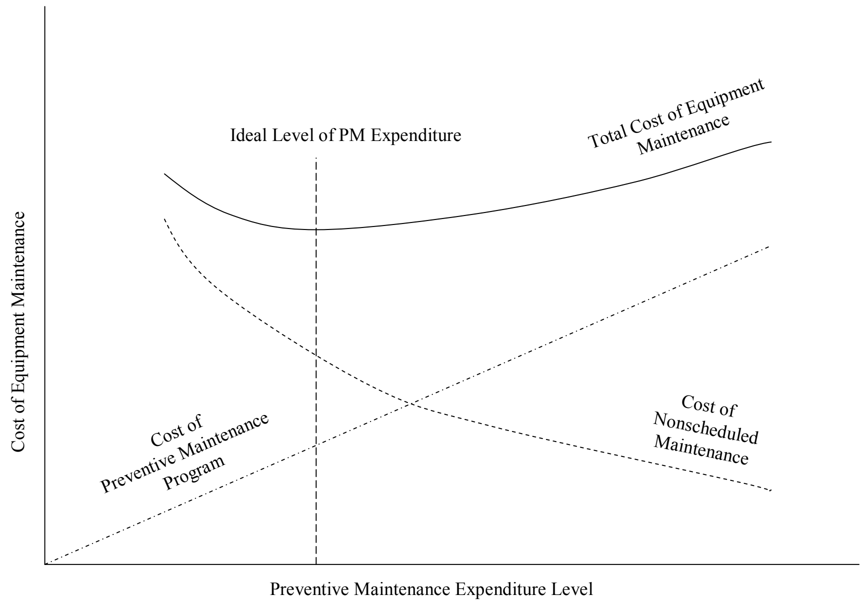


Figure 1. Effect of Preventive Maintenance Levels on Total Equipment Maintenance Costs (Adapted from Fleet Management; Dolce, 1994)

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. The experience and self-study of an individual is an invaluable asset 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 vehicle owner's manuals (e.g., change oil every 3000 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 broken into several different categories because all servicing does not require the same level of work. Dolce [18] notes four different categories of planned maintenance: A, B, C and D. A involves frequent but minor service work whereas, B is less frequent but the work is more complex than that of A. C involves both minor and major testing and overhaul and replacement of certain components and D often requires the planned replacing or rebuilding in entirety [18].

For a preventive maintenance plan to be effective, one should develop a simplistic and user friendly schedule. Though one may find in the scope of fleet needs that maintenance procedures at irregular intervals is most effective, all 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 six months, then category B maintenance should be every 12 months, and category C every 24 months, etc.

If unscheduled or emergency repairs are required, the preventive maintenance schedule should be checked so that any preventive maintenance can be concurrently administered if due in near future. This will help prevent resources from being used more than necessary.

To provide proper inspection techniques, a necessity exists to perform inspections 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 [18].

Preventive maintenance compliance reports should also 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. The possibility exists that these problem areas are severe in nature and could be cause of 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.

Lastly, correction should take place as needed. The possibility exists that further damage can 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 in 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 will 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 [18].

2.3.5 Effects of Technological Advance 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, advancement of technology has enabled the fleet manager 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 technology continually decreases in cost. One has the ability to retrieve extensive databases very quickly enabling that individual to identify trends and support management decisions. Predictability, quite possibly, is 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 *Federal Fleet Management Report* [15])

- 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 applied at least partially 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. Each project will be discussed in the following sections, respectively.

2.4.1 T-Beam Bridges in Pennsylvania

In May of 1998, the Pennsylvania Department of Transportation (PennDOT) presented researchers with a need for determining the capacity of reinforced T-beam bridges in Pennsylvania [11]. 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 stemming the idea of treating the bridge population as a fleet. Though 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 percent 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 expectantly 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 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 of which the structural details and element dimensions are dependent on the span length and width of the bridge; thus, radical geometrical variations in the bridge 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.

Table 2. Critical Parameter Affecting Load Capacity Rating [11]

Nominal Structural Parameters	Condition Parameters	Info 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	

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 PC I-girder bridges [8]. 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 1960’s 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 varying 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 was necessary to better evaluate the process of protection and repair methods. However, like 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, a representative group was not defined by statistical methods within the documentation. That is, 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 [37] is a diagnostic tool developed by the California Department of Transportation (Caltrans) with intentions of maximizing the duration of service while minimizing the 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 [38].

Two primary differences exist between the California Bridge Health Index and the Federal Highway Administration's Sufficiency Rating [32]. 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 only accounts for the structure condition whereas the Sufficiency Rating includes a bridge's function (e.g., 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 that one is suggested to investigate. For the purposes of this study, however, a focus will be placed on the bridge condition description. At the element level a total quantity of each respective element that is present throughout the bridge is determined and 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 that compose the NBI, approximately 30,000 are considered timber structures. The Bridge Engineering Center of Iowa State University (BEC) 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 sub-sample 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, 2703 bridges were categorized as single-span, timber girder bridges with a bituminous wearing surface.

The procedure for random sampling is provided 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 [28] and X^2 Goodness-of-Fit Test [16] 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 (e.g., 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, say, length 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. That is, only if the sample is independent and is from a single probability distribution having mean μ and standard deviation σ [28]. 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 [27] 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.
- the population distribution has a skew index outside the range of +1 to -1 and the sample size equals 25 x SKEW^2.

The skew index is evaluated by the equation,

Equation 1. Skew Index

$$SKEW = \frac{(A - B + C)}{(StndrdDeviation)^3}$$

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 where:

$N =$ Total Number in Population

$n =$ Individual Population Values

$\mu =$ 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 [16] can be used to determine if a sample distribution of a single parameter (e.g., length, etc.) represents the population distribution of that same parameter. In this test the null hypothesis (H_o) states that the proportions in the sample are similar to the population proportions or, more simply, the sample is a good fit. Conversely, the alternate 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 to the expected frequency, E_i, by means of the following equation:

Equation 2. X^2 Statistic

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

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 $X^2_{\alpha,n-1}$ 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 [14]. 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 [16].

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.

Table 3. X^2 Comparison for Sample Size of 20

NBI Category	X^2	$X^2_{0.05,n-1}$
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

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. One should note that 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.

Though sampling procedures were random it was not feasible to continue the investigation into all of those bridges. Due to budgetary and time restraints and bridge owner 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. Even though 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 using [35] and [40]. 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. This form is presented in Appendix A.

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.

Often times, with timber bridges the origin of deterioration is a direct result of deterioration in the wearing surface and deck. 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. 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 can 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 eight bridges in western North Carolina, four in western Montana, and three 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 very nearly the same. A typical truck and truck dimensions are shown in Figure 2 and Figure 3, respectively.

Figure 2. Typical Load Truck

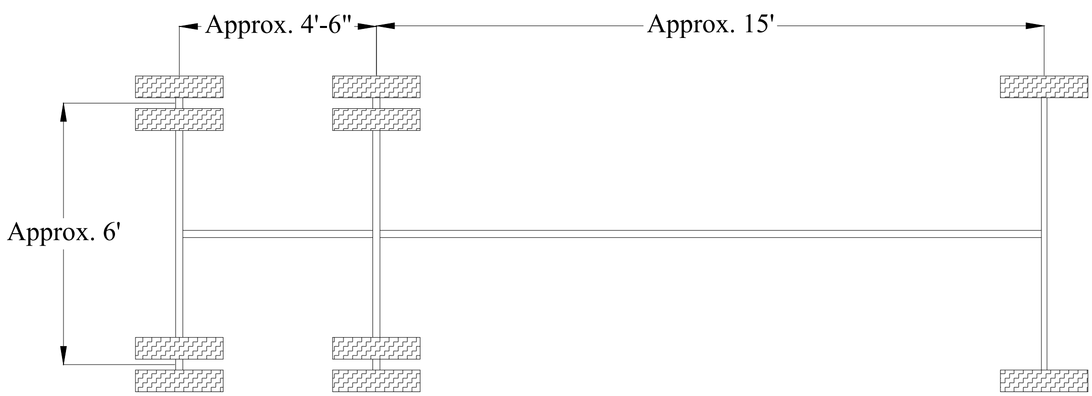


Figure 3. Typical Truck Dimensions

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

Deflection data were collected through the use of ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. and the signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS 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 Figure 5).

Figure 4. Typical Deflection Gage Instrumentation

Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) 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 as shown in Figure 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 two inches from the top of the girder (see Figure 6).

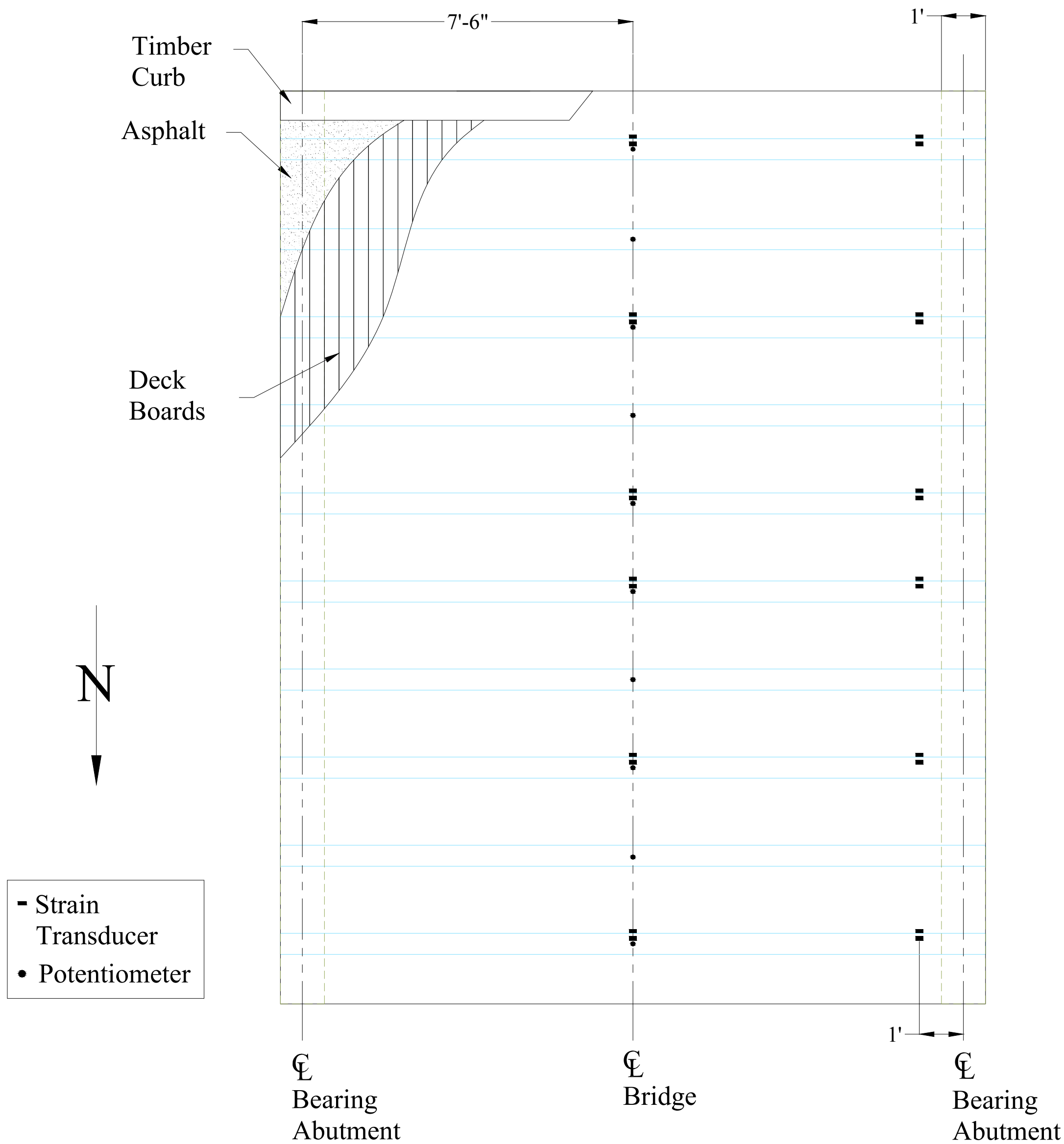


Figure 5. Typical Deflection Gage and Strain Transducer Locations

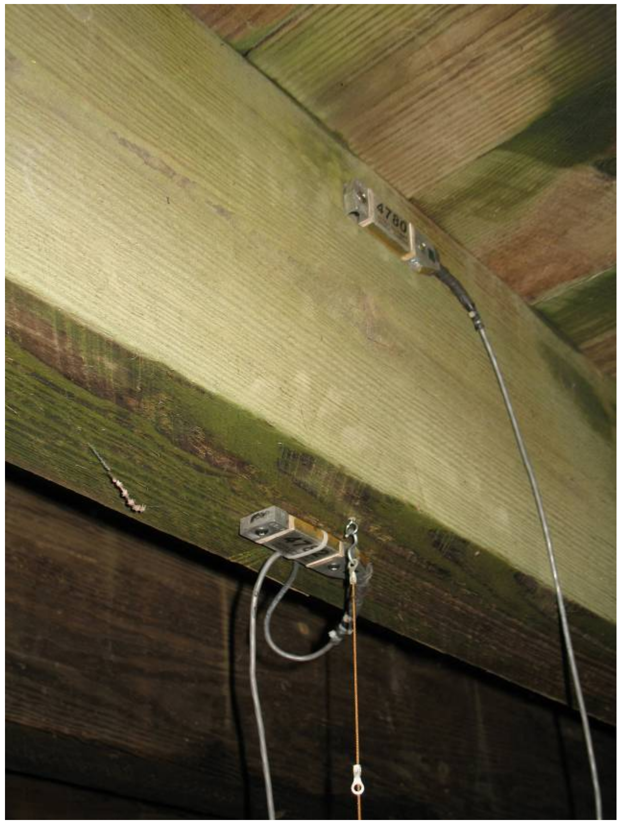
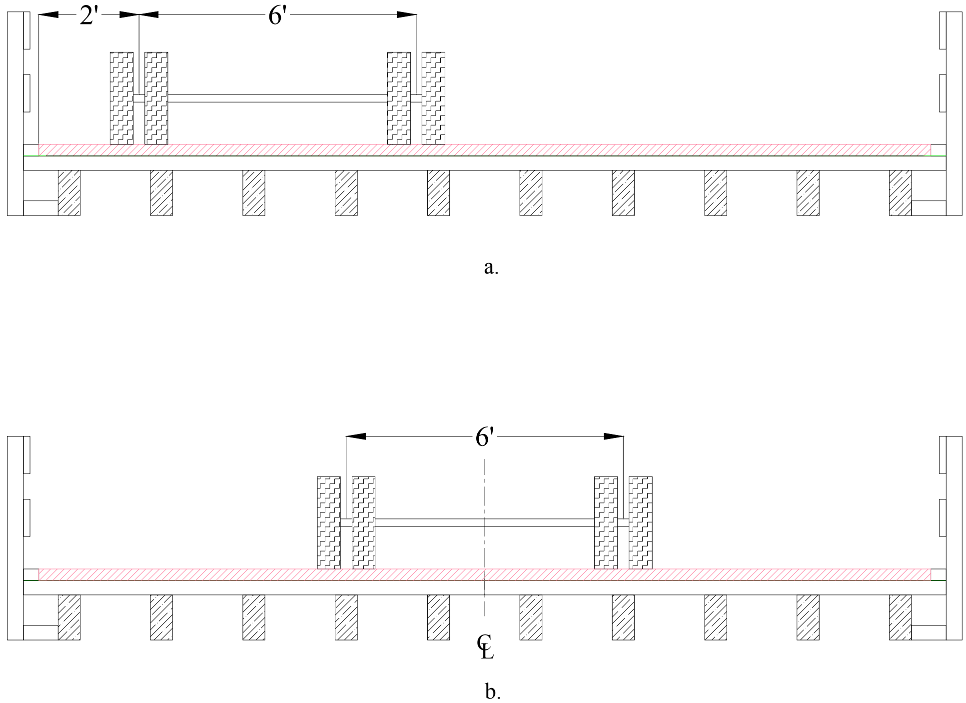


Figure 6. Typical Strain Transducer Placement

To provide consistency and for comparison purposes, 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 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 the collected data were repeatable. Figure 7 illustrates the typical load paths. Note that typical experimental results are given in the following sections.



c.

Figure 7. Typical Load Paths

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 no. 560510, Montana bridge no. L25003009+09001, and Colorado bridge no. P-19-AS.

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 timber solid sawn girder bridge with a bituminous wearing surface located in Madison County, North Carolina. Figure 8 shows a picture of the North Carolina Bridge.

Figure 8. North Carolina Bridge used for Finite Element Modeling

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

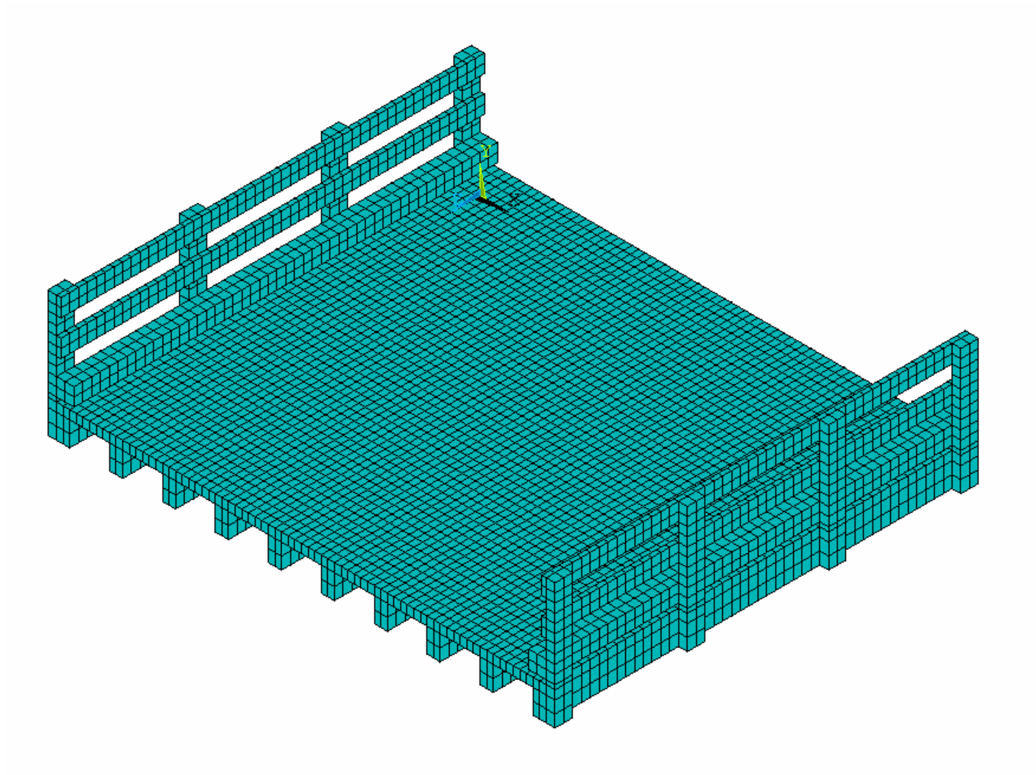


Figure 9. Finite Element Model of North Carolina Bridge

Unlike steel or concrete where the 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 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 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.

Figure 10 through Figure 12 shows the calibration results from when the field and finite element results were nearly equal when subjected to the same loading.

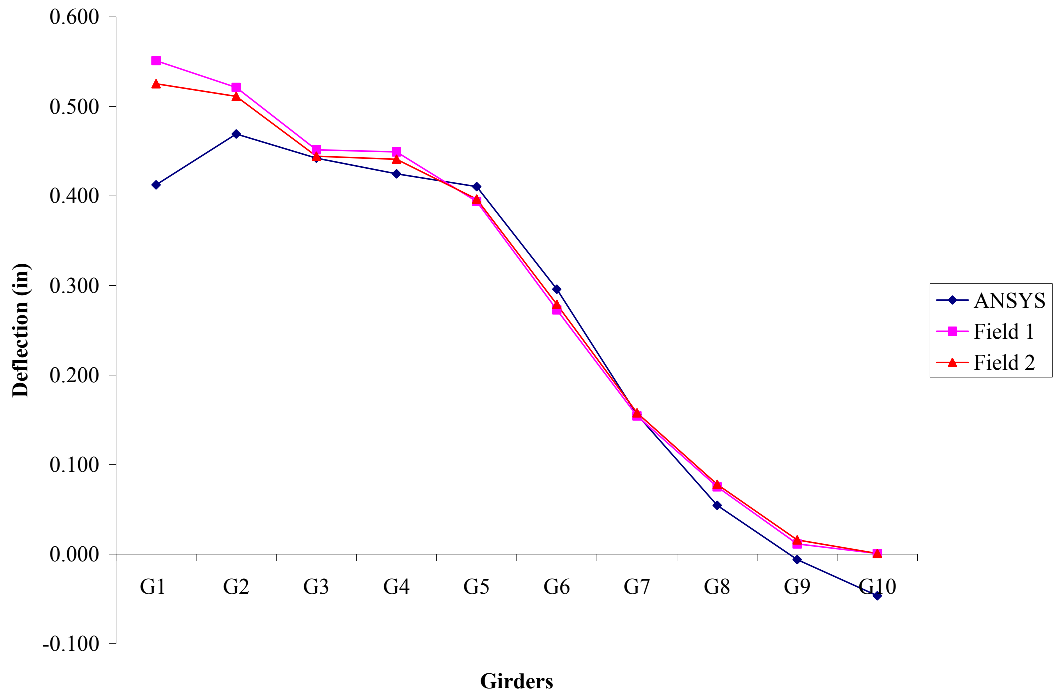


Figure 10. North Carolina Finite Element Calibration Results Load Path 1

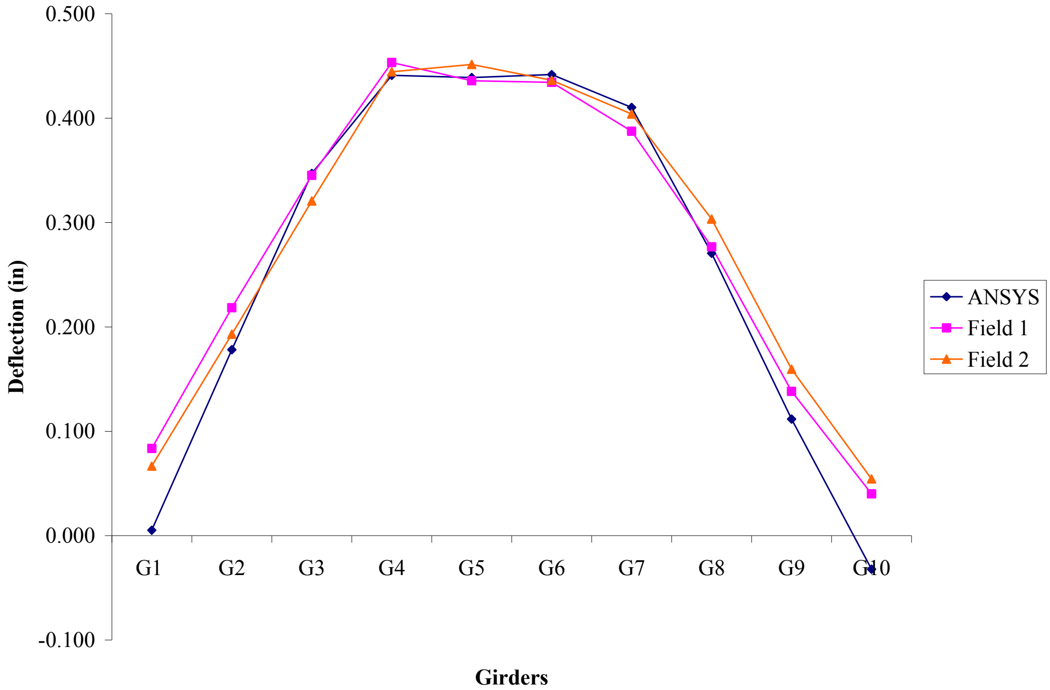


Figure 11. North Carolina Finite Element Calibration Results Load Path 2

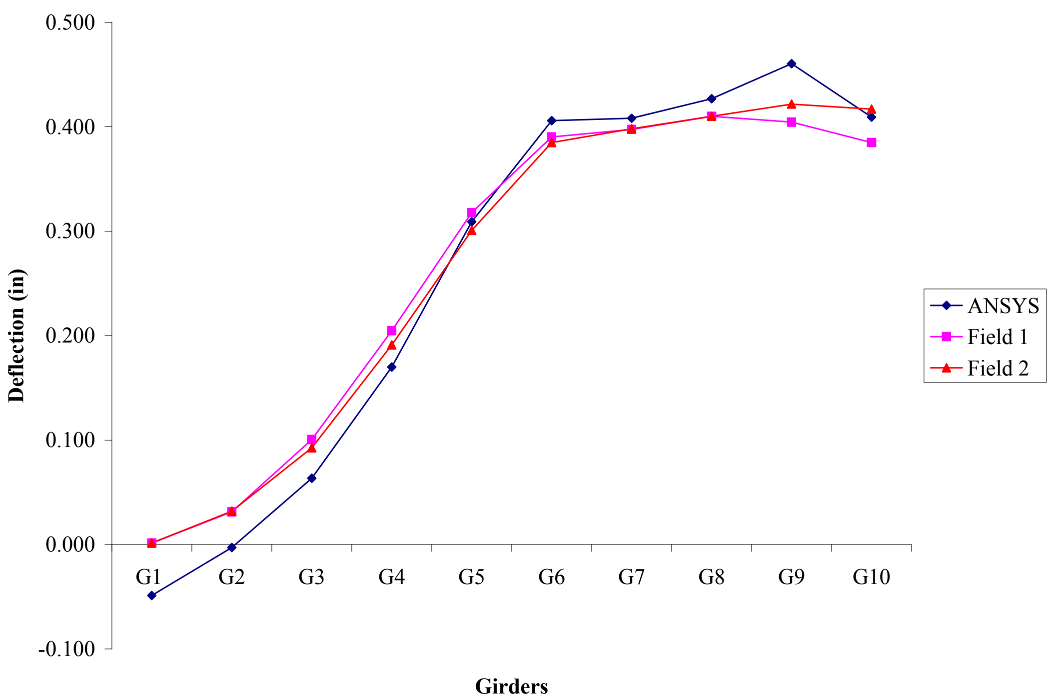


Figure 12. North Carolina Finite Element Calibration Results for Load Path

The Montana Bridge is a single span, two-lane, solid sawn timber girder bridge with a bituminous wearing surface located near Wolf Creek, Montana. Figure 13 shows a picture of the Montana Bridge.

Figure 13. Montana Bridge used for Finite Element Modeling

This bridge was subjected to static live load testing during the summer of 2006. From the load test, deflection results were obtained from which a finite element model could be calibrated. Shown below in Figure 14 is the finite element model developed and much like the previous model, this model consists of solid elements with anisotropic capabilities representing the girders, deck boards, and rails

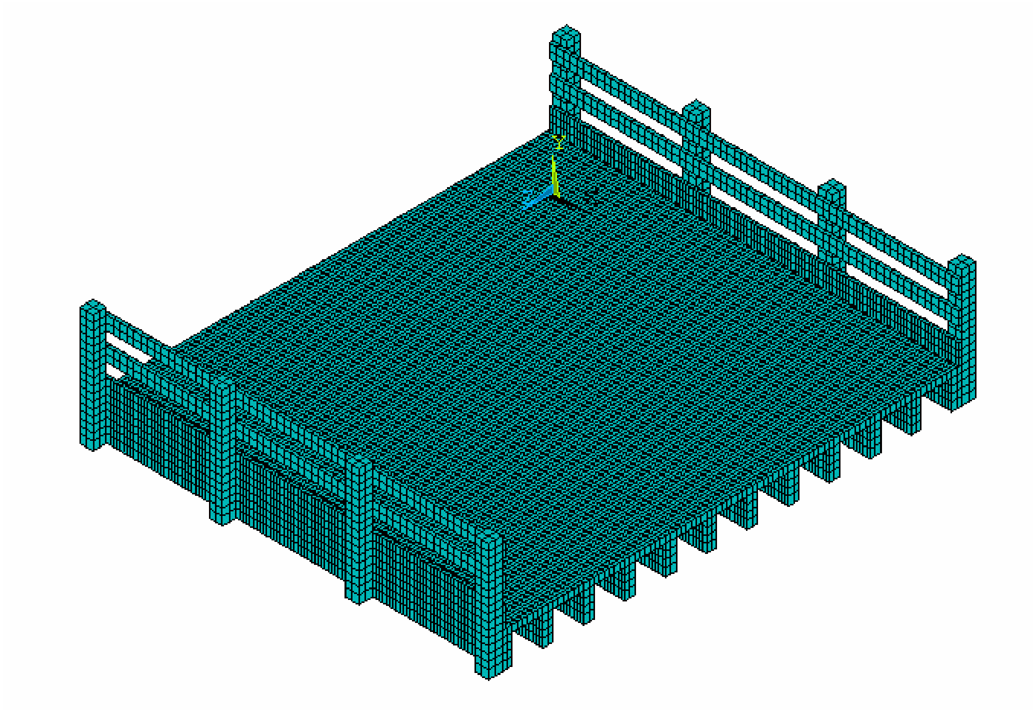


Figure 14. Finite Element Model of Montana Bridge

Similar to 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. Figure 15 through Figure 17 shows the calibration results from when the field and finite element results were nearly equal when subjected to the same loading.

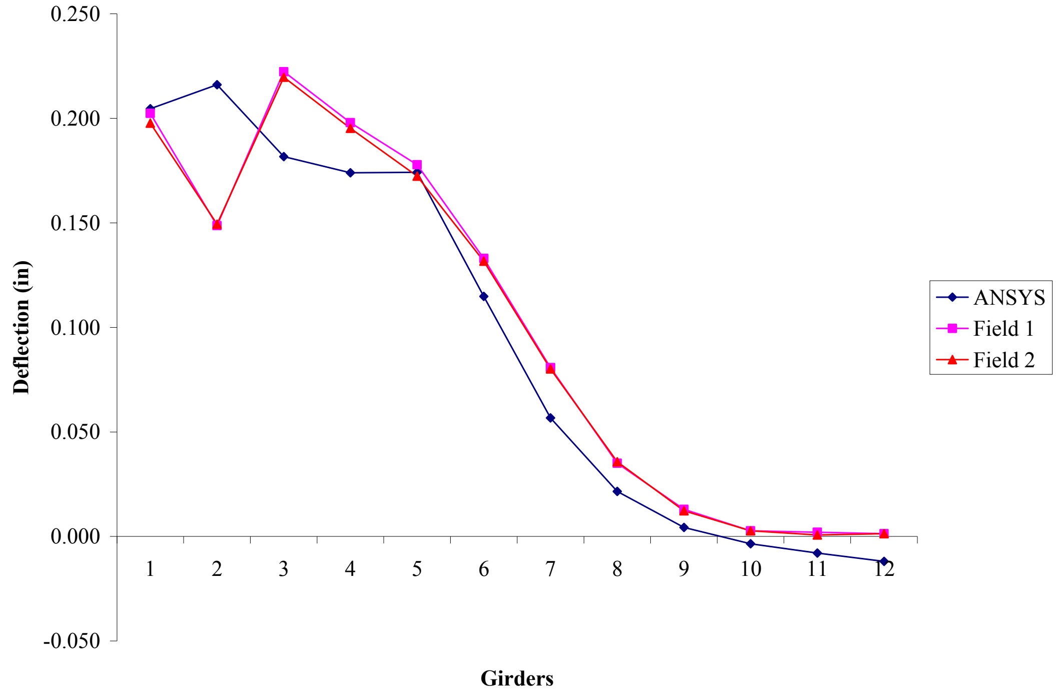


Figure 15. Montana Bridge Finite Element Calibration Results for Load Path 1

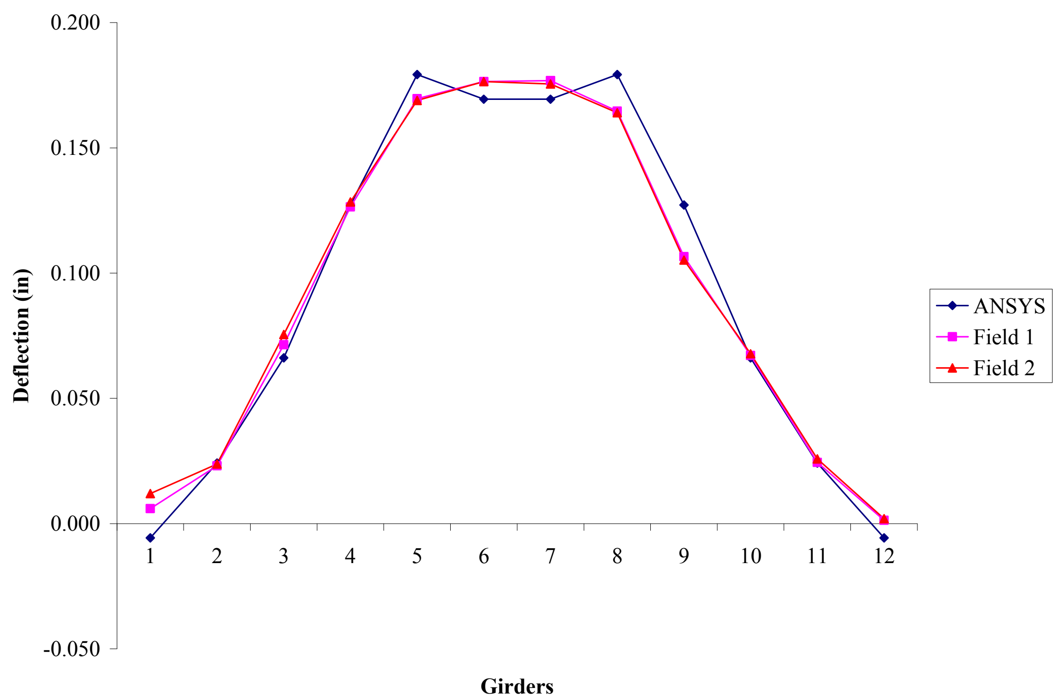


Figure 16. Montana Bridge Finite Element Calibration Results for Load Path 2

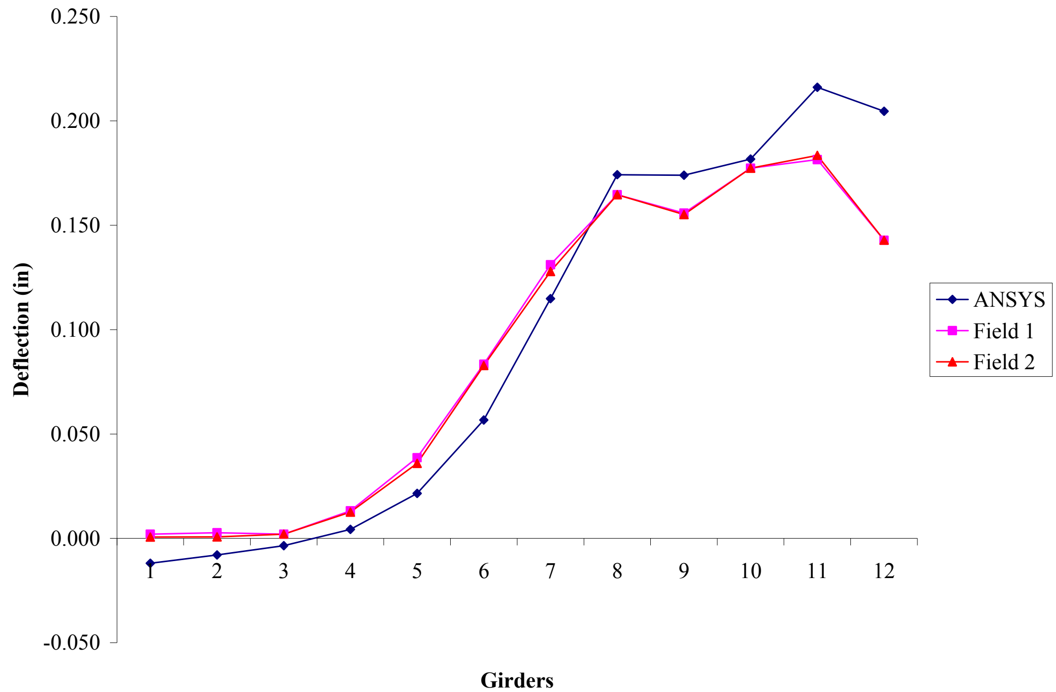


Figure 17. Montana Bridge Finite Element Calibration Results for Load Path 3

The Colorado Bridge is a single span, two-lane, solid sawn timber girder bridge with a bituminous wearing surface located near Trinidad, Colorado. Figure 18 shows a picture of the Colorado Bridge.

Figure 18. Colorado Bridge used for Finite Element Modeling

This bridge was subjected to static live load testing during the summer of 2006. From the load test, deflection results were obtained from which a finite element model could be calibrated. Shown below in Figure 19 is the finite element model developed and 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.

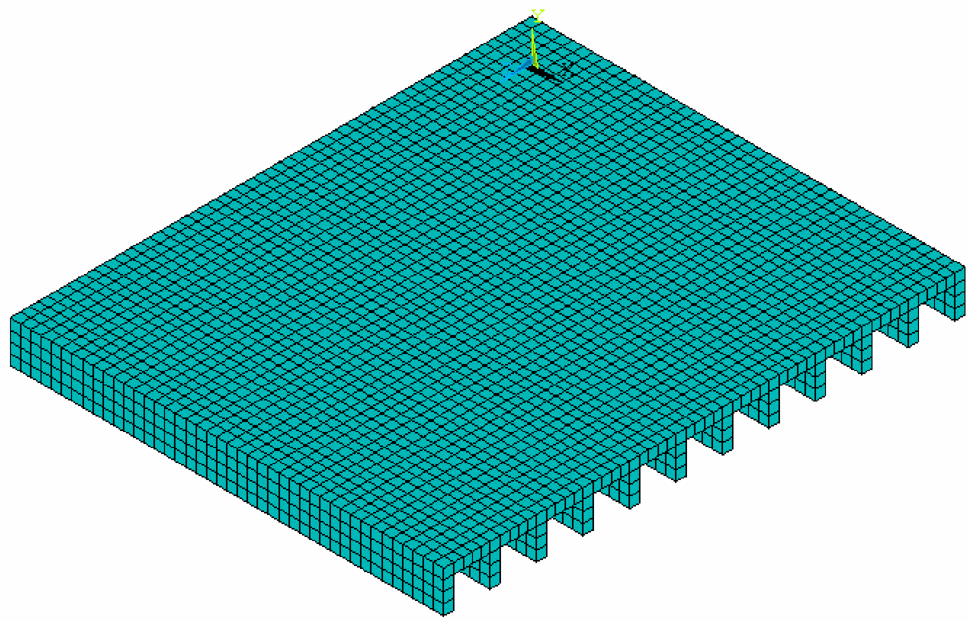


Figure 19. Finite Element Model of Colorado Bridge

Much like the previous two models, a typical range for the modulus of elasticity was determined and a value was arbitrarily selected to first model the bridge. Again, 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. Figure 20 through Figure 22 shows the calibration results where the field results and finite element results were nearly equal when subjected to the same loading.

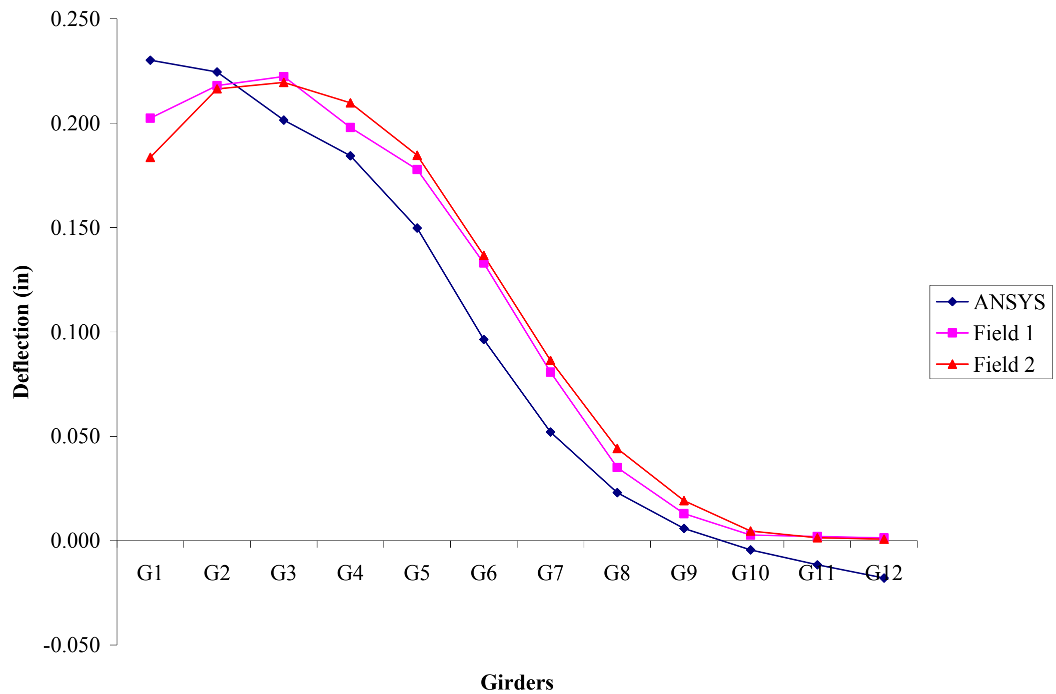


Figure 20. Colorado Finite Element Calibration Results for Load Path 1

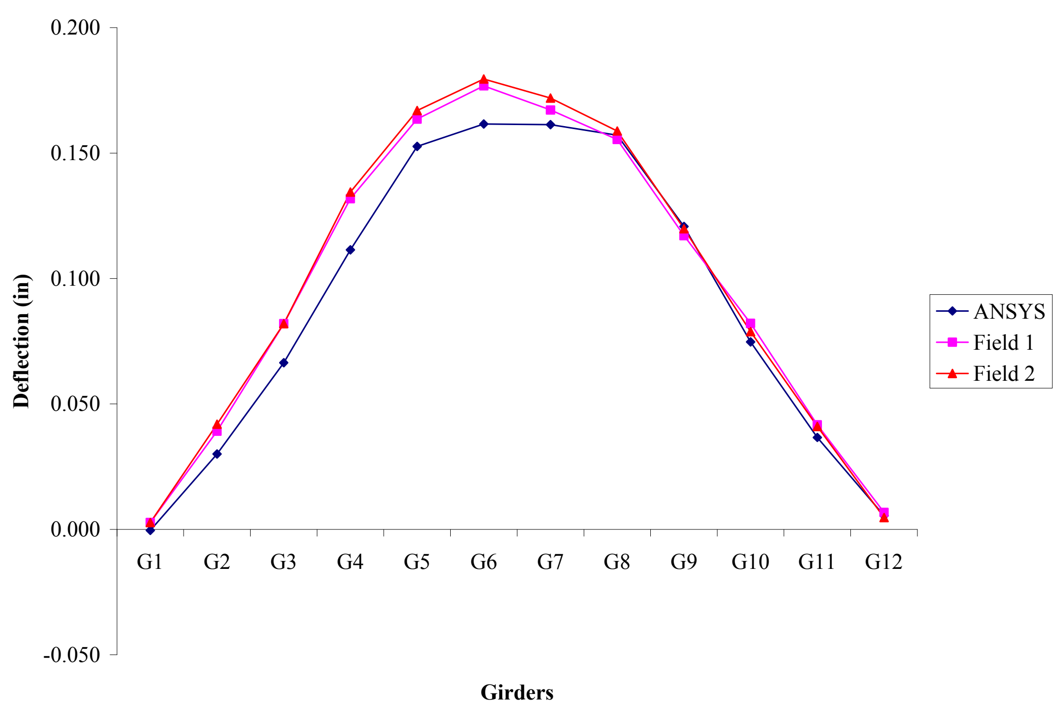


Figure 21. Colorado Finite Element Calibration Results for Load Path 2

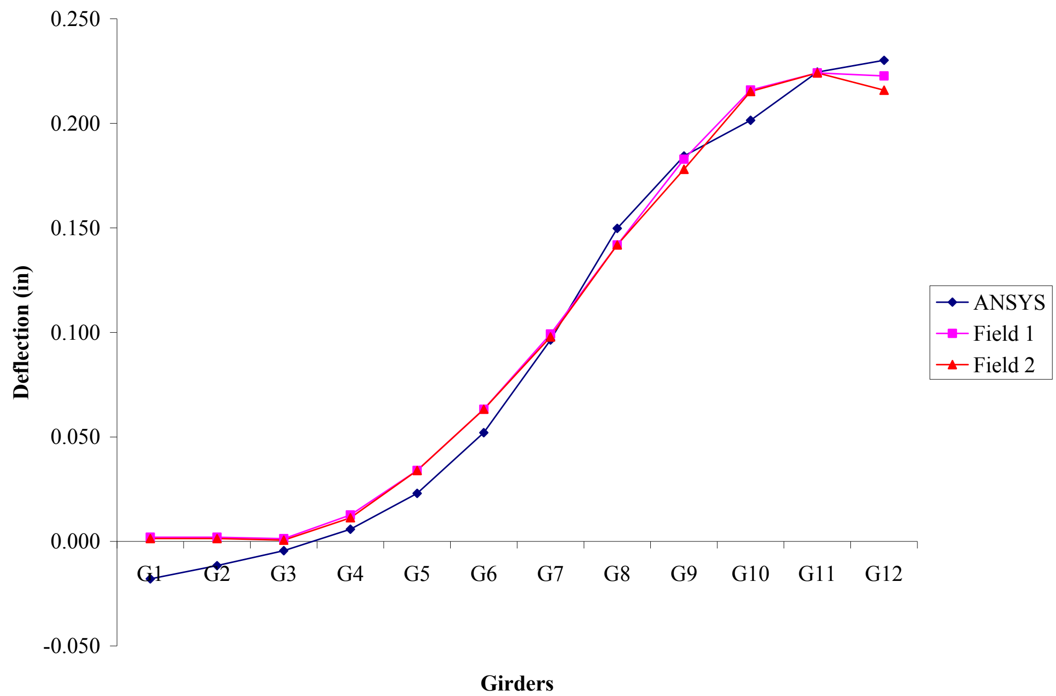


Figure 22. Colorado Finite Element Calibration Results for Load Path 3

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 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 the visual inspection of 23 bridges completed by the research team; the third level is based on the static load test of 15 bridges performed by the research team; the fourth level is based on the finite element analysis of three bridges subjected to the 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.

Through extensive investigation into the correlation of deck and superstructure rating to age, geometrical, and service descriptors, it was found that very little correlation existed with each descriptor alone. This was somewhat surprising. So, a performance parameter was developed that incorporated all of the items that were thought to be of greatest influence on the bridge performance. These items were the deck and superstructure rating obtained from visual inspection, the inventory rating, and the structural evaluation. To briefly recap 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 a 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 which 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, substructure rating, or the code developed by incorporating inventory rating and average daily traffic (ADT) shown in Table 4.

For all bridges in the fleet where 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.

Table 4. Rating by Comparison of ADT and Inventory Rating

Structural Evaluation Rating Code	Inventory Rating Average Daily Traffic (ADT) 0-500	501-5000	>5000
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.		

* MS Designation (typical)

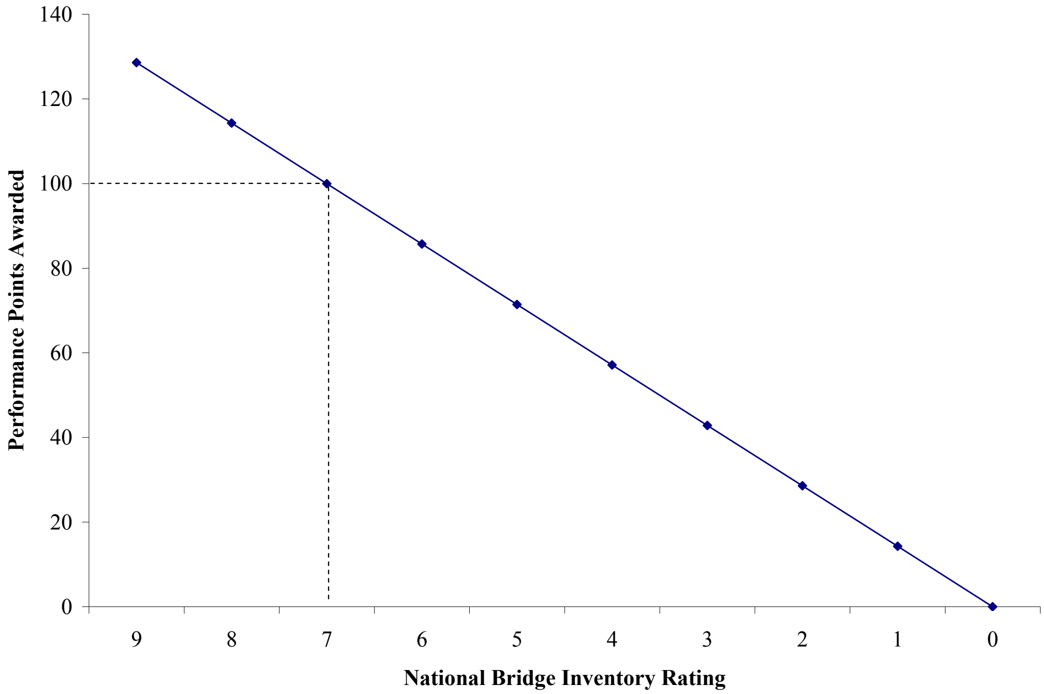


Figure 23. Level 1 Awarded Performance Parameter Points for Deck Rating, Superstructure Rating, and Structural Evaluation Rating

Since 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 like 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 mTon). 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.

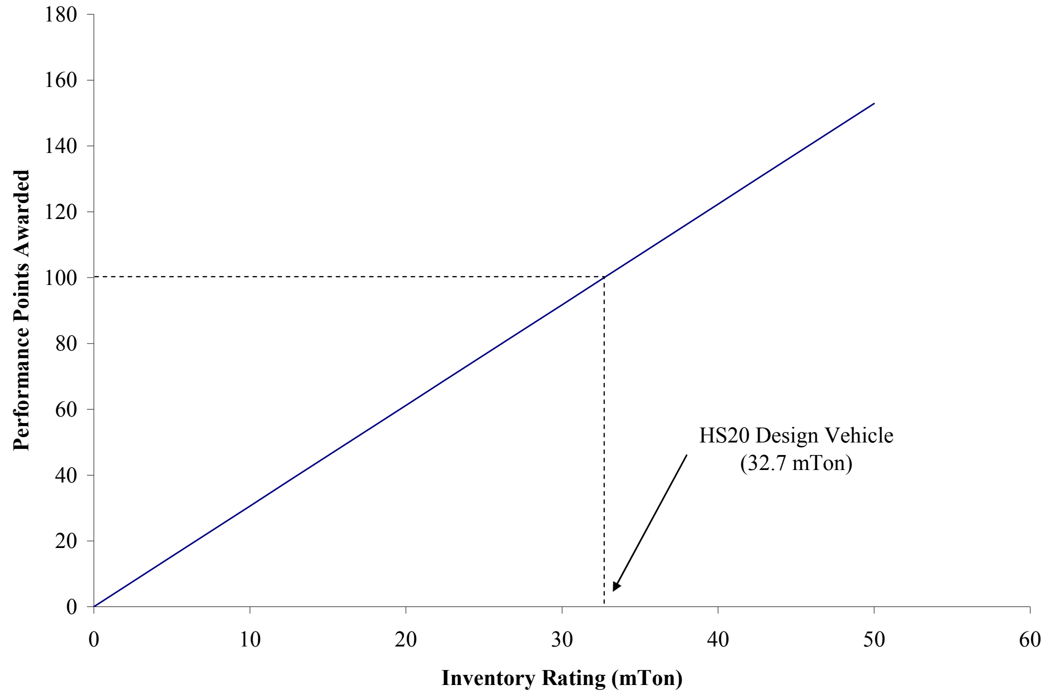


Figure 24. Level 1 Performance Parameter Points Awarded for Inventory Rating

Subsequent to all the 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 evaluation were 30 percent, 20 percent, 40 percent, and 10 percent, 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.

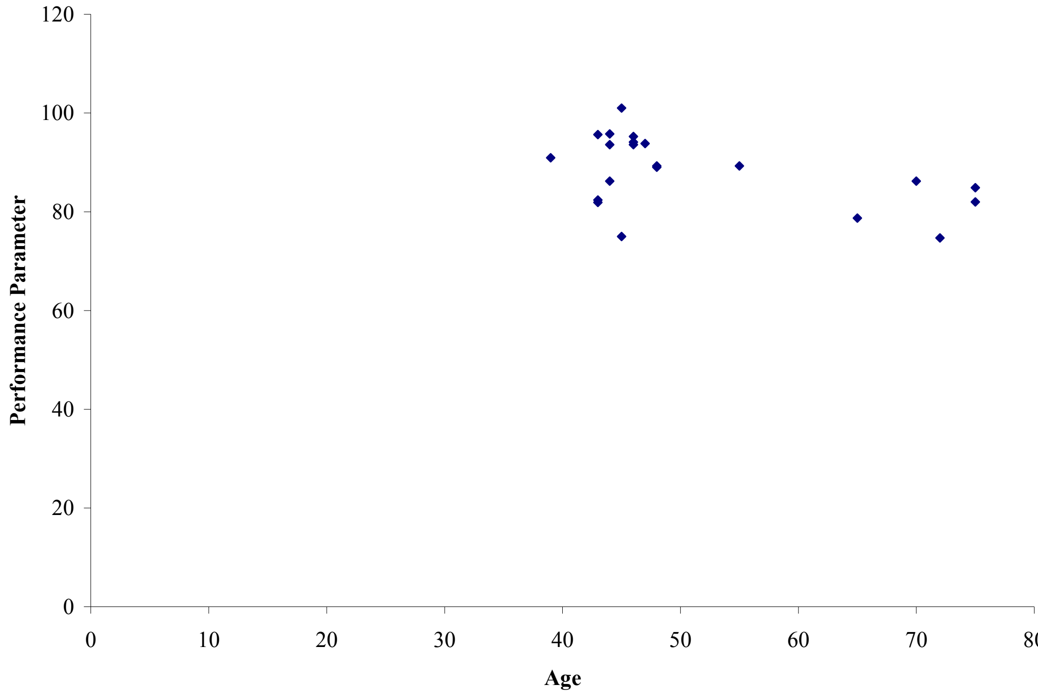


Figure 25. Total Scores After Level 1 Performance Determimination

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 the summer of 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 percent and 60 percent of the total score, respectively.

Within the deck and railing category, the 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 State 1, State 2, State 3, State, 4 and State 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 (i.e., State 1 = 100 percent, State 2 = 75 percent, State 3 = 50 percent, State 4 = 25 percent, and State 5 = 0 percent). For example, if 65 percent of the deck and railing was in excellent condition with respect to color change, 30 percent in good condition, and 5 percent 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 to 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 percent of the total deck and railing score. The other 40 percent 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.

						Points	
						Awarded	**Possible**
Deck Moisture Percentage							100
	State 1 1	**State 2** 0.75	**State 3** 0.5	**State 4** 0.25	**State 5** 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
						Deck and Railing Score	

Figure 26. Deck and Railing Scoring Example

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[9, 35], it was determined that timber at 19 percent moisture content was still apt to achieve desirable strength performance and was thus awarded 100 points. If it is assumed that timber loses 25 percent of its strength by increasing the moisture content to 25 percent, the assumed fiber saturation point, then the scale for apportioning points is shown below 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.

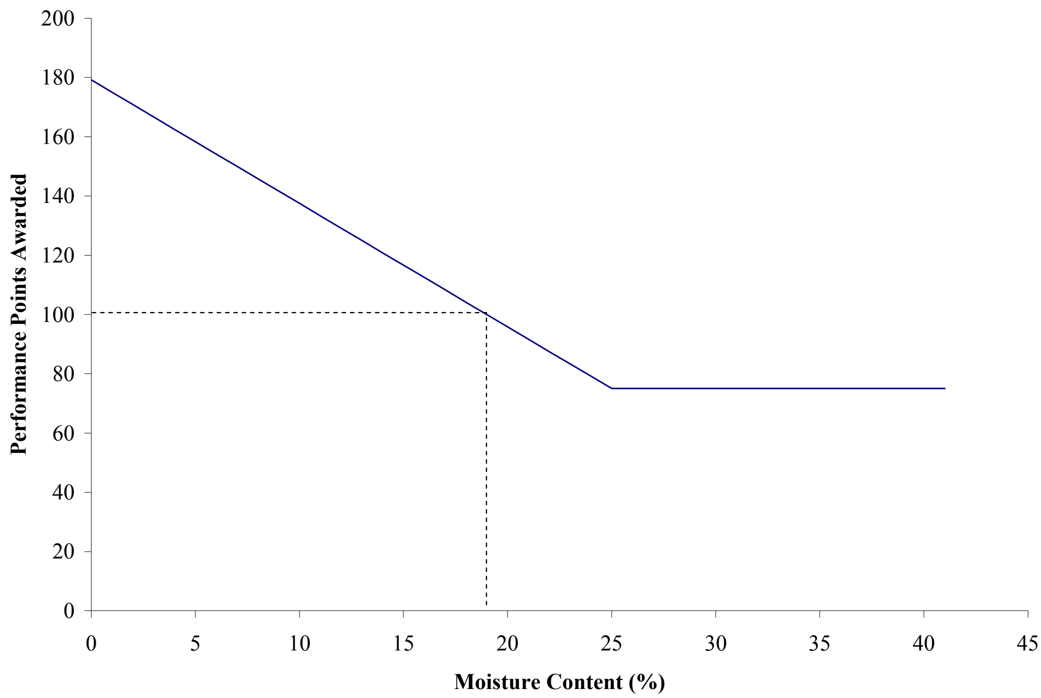


Figure 27. Level 2 Moisture Content Performance Parameter Points

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.

Clearly, 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 and there are 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 percent of the total score, whereas the 3) wearing surface portion makes up only 10 percent of the total score. It was thought that this weighting reflected the importance of each category to the structural integrity of the bridge. The total scores after the determination of Level 2 scores are shown in Figure 28.

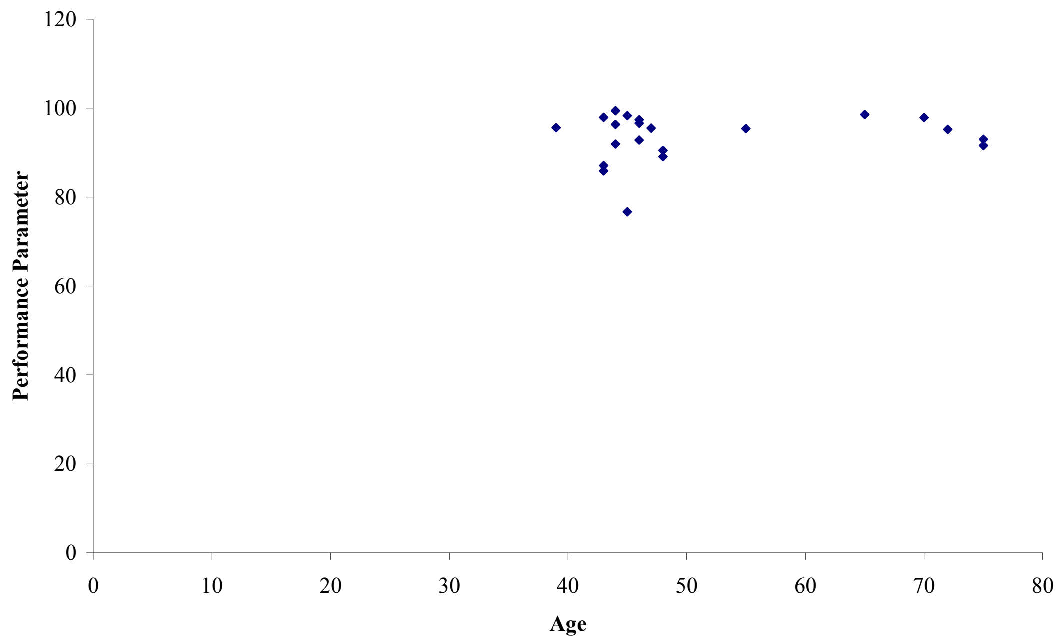


Figure 28. Total Scores After Level 2 Performance Determination

3.2.1.3 Level 3

Level 3 scoring utilized the data acquired from load testing a bridge. Much like Level 2, three major categories make up the final Level 3 scoring. These categories include 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. Level 1, Level 2, and Level 3 composed 20 percent, 35 percent, and 45 percent of the total score, respectively.

The recommended midspan deflection limit for timber girders is L/360 [35], where L equals the clear span distance of the girders. For each load path the maximum normalized midspan deflection was identified and compared to 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, zero points were awarded. Conversely, if the maximum midspan deflection was equal to zero, a maximum of 200 points was awarded. The scale by which points were awarded for midspan deflection is shown in Figure 29.

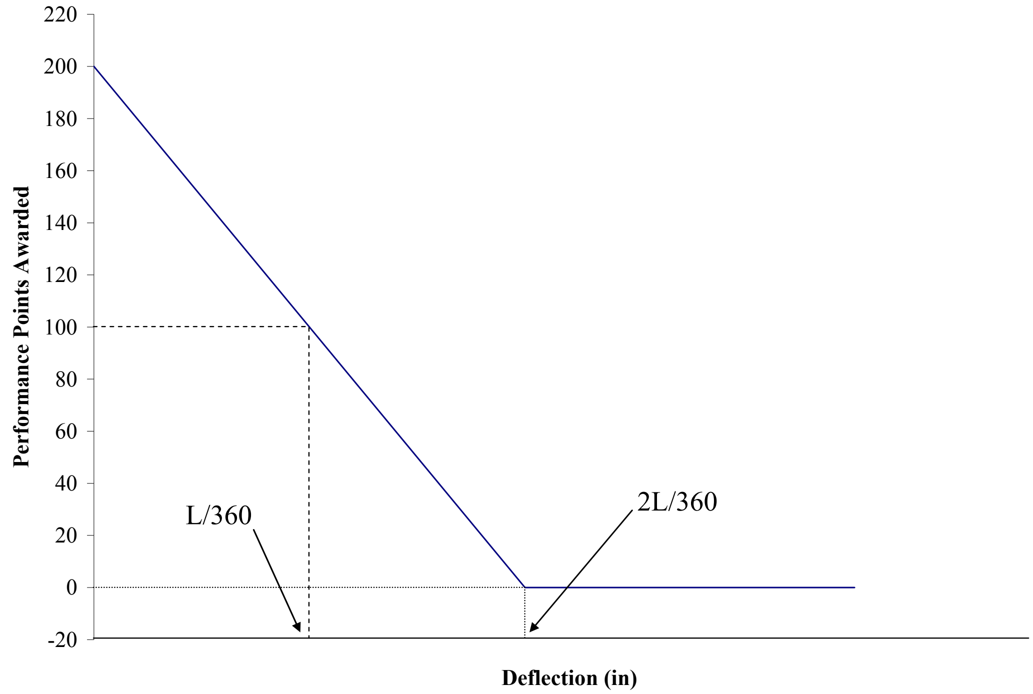


Figure 29. Level 3 Performance Points Awarded for Maximum Midspan Deflection

Little information regarding differential deflection between adjacent girders is published, so unlike the midspan deflection limits, recommended differential deflection limits do not exist. Nevertheless, it is known that large differential deflections between adjacent girders can have adverse affects 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 zero points were awarded. If the maximum differential deflection was equal to zero, 200 points were awarded and 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.

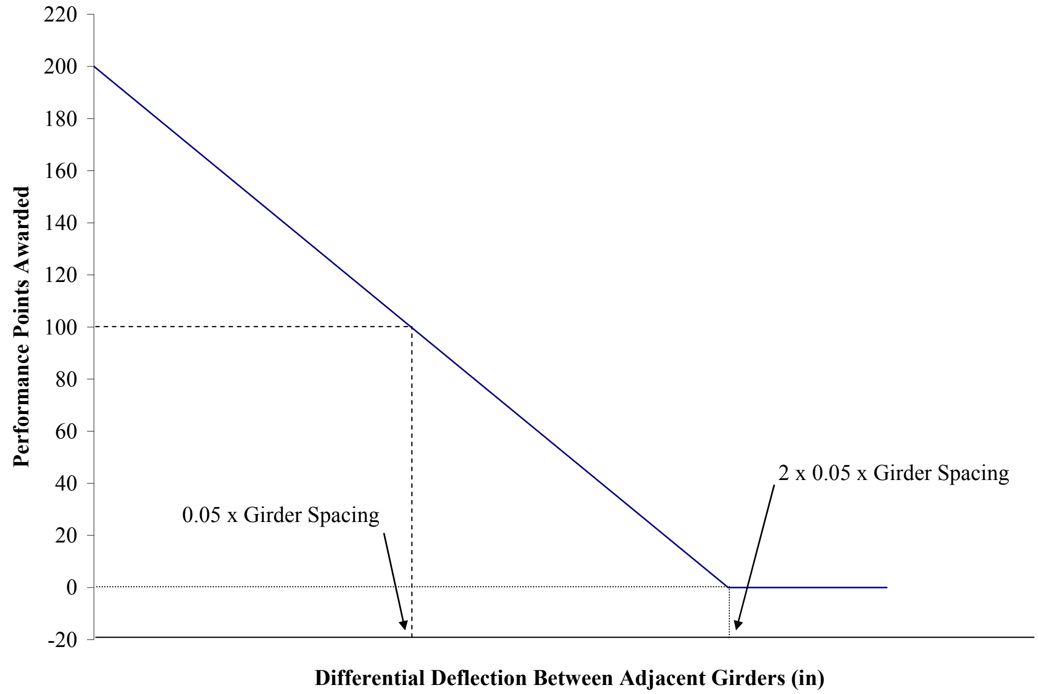


Figure 30. Level 3 Performance Points Awarded for Differential Deflection

AASHTO code provisions specify a distribution factor limit for the types of bridges included in this study. For single lane loading a distribution factor of S/6.7 and for two-lane loading a distribution factor of S/7.5 is specified, where S is the spacing between girders in ft. 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 only considered a one-lane bridge and the two-lane distribution factor is not applicable. Some of the clear roadway widths of the bridges in this study are less than 20 ft and for these bridges only one-lane distribution factors were considered. That is, the distribution results obtained from the normalized vehicle were only compared to 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 summed and compared to 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 path 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 zero points, and in the case where the distribution factor was equal to zero, 200 points were awarded. Figure 31 shows the scale by which the distribution factor points were awarded. Shown in Figure 32 is the total score after the Level 3 scores were determined.

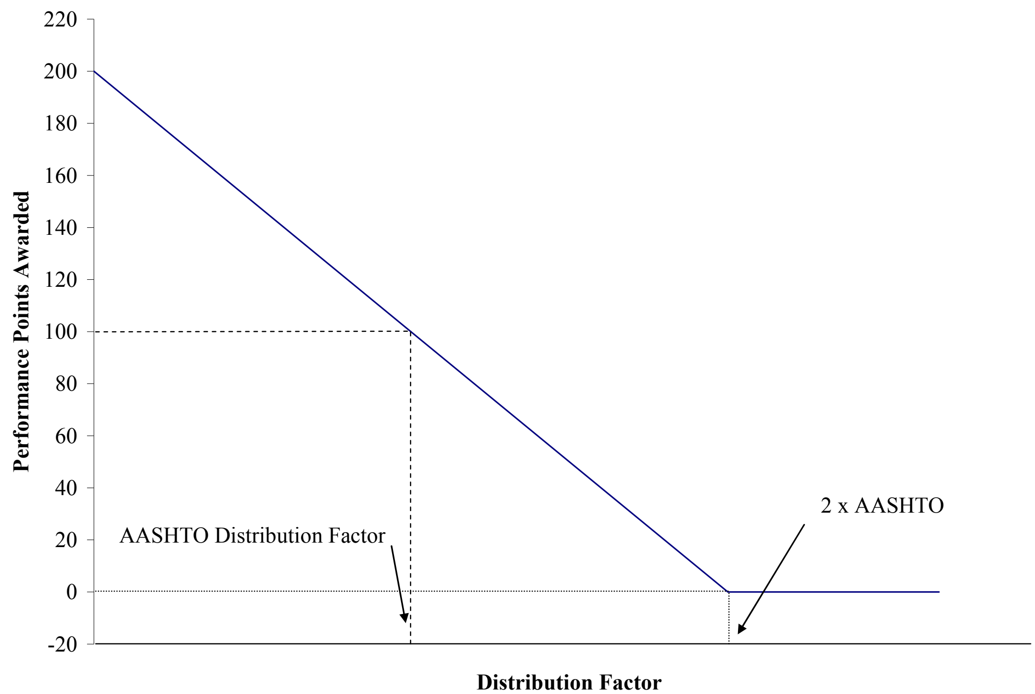


Figure 31. Level 3 Performance Parameter Points Awarded for Distribution Factor

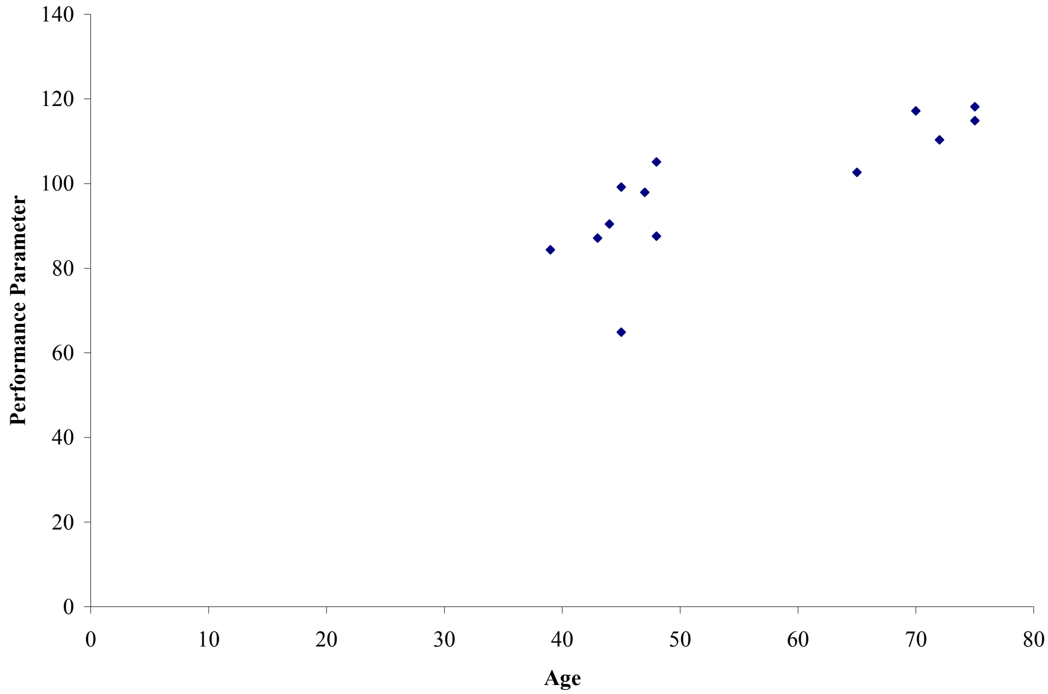


Figure 32. Total Scores After Level 3 Performance Determination

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. The results from the models were used to award points much like previous levels. Besides one added major category, several similarities exist between Levels 3 and 4 as 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 percent 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. Level 1, Level 2, Level 3, and Level 4 composed 10 percent, 20 percent, 30 percent, and 40 percent 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, Figure 29 through Figure 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 [33], the allowable bending stress for the girder timber can range from approximately 1150 psi to 1750 psi with respect to the wood grade. Because the exact grade of the girder timber is unknown, an average value of 1450 psi was selected as the maximum allowable stress. Analytical stress results at midspan were obtained and compared to the abovementioned 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 zero points were awarded. Conversely, if the maximum midspan stress was zero, the maximum 200 points were awarded. One can see the scale by which allowable bending stress points were awarded in Figure 33. 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.

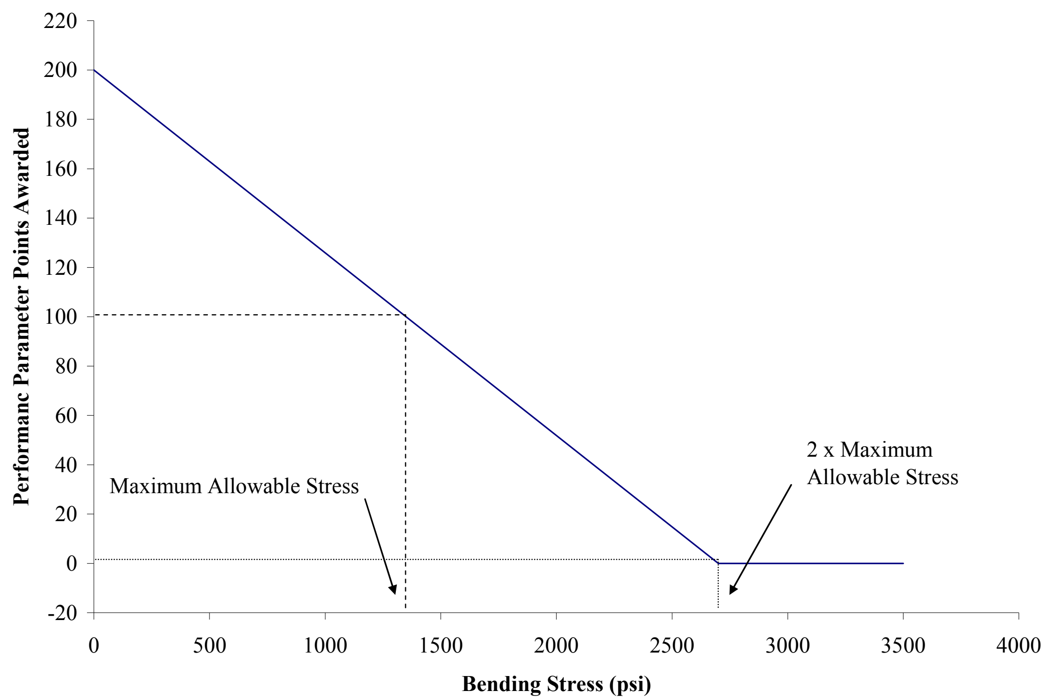


Figure 33. Level 4 Performance Parameter Points Awarded for Bending Stress

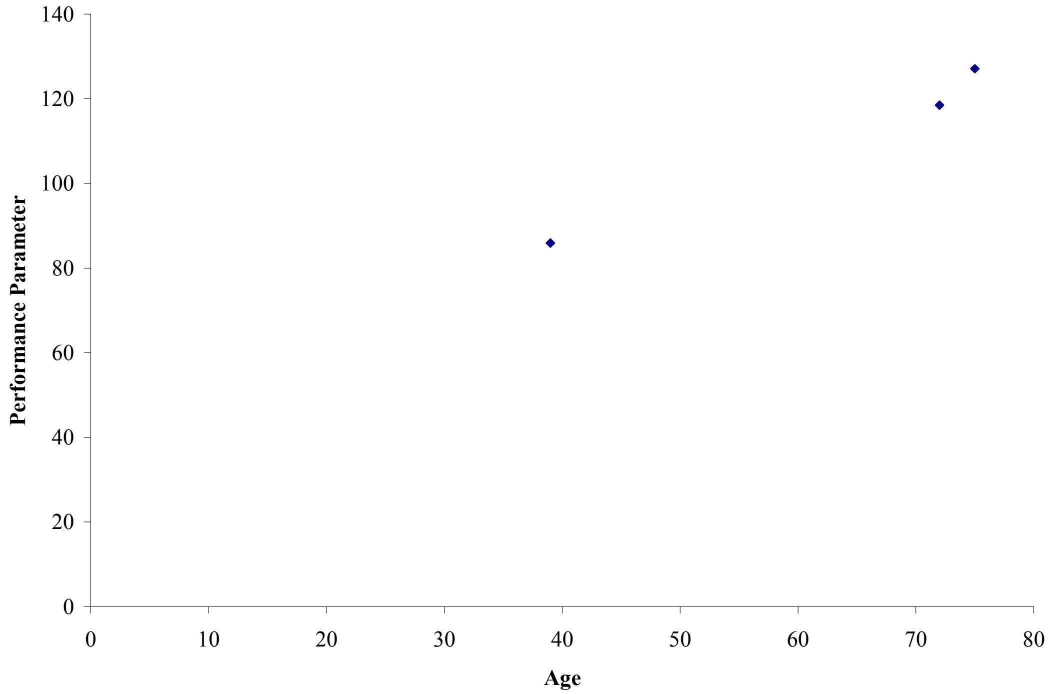


Figure 34. Total Scores After Level 4 Performance Determination

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. Figure 35 through Figure 37 show the change in performance with respect to age between Level 1 and Level 4, Level 2 and Level 4, and Level 3 and Level 4, respectively.

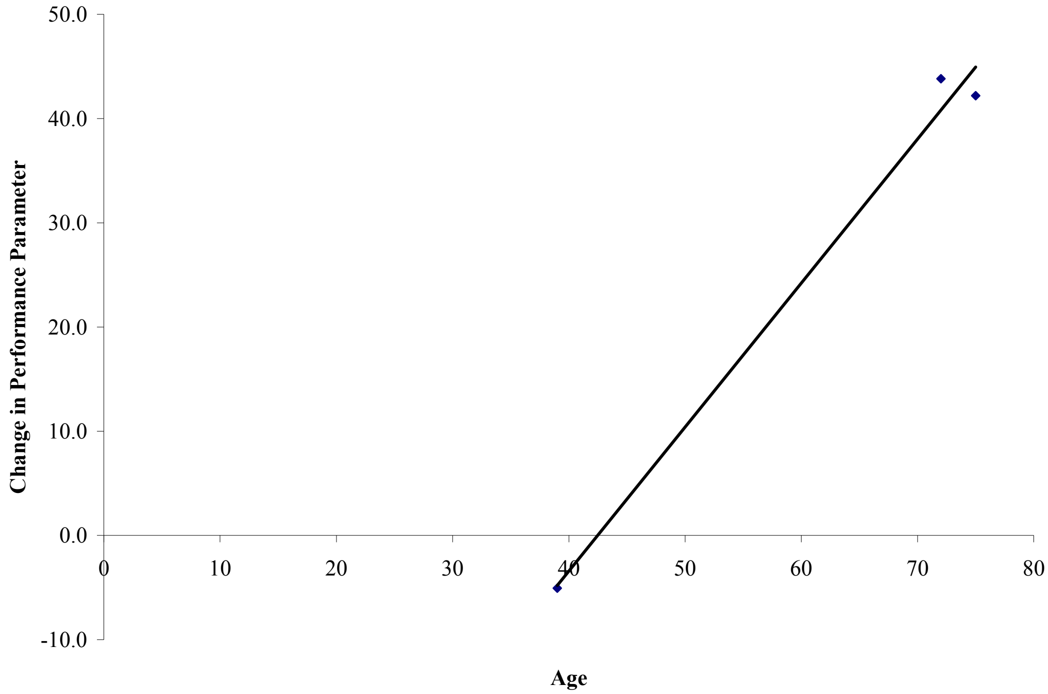


Figure 35. Change in Performance between Levels 1 and 4

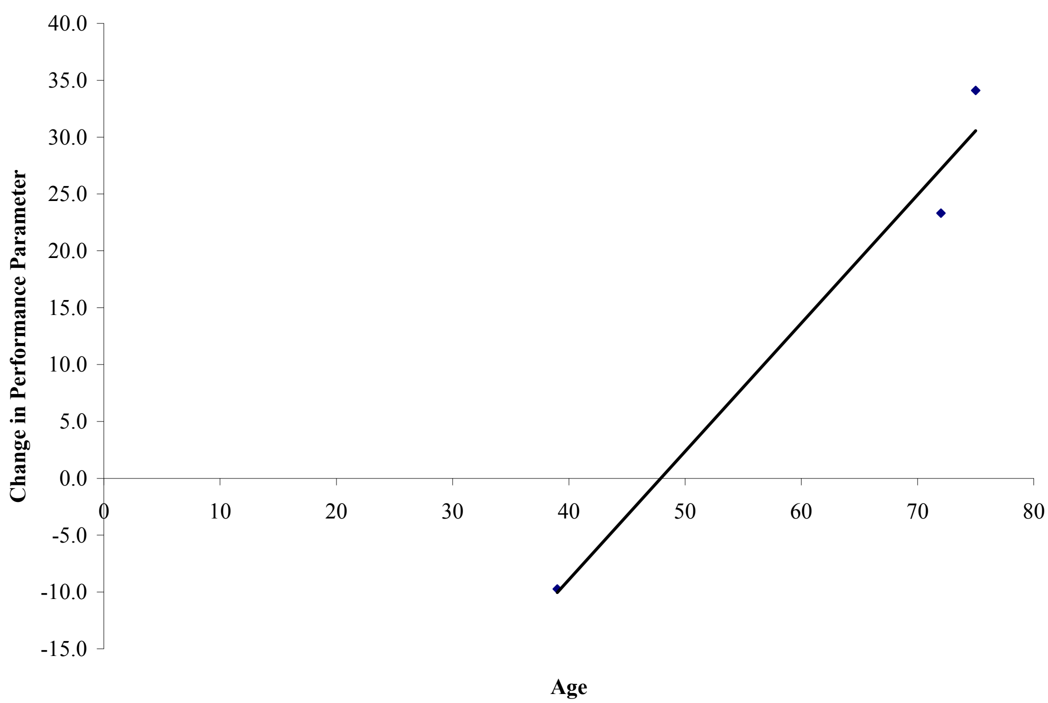


Figure 36. Change in Performance between Levels 2 and 4

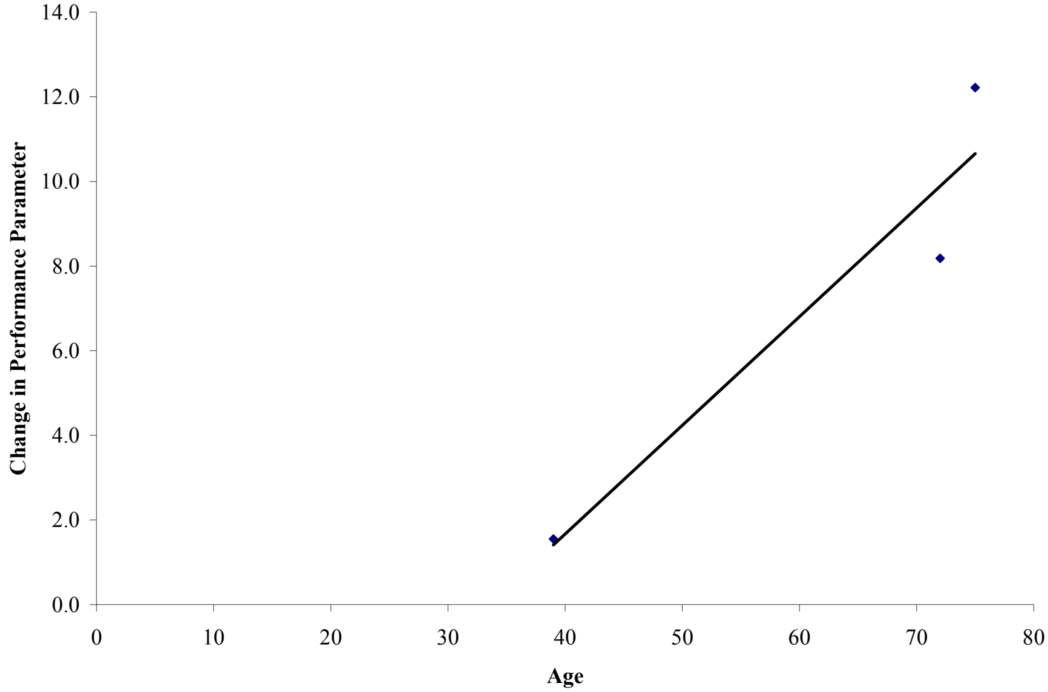


Figure 37. Change in Performance between Levels 3 and 4

Stated below are Equation 3 through Equation 5 which were the results from the trend lines shown in the above figures.

Equation 3. Level 4 Fleet Performance Derived from Level 1

$$L4 = L1 + f(L4 - L1)$$

where,

$$f(L4 - L1) = 1.382 \times Age - 58.69$$

Equation 4. Level 4 Fleet Performance Derived from Level 2

$$L4 = L2 + f(L4 - L2)$$

where,

$$f(L4 - L2) = 1.128 \times Age - 54.04$$

Equation 5. Level 4 Fleet Performance Derived from Level 3

$$L4 = L3 + f(L4 - L3)$$

where,

$$f(L4-L3) = 0.257 \times Age - 8.60$$

If one applies the respective transforming function to the Levels 1, 2, and 3 results, the plots shown in Figure 38 through Figure 40 are obtained.

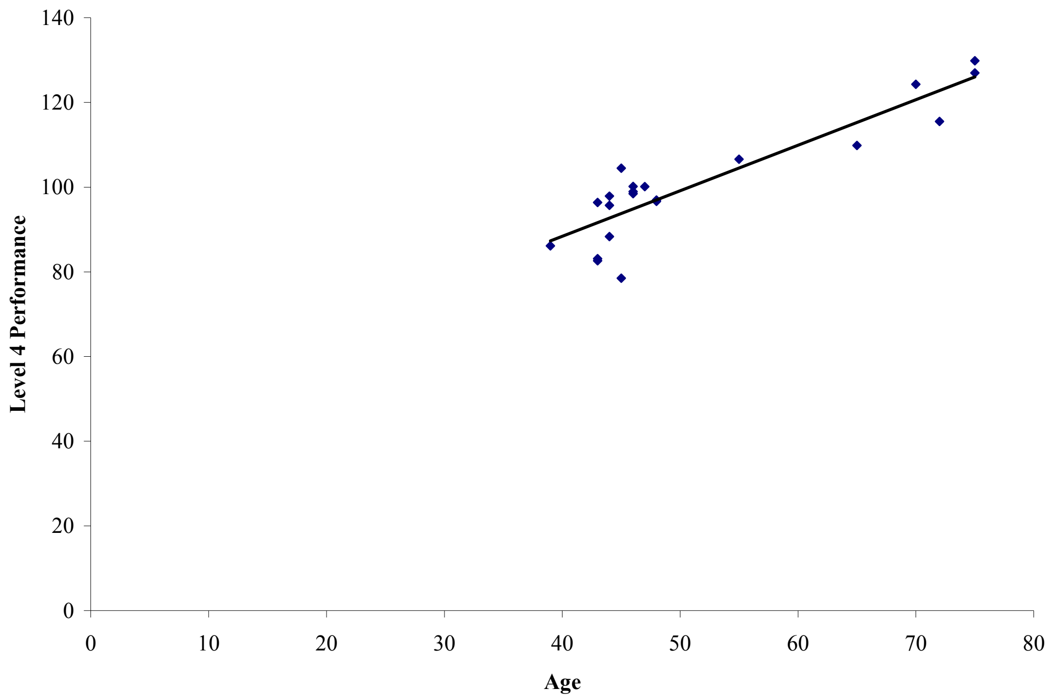


Figure 38. Level 4 Performance from Level 1

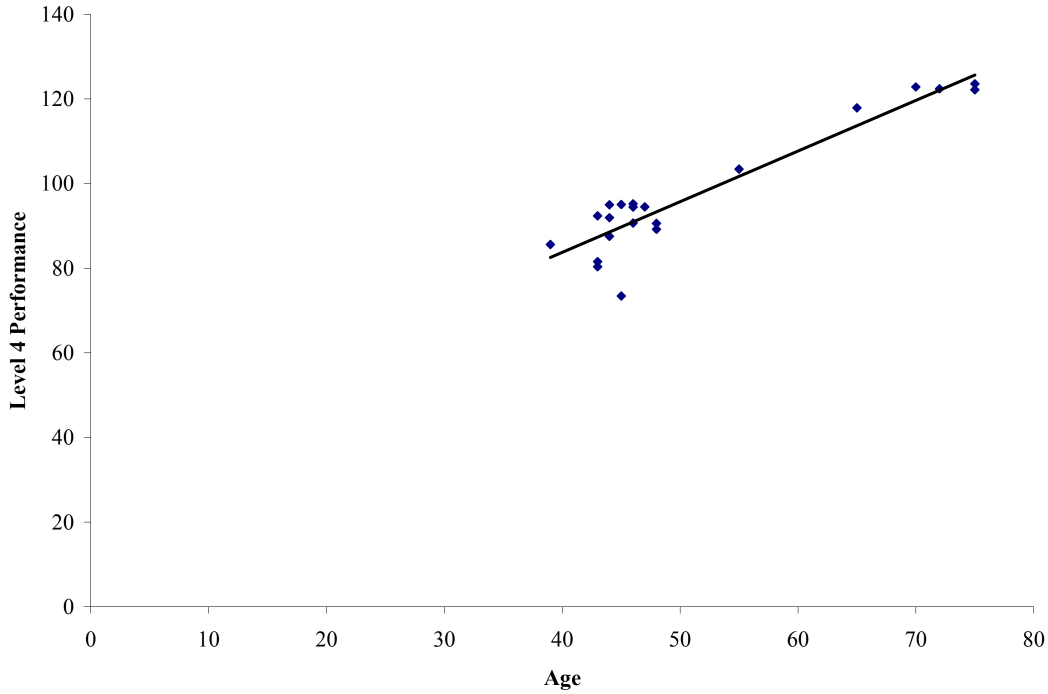


Figure 39. Level 4 Performance from Level 2

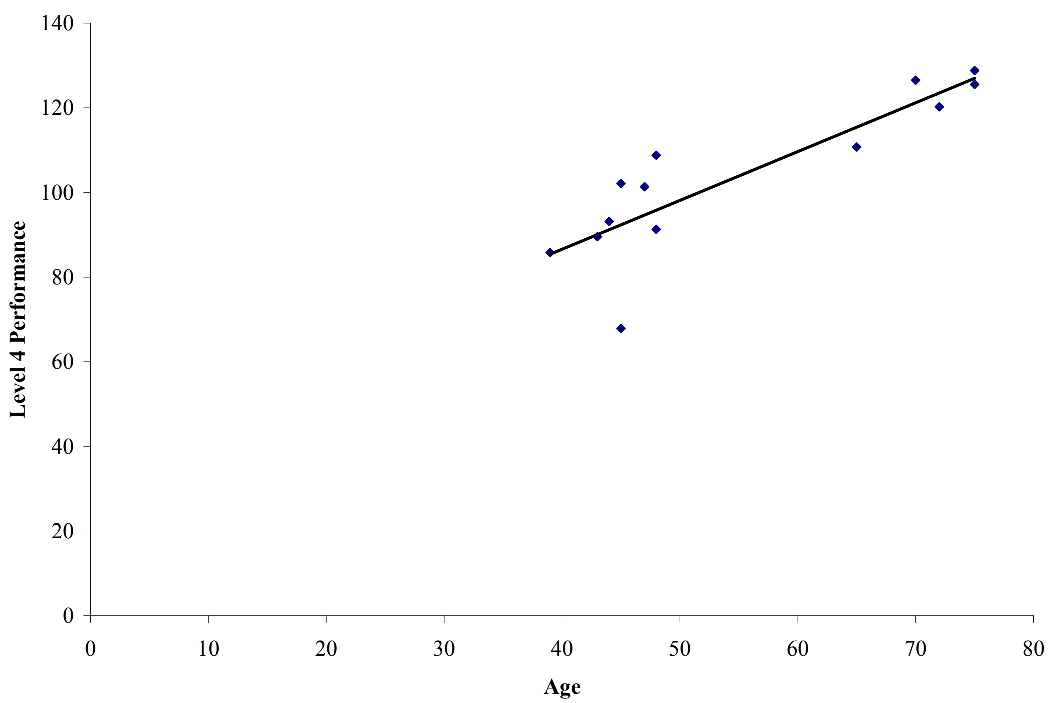


Figure 40. Level 4 Performance from Level 3

One would expect for 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 overlaying the Level 4 performance scores of the previous figures.

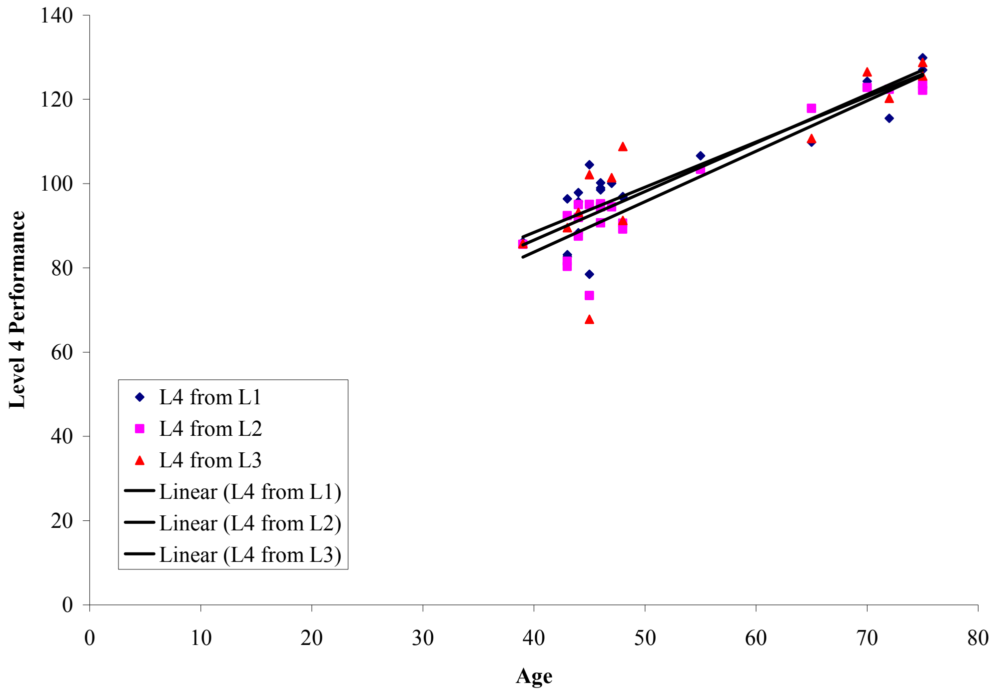


Figure 41. Level 4 Performance from Each Level

Engineering judgment says that performance should not increase over time, yet Figure 38 through Figure 41 shows 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 the bridge age a pattern was revealed. Coincidentally, the older bridges of group number 2 had girders of a greater moment of inertia than the younger bridges of group number 1 (see Figure 43) even though age and girder size are independent parameters. That is, the moment of inertia of the girders of bridges in group number 1 tends to be smaller than that of group number 2.

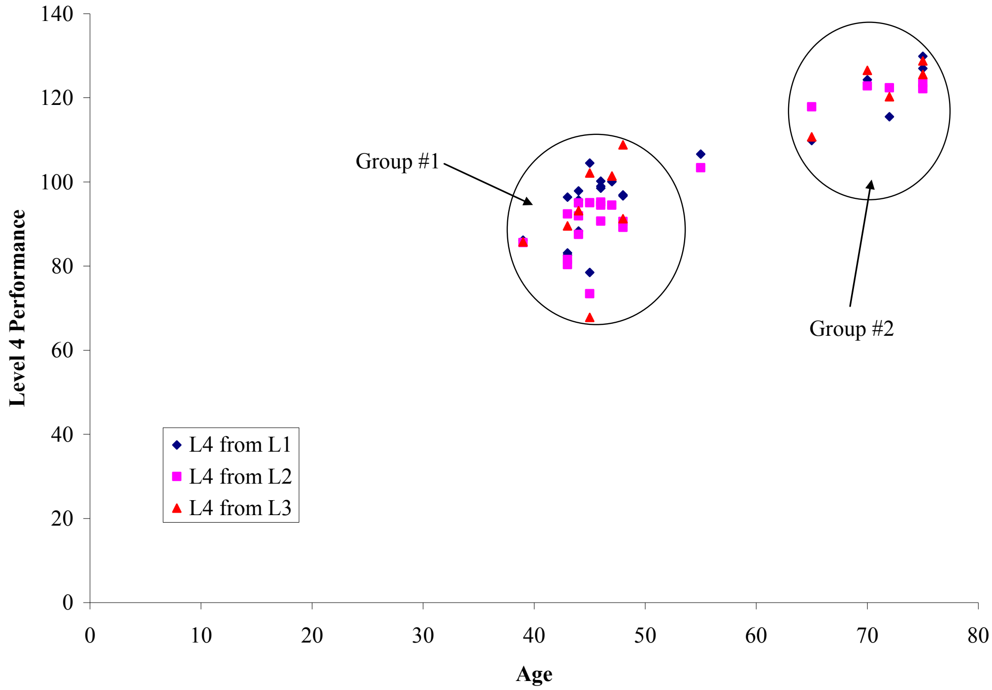


Figure 42. Data Groups within Level 4 Performance

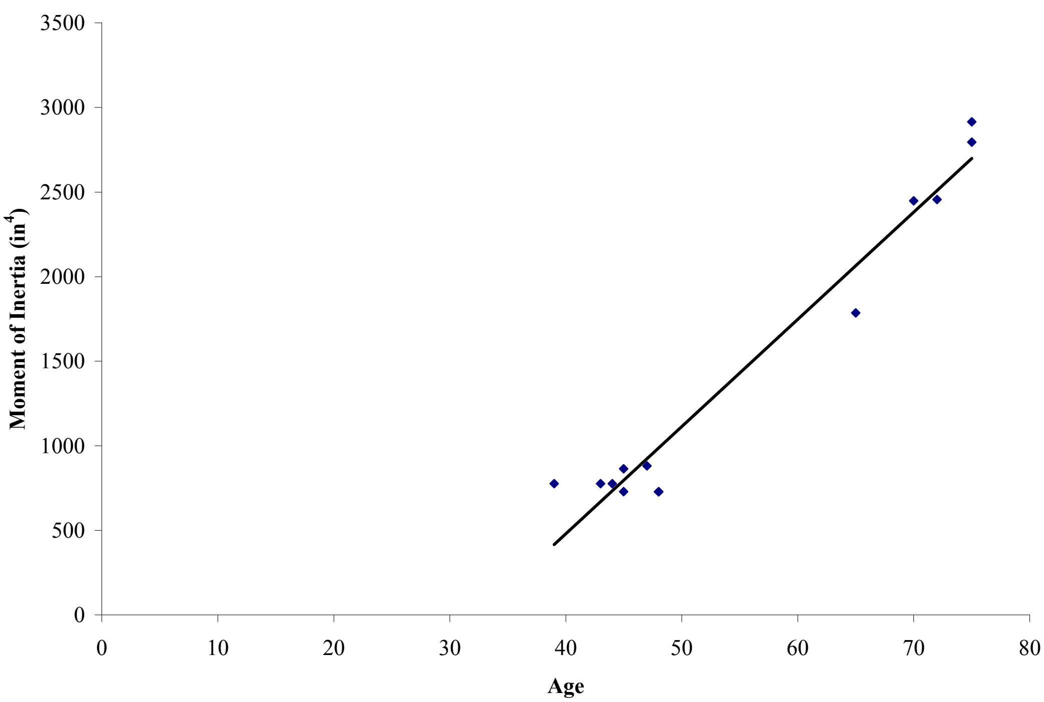


Figure 43. Relationship between Moment of Inertia and Age

Figure 44 shows the Level 4 performance plotted against the moment of inertia of the girders for each respective bridge. It appears that the overall bridge performance may be better in bridges with girders of greater moment of inertia.

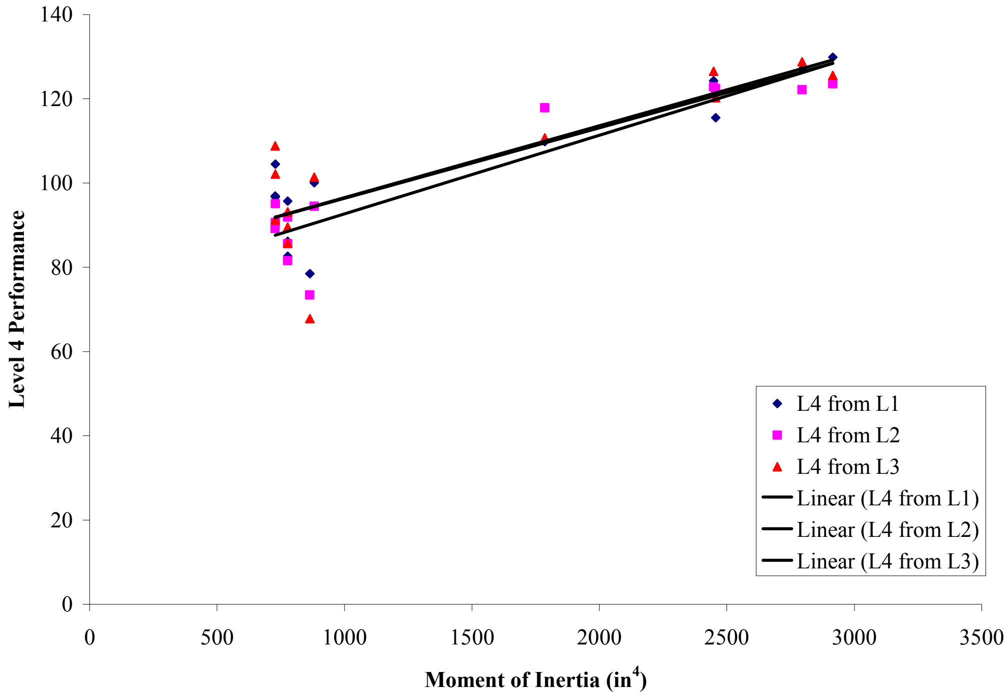


Figure 44. Level 4 Performance vs. Girder Moment of Inertia

Another confounding variable and common factor found between groups 1 and 2 of Figure 42 was the average superstructure moisture content. When the average superstructure moisture content was plotted against the age of the bridges a trend was identified (see Figure 45). Seemingly, as the age the bridge increases the 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 (see Figure 46). The Level 4 performance may increase with decreasing average superstructure moisture content. One should note that there was a strong correlation between moisture content and geographic location, as would be expected. This fact could prove useful in management practices as it is much less difficult to determine the geographic location of a bridge than the average superstructure moisture content.

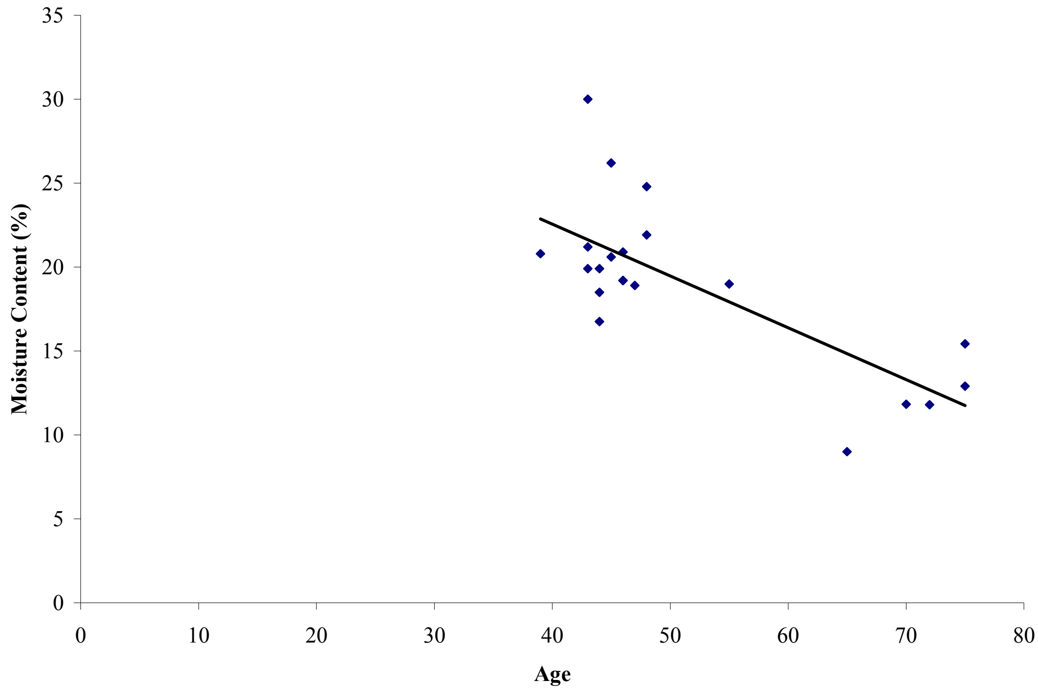


Figure 45. Average Superstructure Moisture Content vs. Age

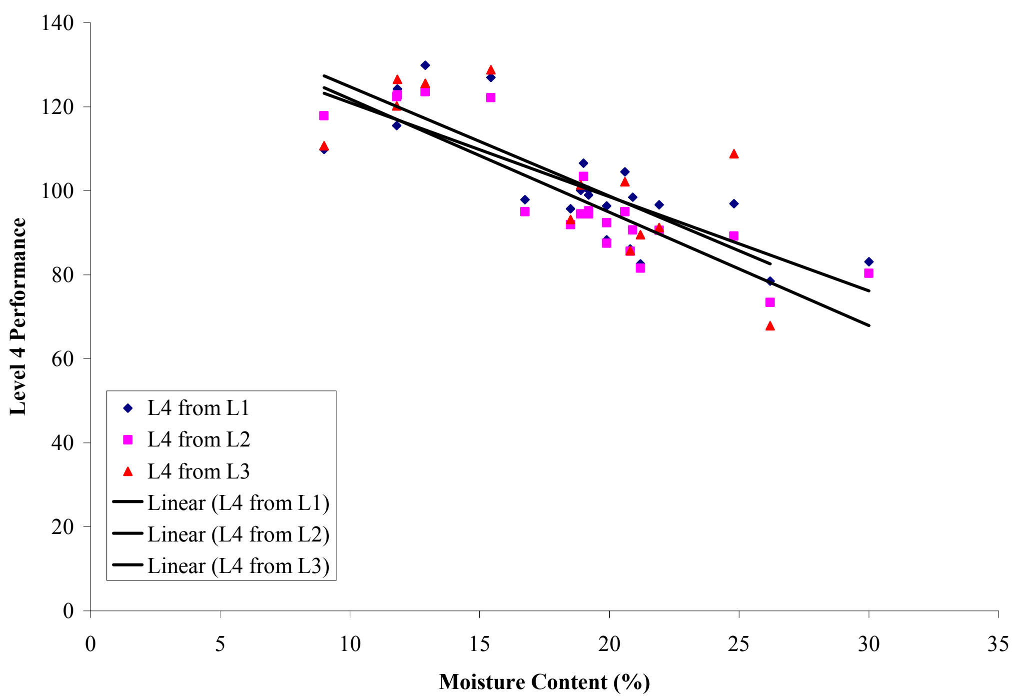


Figure 46. Level 4 Performance vs. Average Superstructure Moisture Content

The girder size and moisture content appear to be two factors governing the Level 4 performance. Even though a good correlation was found between the Level 4 performance and the 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 the performance should increase with a greater moment of inertia and lesser moisture as shown in Figure 44 and Figure 46. The aging of a bridge most certainly is still a factor among bridges with very similar girder sizes and moisture contents, so age should not be discarded as a governing factor.

One could conclude from the above figures that bridges with girders of a lesser moment of inertia will perform lower than bridges with girders of a higher moment of inertia and bridges with greater average superstructure moisture content will perform lower than bridges with lesser 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. One should note that moisture contents are not readily available as are the girder dimensions for each bridge; therefore the bridge owner may have to predict the average moisture content through climatology reports.

4 OBSERVATIONS, CONCLUSIONS, AND RECOMMENDATIONS OF FLEET MANAGEMENT CONCEPT

4.1 WHEN TO USE FLEET MANAGEMENT CONCEPT

As with any deterioration model one must determine the level at which it is efficient and/or necessary to take corrective action. With the information presented in the previous chapter, 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 non-scheduled 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.

4.2 LIMITATIONS OF FLEET MANAGEMENT CONCEPT

A number of limitations for the use of this specific fleet management concept to timber bridges exist and one should consider these limitations before using this concept. These limits are as follows:

- 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 was provided, it is beneficial to each bridge owner to carefully judge the applicability to his or her own fleet.
- A completely random sample was not used for the visual inspection and testing as 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, one should be aware of the sample when determining the applicability to his or her fleet.

- A significant amount of variability exists within timber products. Unlike steel or concrete where 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. It is thought that this point of interest may be negligible because most bridge owners maintain bridges within a region that is environmentally consistent.
- Along with differing regions comes past differing maintenance procedures and bridge management philosophies. Current maintenance practices may differ between fleets of bridges thereby creating differing 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

This research was performed with the objectives of optimizing system preservation activities by enhancing management approaches by taking advantage of structural similarities and better performance indicators; changing management decisions from being based upon 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. These tasks included: investigation of fleet management strategies within profit driven industries like 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 complete 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 is 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, enough positive evidence of the possible applicability of the concept was generated.

4.4 Recommendations

Initially this fleet management concept was intended for nation wide 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 are present 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 to be more distinct. Namely, a fleet could take into account location and maintenance jurisdiction and possibly other features not examined in this work.

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 the 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 creates a more similar fleet and eliminates a number of variables which are difficult to account for.

- More specifically, one may need to limit the fleets to be within a single maintenance jurisdiction to hopefully eliminate variability in maintenance practices.
- More specifically, one may need to limit the fleets to a single climate region to hopefully 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 (i.e., wearing surface, girders, etc.)
- One may consider a sample of bridges with greater age variance to identify a better performance vs. time curve.

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APPENDIX A

VISUAL INSPECTION FORM FOR RURAL TIMBER BRIDGES

VISUAL INSPECTION FORM FOR RURAL TIMBER BRIDGES

Part 1: General Information

1. Bridge Identification ____
2. Date ______________ Time ______________ Weather ______________________
3. Location ____

Part 2: Bridge Geometry

1. Size of Girders: Length ____________ Width ____________ Depth ____________
2. Number of Girders ________________ Girder Spacing _________________________
3. Girder End Conditions ____
4. Type of Deck ____
5. Size of Deck Boards ____
6. Bridge Length ___________ Width ____________
7. Depth of asphalt wearing surface _________

Part 3: Entire Structure Inspection

1. Are there changes in color of the wood – brown or white, sunken faces, staining or discoloration?
Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ____

____

____

2. Is there vegetation in splits and cracks?
Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ____

____

3. Is there rapid absorption of water and odor like anise or wintergreen?

Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ________________________________

4. Is there excessive sagging or crushing of timber?

Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ________________________________

5. Is there holes, frass, or powder posting present?

Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ________________________________

6. Is there knots, sloped grains, or cracks present?

Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ________________________________

7. Has the bridge been damaged mechanically, i.e. vehicle abrasion, vehicle overload, foundation settlement, or debris in stream channel?

Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ________________________________

8. Is there ultraviolet light degradation present, i.e. darkening or lightening of light or dark woods, respectively?

Deck ☐ Yes ☐ No Superstructure ☐ Yes ☐ No Substructure ☐ Yes ☐ No

Remarks: ________________________________

9. Is there corrosion near metal fasteners?
   Deck Yes No   Superstructure Yes No   Substructure Yes No

   Remarks: ____

Part 4: Deck Inspection

1. Is there detachment of the deck boards from girders? Yes No

   Remarks: ____

2. Is the deck surface uneven? Yes No

   Remarks: ____

3. Are there cracks, holes, or delaminating in the wearing surface? Yes No

   Remarks: ____

4. Is the deck completely secured to the girders? Looseness of deck boards? Yes No

   Remarks: ____

5. Describe the pavement approach conditions.

Part 5: Superstructure Inspection

1. Is there abrasion and deterioration between the deck and girders? ☐ Yes ☐ No

   Remarks: ______________________________

2. Is there surface materials and drainage filtering through the floor system? ☐ Yes ☐ No

   Remarks: ______________________________

5. Is the bearing area on the support sufficient for all girders? ☐ Yes ☐ No

   Remarks: ______________________________

6. Are any girders vertically or transversely misaligned? ☐ Yes ☐ No

   Remarks: ______________________________

7. Is there any abnormal girder behavior present? ☐ Yes ☐ No

   Remarks: ______________________________

8. Is there any looseness of fasteners? ☐ Yes ☐ No

   Remarks: ______________________________

Part 6: Substructure Inspection

General Comments: ____

____

____

____

____

____

Part 7: Moisture Content

Location 1 ____ Reading ________

Location 2 ____ Reading ________

Location 3 ____ Reading ________

Location 4 ____ Reading ________

Location 5 ____ Reading ________

Location 6 ____ Reading ________

Location 7 ____ Reading ________

Location 8 ____ Reading ________

Location 9 ____ Reading ________

Location 10 ____ Reading ________

Location 11 ____ Reading ________

Location 12 ____ Reading ________

APPENDIX B

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 860131

United States
Department of
Agriculture

Forest Service

Forest Products
Laboratory

Iowa State
University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 860131

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

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 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.

Acknowledgements

We would like to 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.

Chris Lee, North Carolina Department of Transportation employee who organized the load testing.

Donny Warren, North Carolina Department of Transportation employee who operated the load truck during testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Chestnut Cove Creek Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 860131, hereinafter referred to as the Chestnut Cove Creek Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 1. Chestnut Cove Creek Bridge in North Carolina

The Chestnut Cove Creek Bridge was built in 1962 and is located in Swain County in western North Carolina 0.1miles south of junction SR1177 across Chestnut Cove 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 Figure 2 and Figure 3.

Figure 2. Chestnut Cove Creek Parapet Support

Figure 3. 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 in. x 11-3/4 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 4 in. x 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 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.

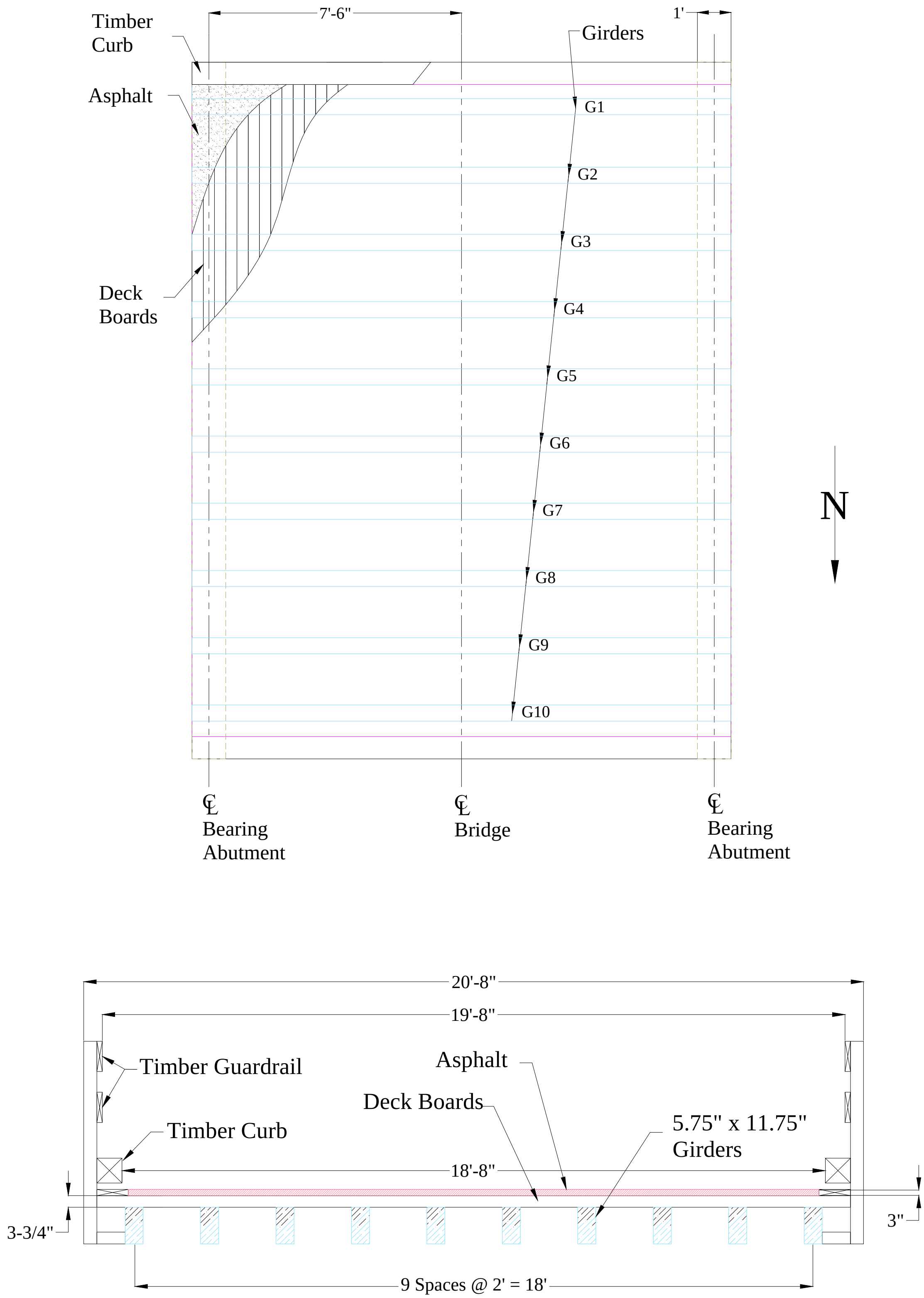

Figure 4. Plan and Profile Layout of Chestnut Cove Creek Bridge

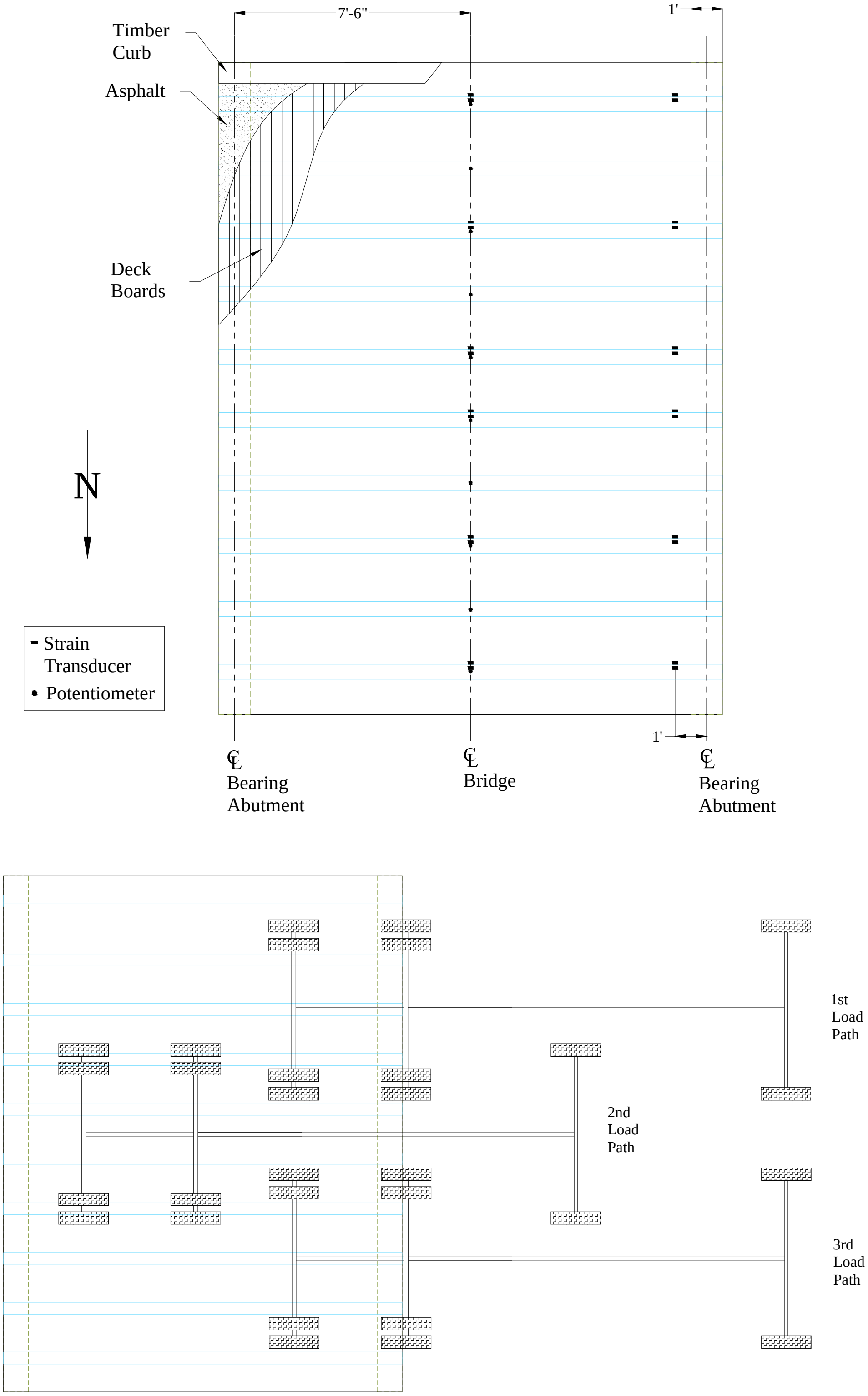

Figure 5. Instrumentation and Load Paths of Chestnut Cove Creek Bridge

Figure 6. Deflection Instrumentation

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 Figure 5). 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 Figure 7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 7. Strain Transducers

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.

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 8. The rear wheel base was 6 ft-0in.; 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 as a scale was not accessible during testing. The driver approximated the total weight of the truck and load to be 45,000 lbs. Typically, 70 percent 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 lbs and 13,500 lbs, respectively. A truck similar to the one shown in Figure 9 was used for the load test.

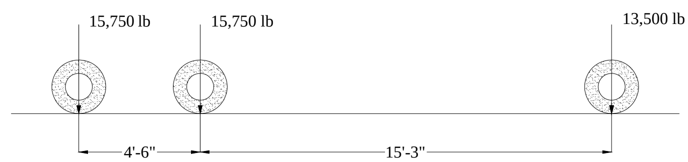

Figure 8. Truck Configuration and Axle Loads

Figure 9. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

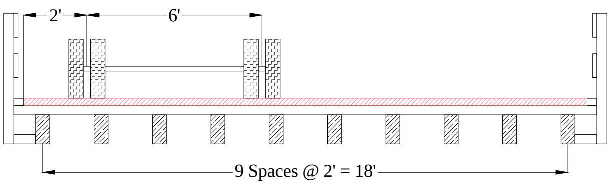

Figure 10. Transverse Truck Position - Load Path 1

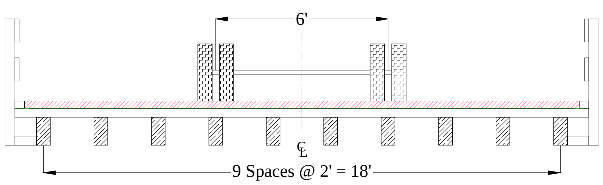

Figure 11. Transverse Truck Position - Load Path 2

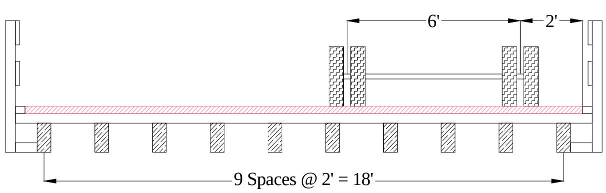

Figure 12. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 ensuing.

Wearing Surface

According to the NC-DOT 2005 report, scattered transverse cracks were present at the floor board seems. 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 Figure 13). 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, 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.

Figure 13. Pavement Chipping at West End

Deck

The deck appeared to be in good condition and 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. 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 has also formed on the faces of many of the girders and is assumed to be biotic growth from high moisture conditions. The residue does not appear to permeate the girder, however. Figure 14 shows a typical example of white residue.

Figure 14. Biotic Growth on Girders

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

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, 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. Moss growth was prevalent at the water line at the bottom of the northeast substructure posts (see Figure 15). There were minor checks in the parapet and parapet curb.

Figure 15. Moss Growth at Base of Substructure Posts

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.

Time-History Deflections

Figures 16 through 18 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

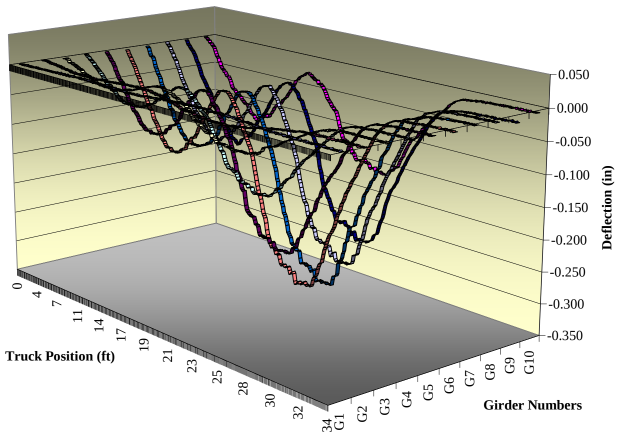

Figure 16. Deflections vs. Truck Position for Load Path 1

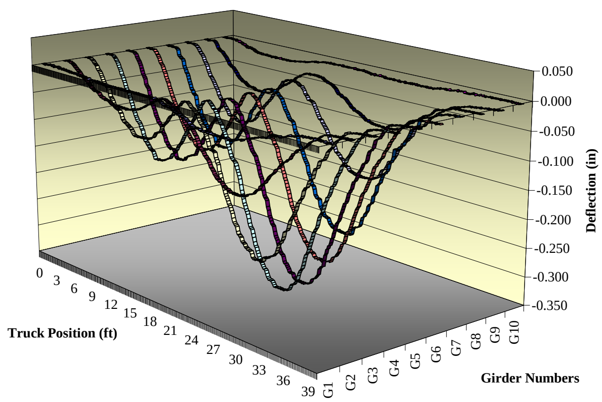

Figure 17. Deflections vs. Truck Position for Load Path 2

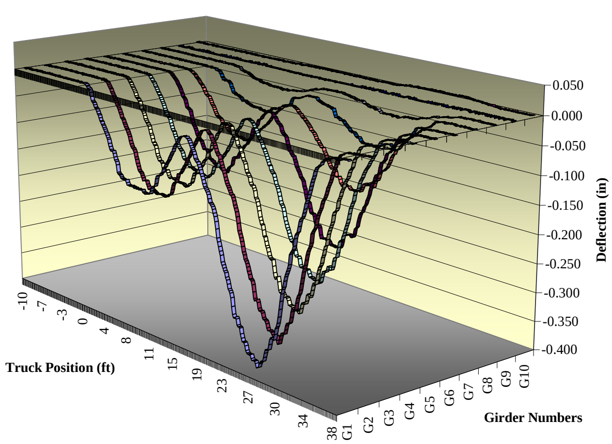

Figure 18. Deflections vs. Truck Position for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 19 through 21. 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.

Table 1. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.318	0.334	0.390

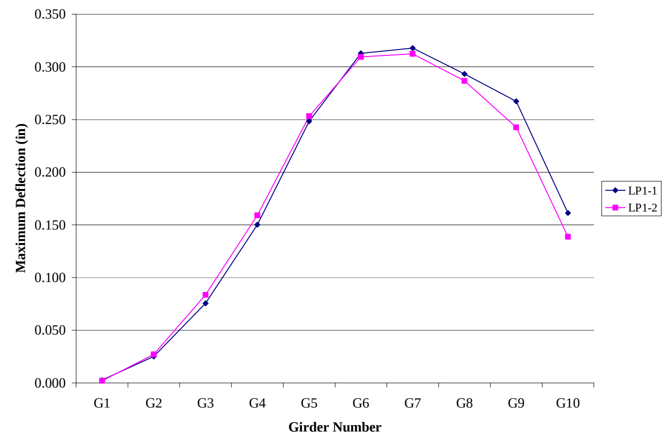

Figure 19. Maximum Deflections for Load Path 1

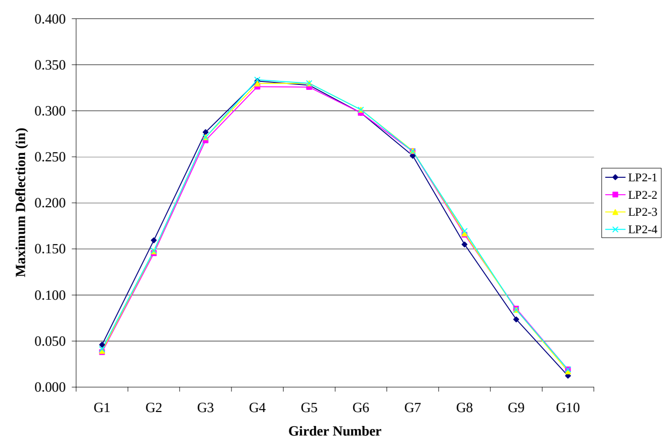

Figure 20. Maximum Deflections for Load Path 2

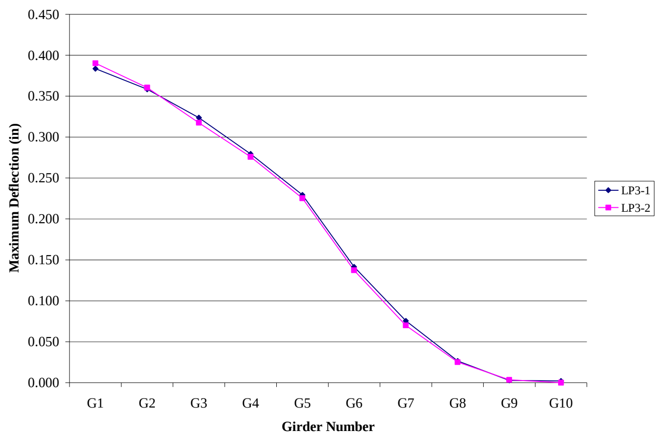

Figure 21. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 2. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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

Equation 1

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 3. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
185	177	178

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 of the n-values are below the recommended n-values stated in Table 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 2

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

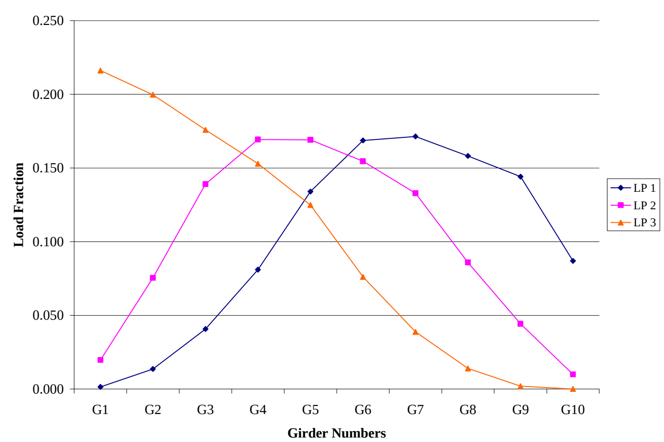

Figure 22. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

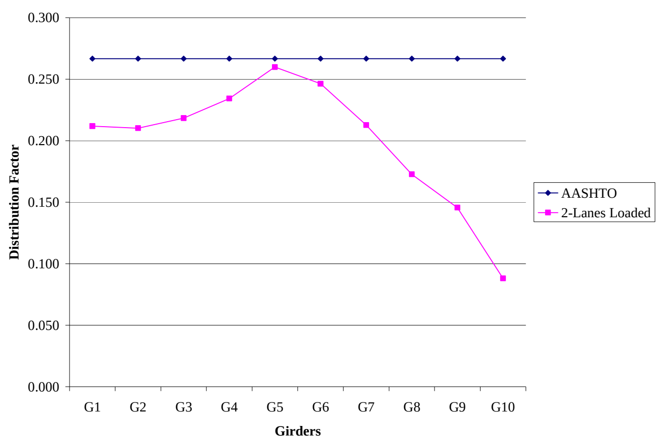

Figure 23. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

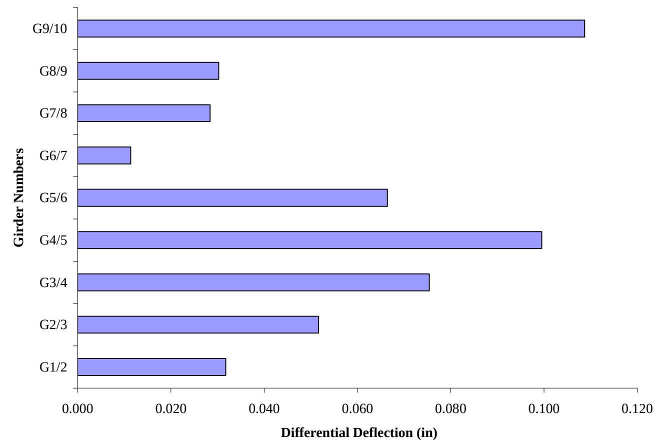

Figure 24. Differential Deflections for Load Path 1

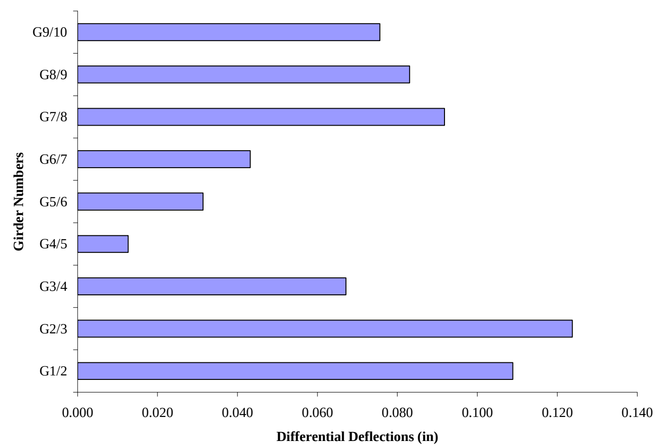

Figure 25. Differential Deflections for Load Path 2

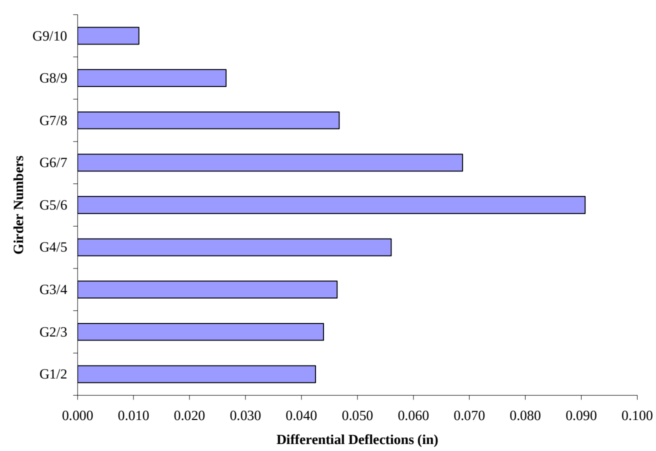

Figure 26. Differential Deflections for Load Path 3

Table 4. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.109	0.125	0.092

The maximum differential deflection of 0.125 in. occurs in load path 2. This is nearly 38 percent 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 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 27 through 29 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 through 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, i.e., the girders act independently of the deck when subjected to bending.

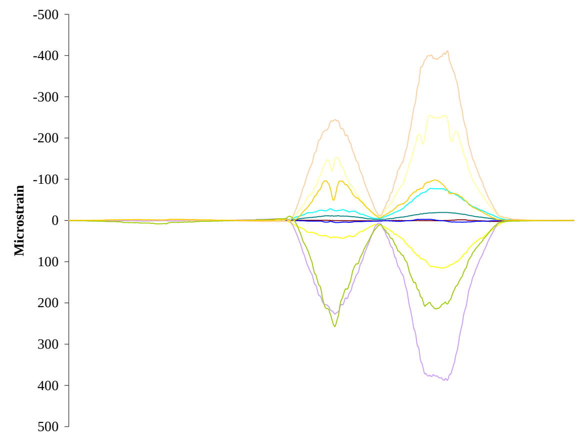

Figure 27. Strain at Midspan for Load Path 1

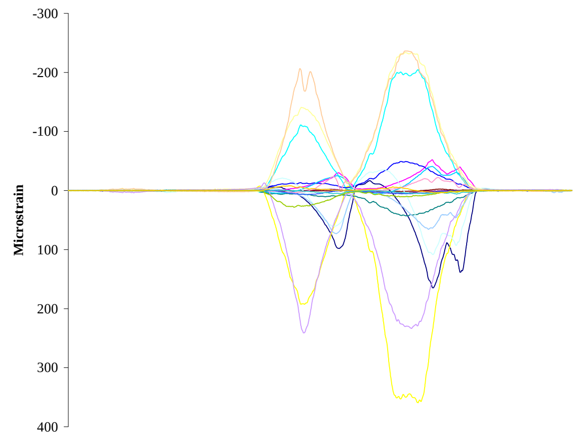

Figure 28. Strain at Midspan for Load Path 2

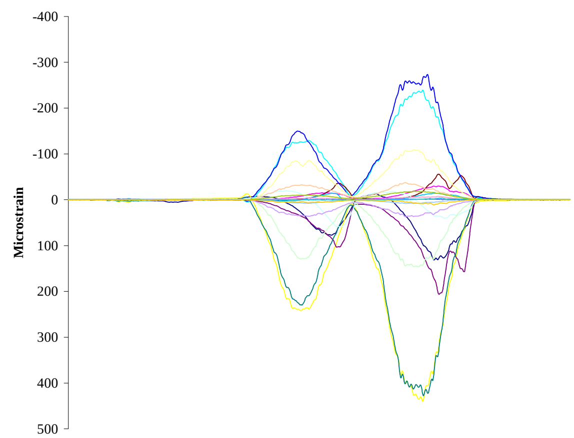

Figure 29. Strain at Midspan for Load Path 3

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 percent. Overall, significant moisture content differences were not found throughout the bridge. The moisture content measurements are summarized in Table 5.

Table 5. Moisture Content Summary

Moisture Content Reading Locations and Values	
Location	%
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 & 7	22.8

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 0.023 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, i.e., 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 27 though 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 403 and 437 microstrain, respectively. These strains equate to maximum stresses of 463 and 503 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 736 and 798 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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 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; 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. 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 1500 psi 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.

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APPENDIX C

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 560460

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 560460

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 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.

Acknowledgements

We would like to 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.

Brett Rhindhart, North Carolina Department of Transportation employee who assisted during the load testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Madison County Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 860131, hereinafter referred to as the Madison County Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 30. Madison County Bridge in North Carolina

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 (see Figure 31).

Figure 31. Substructure Sill and Columns

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 3.

Figure 32. Madison County Parapet

Girders measure 14 ft-5 in. from end to end and have a clear span of 12 ft-5 in. A total of 9 girders, spaced 2 ft center-to-center, measuring 5-3/4 in. x 11-1/2 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 4 in. x 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 asphalt wearing surface. Figure 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 9 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.

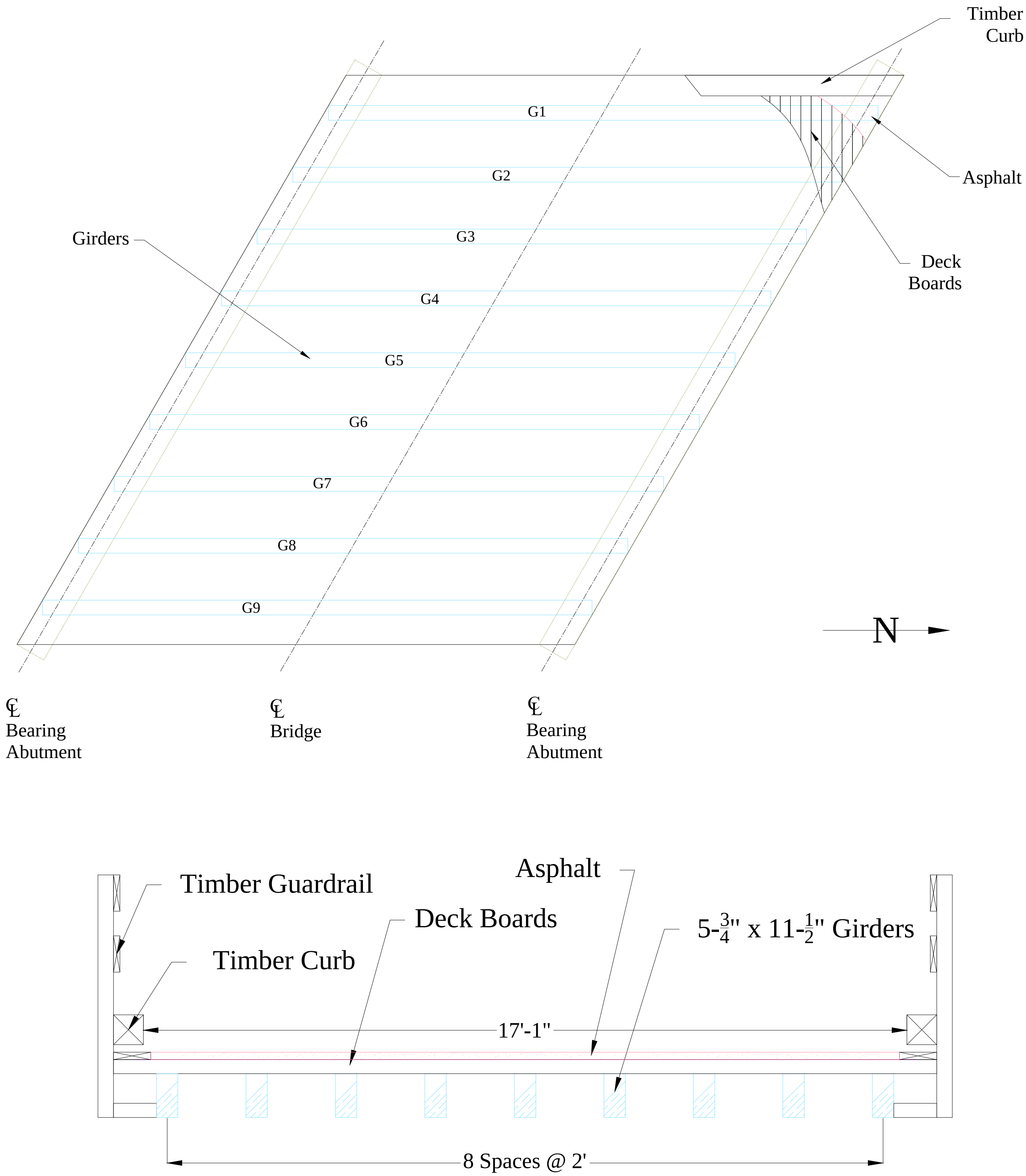

Figure 33. Plan and Profile Layout of Madison County Bridge

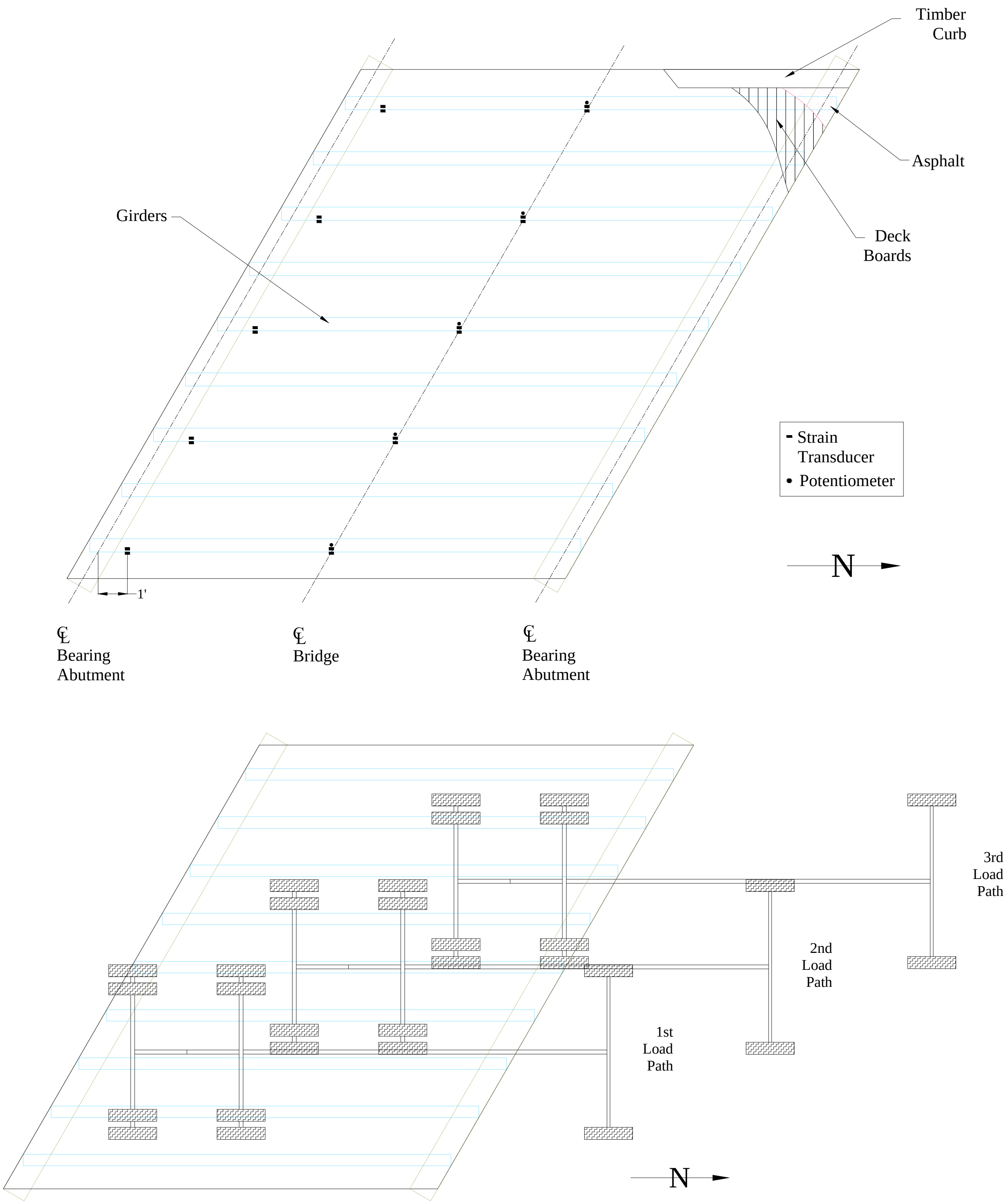

Figure 34. Instrumentation and Load Paths of Madison County Bridge

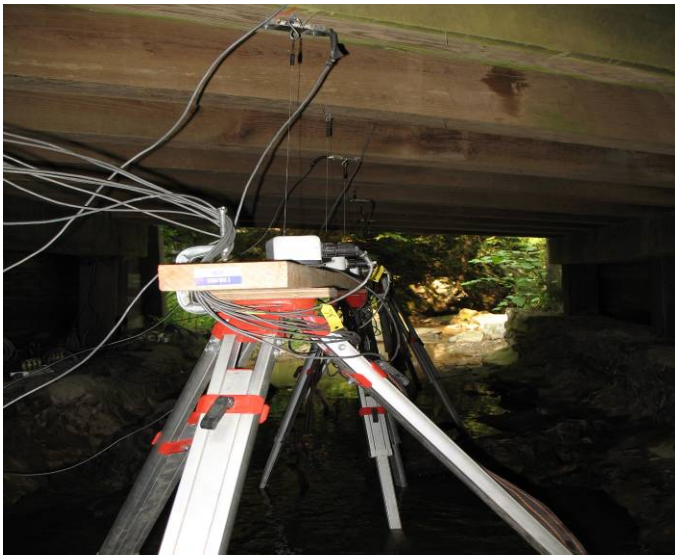

Figure 35. Deflection Instrumentation

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 bridge. The midspan and one abutment were instrumented (see Figure 5). 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 Figure 36). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 36. Strain Transducers

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.

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 8. 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 lbs. The load over the front axle was 16,280 lbs and, assuming that the load over each rear axle was equal, the load was 19,060 lbs over each rear axle. Figure 9 shows the truck used for the load testing.

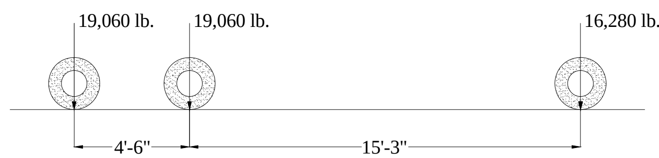

Figure 37. Truck Configuration and Axle Loads

Figure 38. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

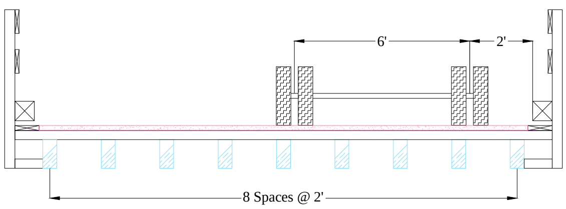

Figure 39. Transverse Truck Position - Load Path 1

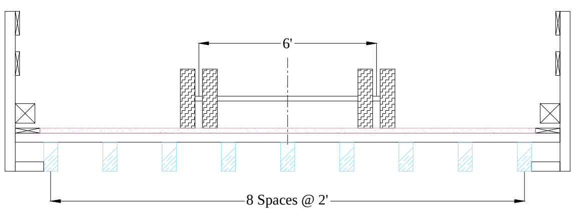

Figure 40. Transverse Truck Position - Load Path 2

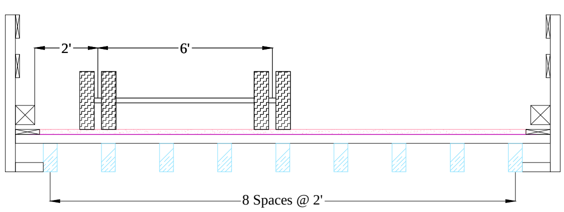

Figure 41. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 Figure 13). 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 Figure 43). Imminent decay is not likely, though these cracks could be cause for concern in the future

Figure 42. Pavement Chipping at West End

Figure 43. Transition Between Roadway and Bridge

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 note worthy observation was the checking found along the centerline of several girders. This checking is seen in Figure 44 and Figure 45. 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.

Figure 44. Checking at Centerline of Girder

Figure 45. Checking at Centerline Near Abutment

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, 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. 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.

Time-History Deflections

Figures 17 through 19 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

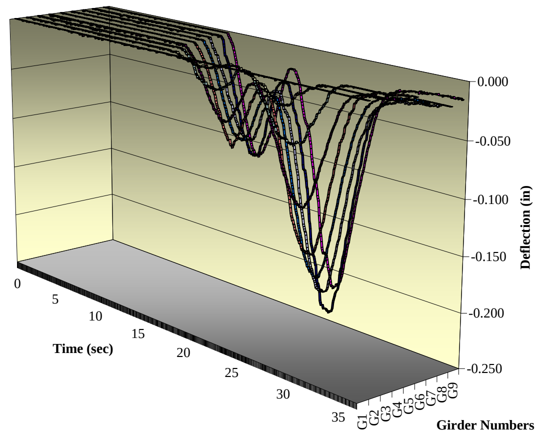

Figure 46. Deflections Load Path 1

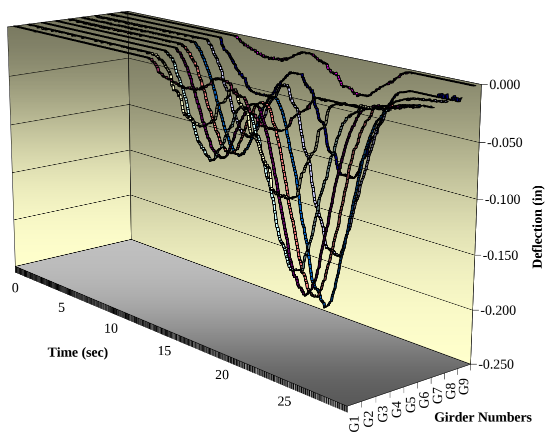

Figure 47. Deflections Load Path 2

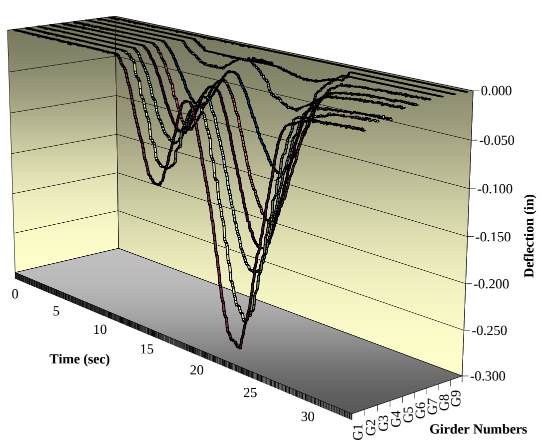

Figure 48. Deflections Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 20 through 22. 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.

Table 6. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.233	0.218	0.285

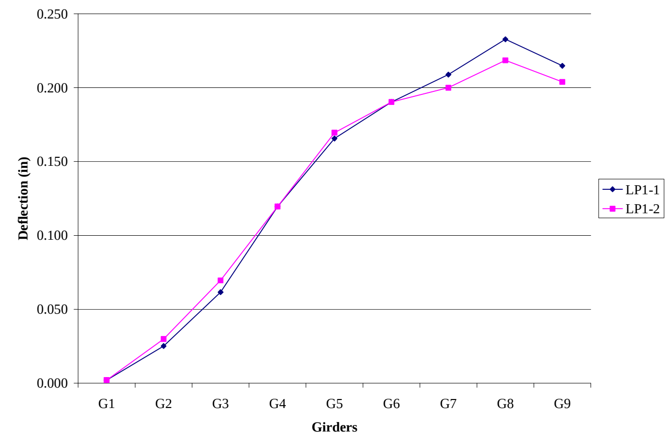

Figure 49. Maximum Deflections for Load Path 1

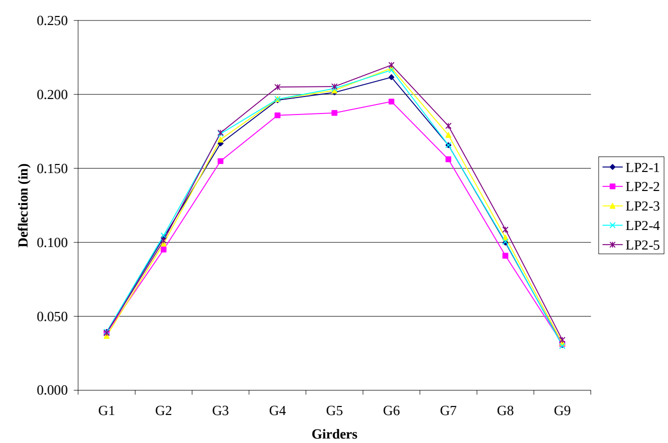

Figure 50. Maximum Deflections for Load Path 2

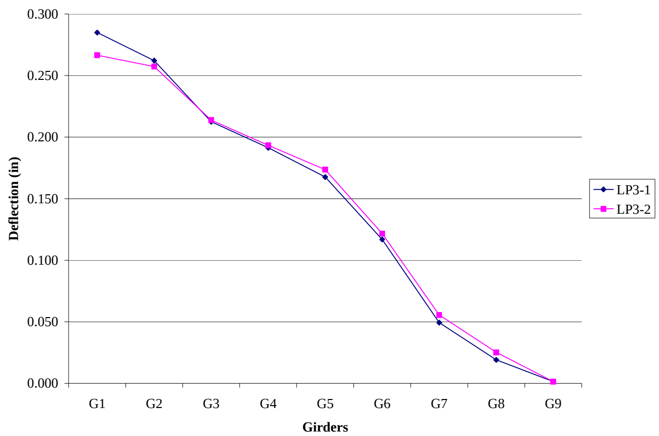

Figure 51. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 7. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 3

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 8. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
504	539	412

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 [8]. Load path 1 and 2 minimum n-values are greater than the recommended value from all three sources.

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 Equation 2.

Equation 4

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

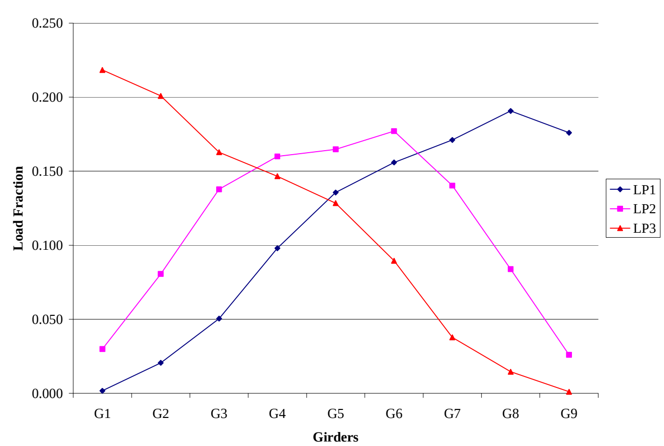

Figure 52. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

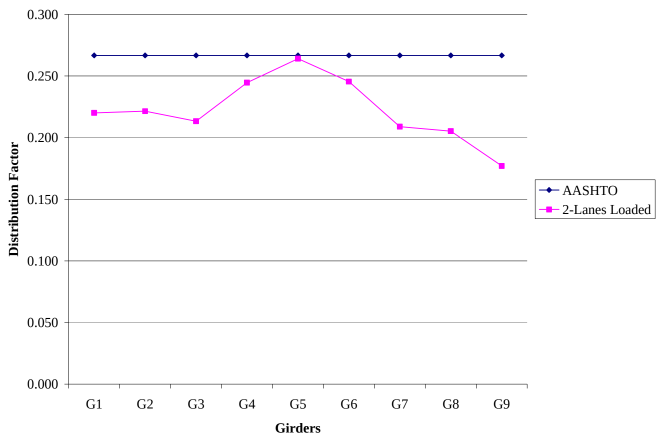

Figure 53. AASHTO Design Live Load Distribution

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. Figures 25 through 27 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 4.

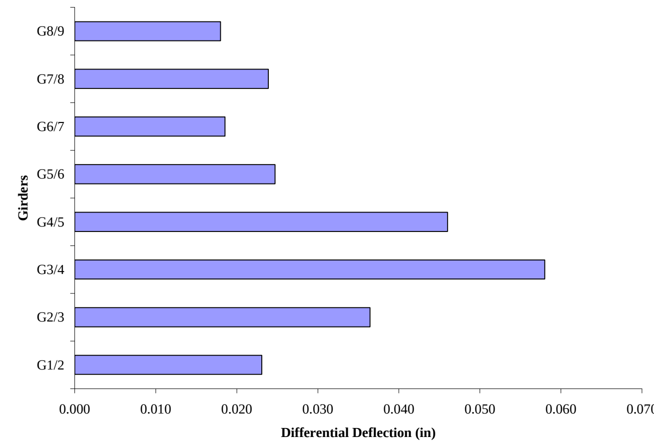

Figure 54. Differential Deflections for Load Path 1

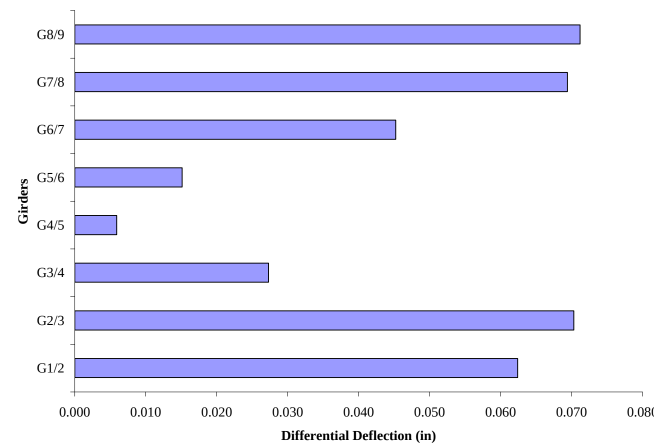

Figure 55. Differential Deflections for Load Path 2

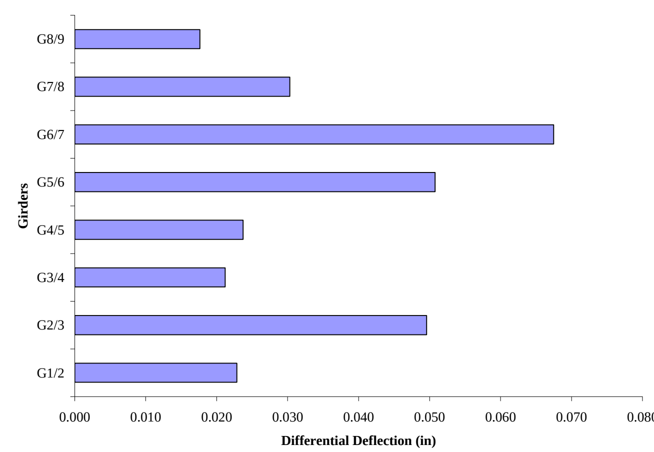

Figure 56. Differential Deflections for Load Path 3

Table 9. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.058	0.071	0.067

The maximum differential deflection of 0.071 in. occurs in load path 2. This is nearly 33 percent 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 28 through 30 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 28 through 30 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, i.e., the girders act independently of the deck when subjected to bending.

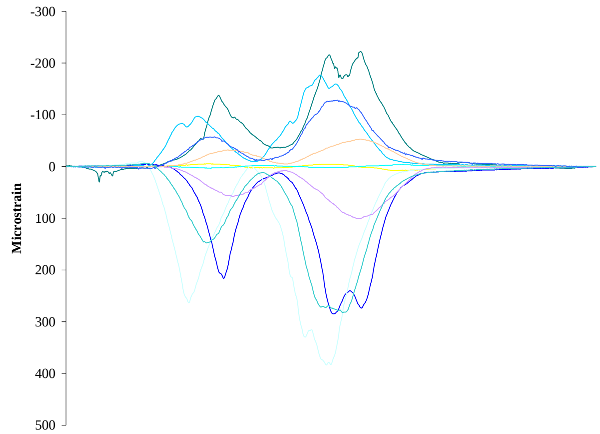

Figure 57. Strain at Midspan for Load Path 1

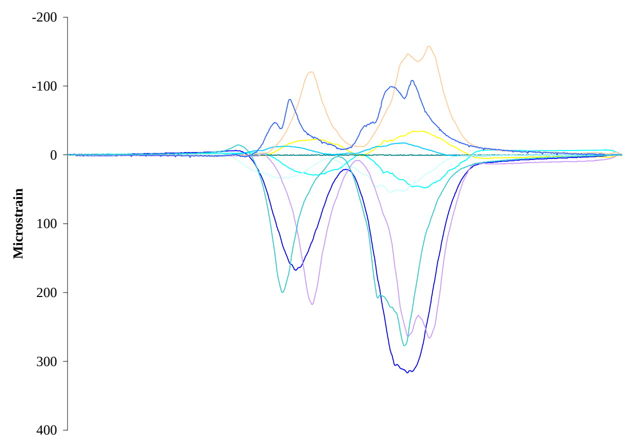

Figure 58. Strain at Midspan for Load Path 2

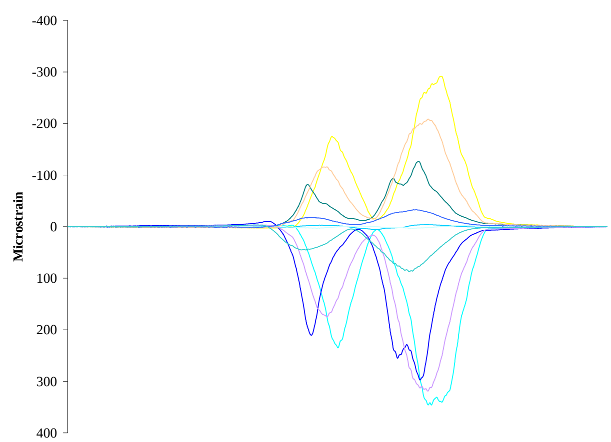

Figure 59. Strain at Midspan for Load Path 3

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 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 percent. 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 5.

Table 10. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	21.0
Bottom of Deck Between Girders 5 & 6	26.2
Bottom of Deck Between Girders 8 & 9	23.9

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 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, 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, i.e., 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 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 as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures 28 though 30 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 335 and 580 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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 1500 psi 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.

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APPENDIX D

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 560510

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 560510

Terry Wipf
Brent Phares
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Michael Ritter
Justin Dahlberg

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 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.

Acknowledgements

We would like to 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.

Brett Rhindhart, North Carolina Department of Transportation employee who assisted in the static load testing

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Red Horse Creek Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 560510, hereinafter referred to as the Red Horse Creek Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

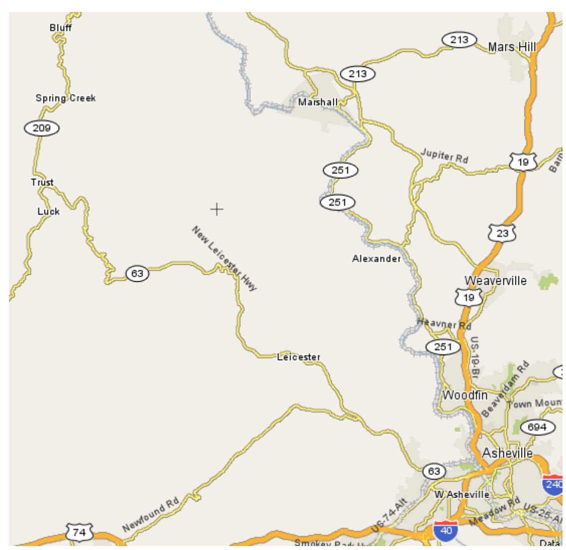

Figure 60. Bridge Location

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 61, 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 Figure 62).

Figure 61. Red Horse Creek Bridge

Figure 62. 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 in. x 11-3/4 in. in cross-section are present and are seated and toe-nailed to the 12 in. x 12 in. timber sill with spikes. The deck consists of individual 4 in. x 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 63 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.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 64. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.

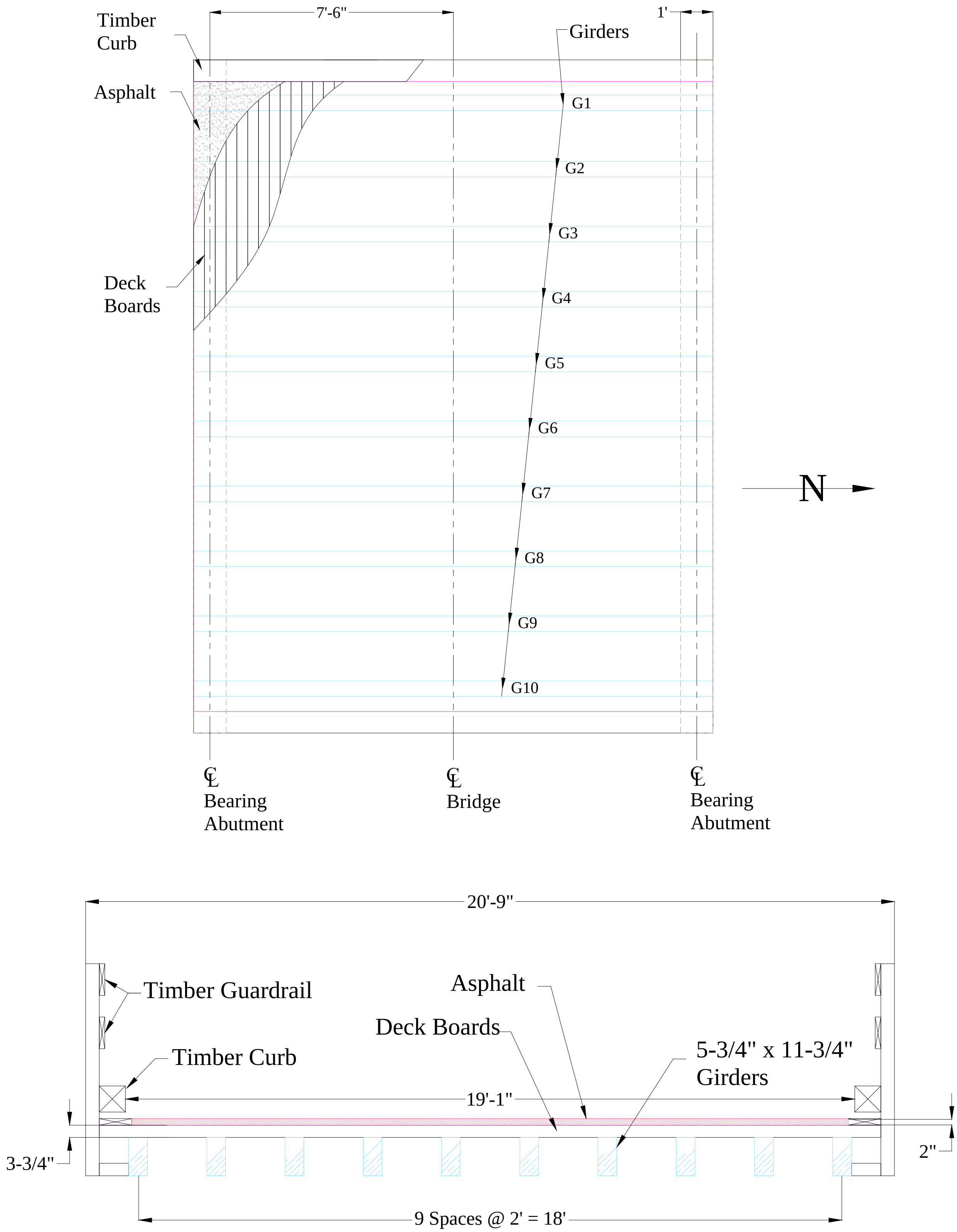

Figure 63. Plan and Profile Layout of Red Horse Creek Bridge

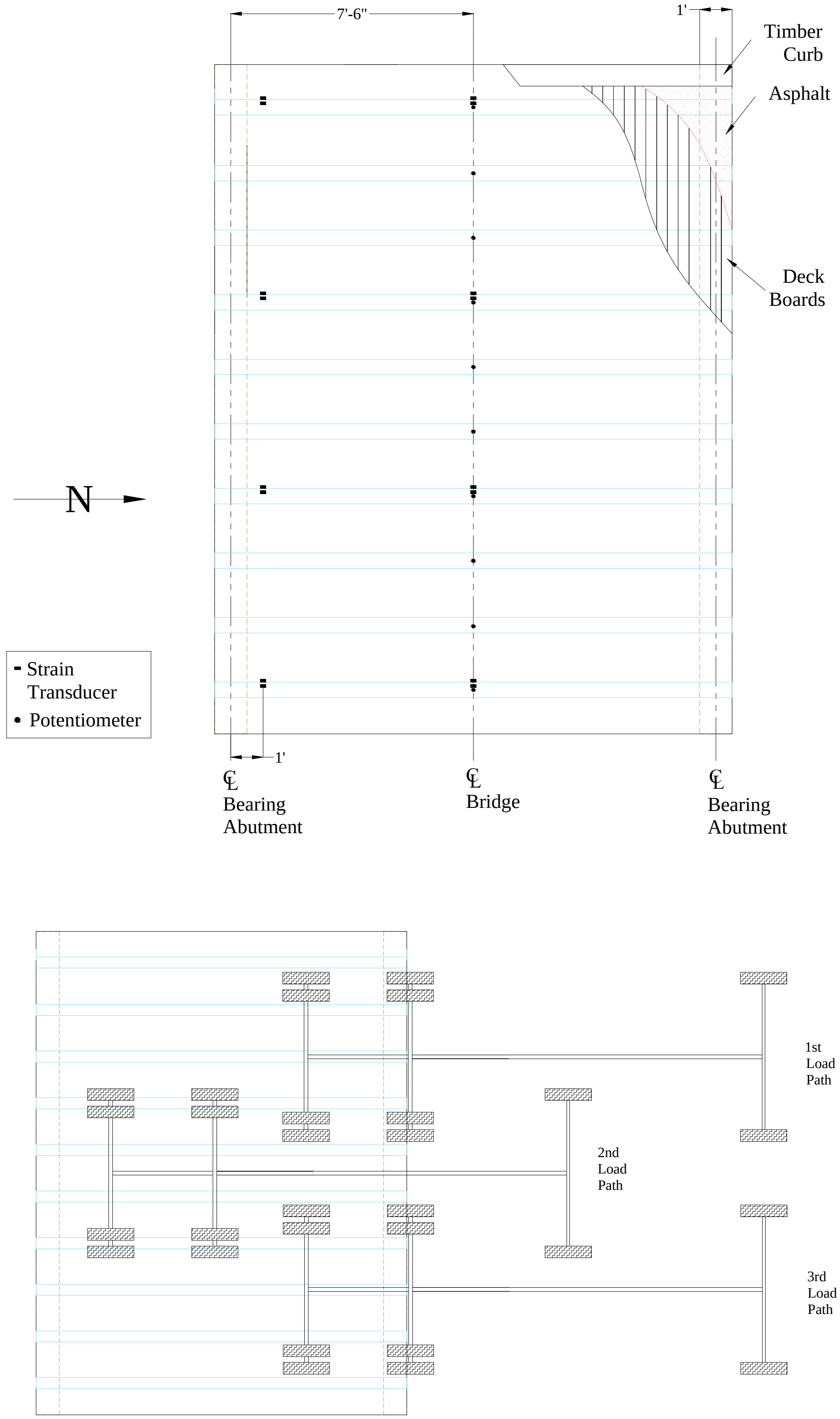

Figure 64. Instrumentation and Load Paths of Red Horse Creek Bridge

Figure 65. Deflection Instrumentation

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 Figure 66). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 66. Strain Transducers

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.

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 8. 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 lbs. The load over the front axle was 16,280 lbs and, assuming that the load over each rear axle was equal, the load was 19,060 lbs over each rear axle. Figure 68 shows the truck used for load testing.

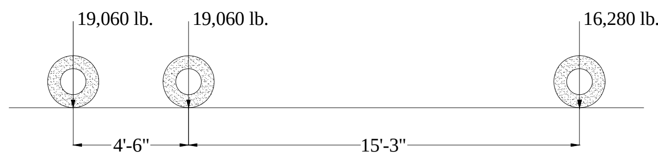

Figure 67. Truck Configuration and Axle Loads

Figure 68. Tandem Axle Load Truck

Three load paths were considered when testing the bridge and each are shown in Figures 10 through 12. 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.

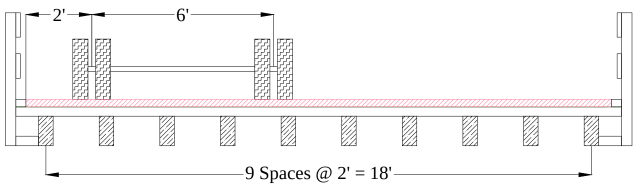

Figure 69. Transverse Truck Position - Load Path 1

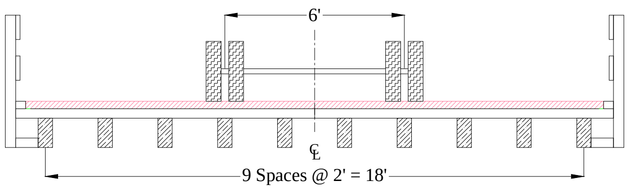

Figure 70. Transverse Truck Position - Load Path 2

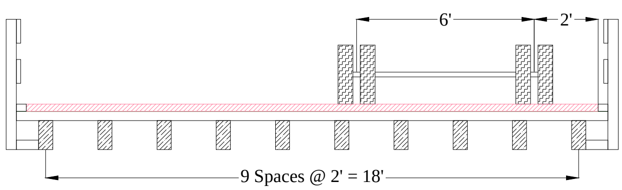

Figure 71. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 13. 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 the bridge. Excessive vegetation can promote pavement deterioration.

Figure 72. Pavement Condition

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

Figure 73). Minor water staining from seepage through the wearing surface was present throughout and white residue was present in a number of locations, though there were no signs of imminent decay.

Figure 73. Deck Board Detachment

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 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 74 shows an example of observed checks. The girder bearing on the sill was sufficient and there is no misalignment.

Figure 74. Observed Checking in Girders

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, 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. 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 16 through 18 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections reflect the difference in load from the front axle to the back axles.

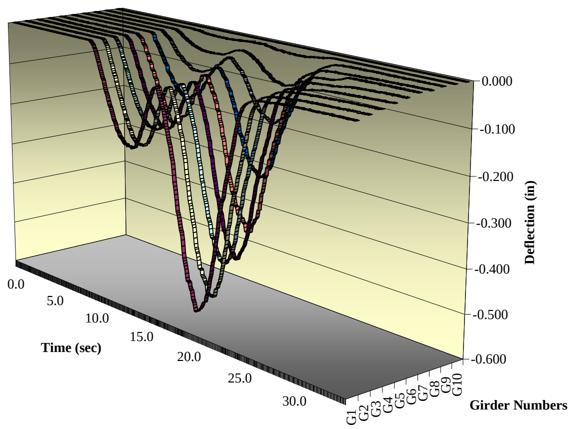

Figure 75. Deflections Load Path 1

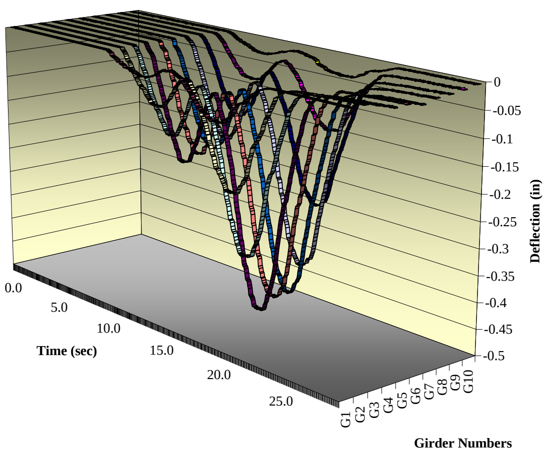

Figure 76. Deflections Load Path 2

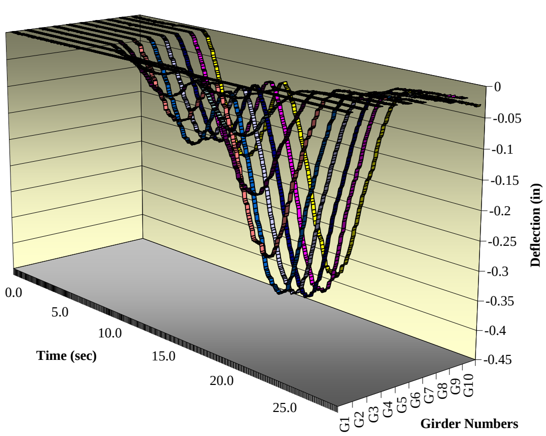

Figure 77. Deflections Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figure 19 through 21. 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.

Table 11. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in)		
Load Path 1	Load Path 2	Load Path 3
0.551	0.453	0.422

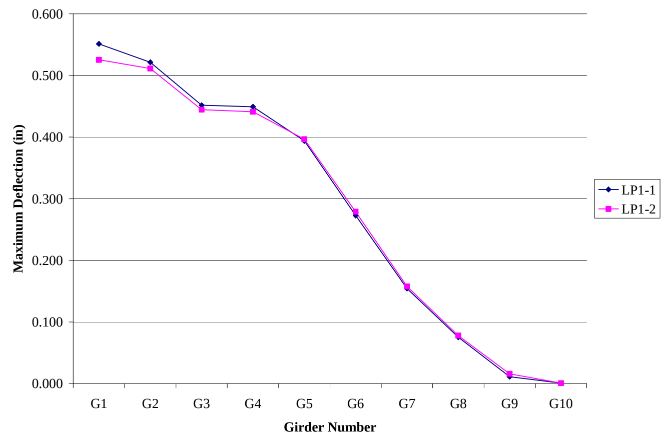

Figure 78. Maximum Deflections for Load Path 1

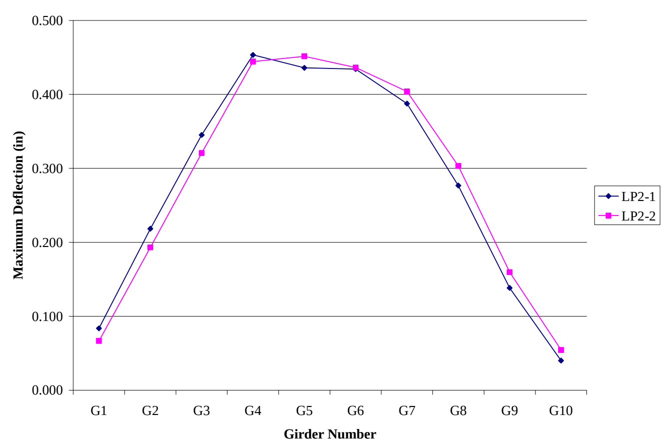

Figure 79. Maximum Deflections for Load Path 2

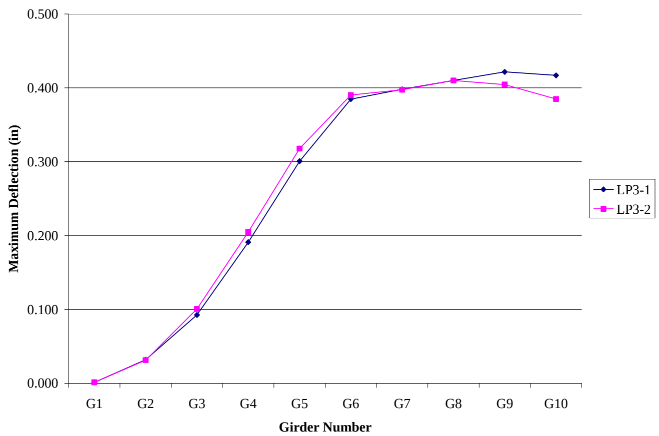

Figure 80. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 12. Live Load Deflection Limit States

Source	n-Value
Timber Bridges	L/360
Highway Bridges	L/425
AASHTO	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 5

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 13. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
238	283	308

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 of these n-values are below the recommended n-values stated in Table 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 6

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the distribution factors for each girder for each load path.

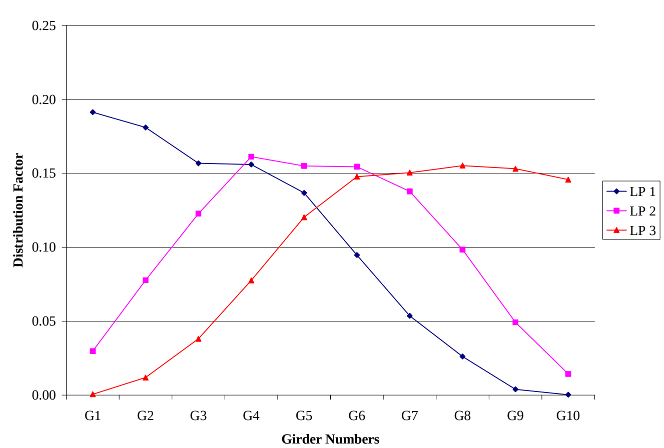

Figure 81. Load Fractions for Each Load Path

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, and S is equal to the transverse spacing between adjacent girders. For this case the exterior lane live load distribution factors equal that of the interior lanes. Shown in Figure 23 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.

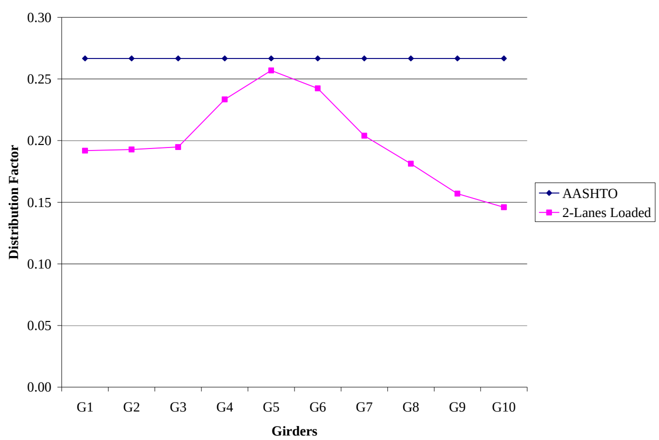

Figure 82. AASHTO Design Live Load Distribution

Differential Deflections

It was shown that 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. Figures 24 through 26 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 4.

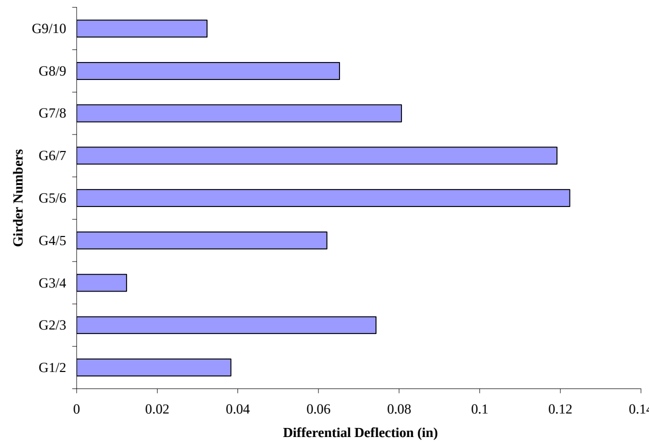

Figure 83. Differential Deflections for Load Path 1

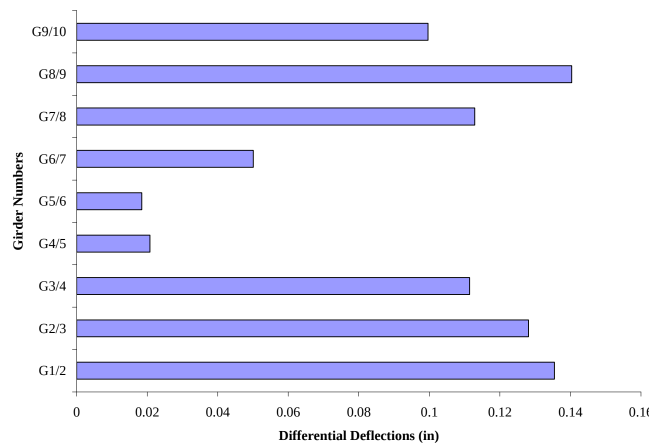

Figure 84. Differential Deflections for Load Path 2

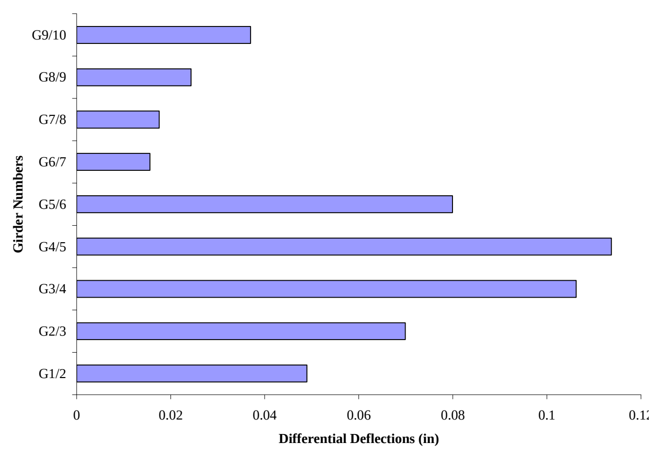

Figure 85. Differential Deflections for Load Path 3

Table 14. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in)		
Load Path 1	Load Path 2	Load Path 3
0.123	0.144	0.114

The maximum differential deflection 0.144 in. occurs in load path 2. This is nearly 32 percent 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.

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 27 through 29 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 through 29 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, i.e., the beams act independently of the deck when subjected to bending.

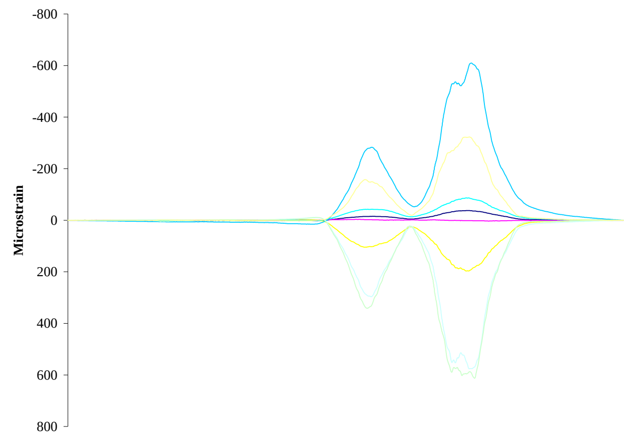

Figure 86. Strain at Midspan for Load Path 1

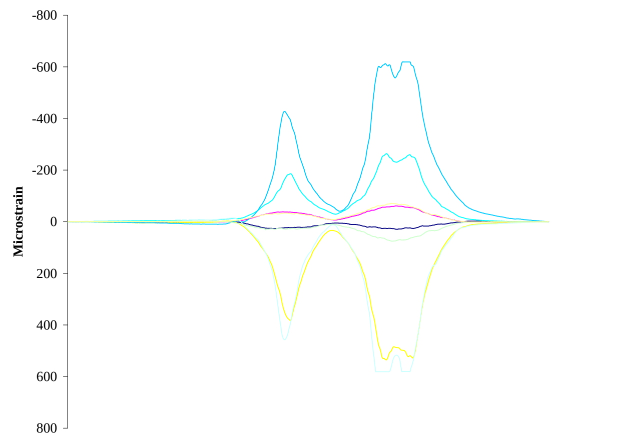

Figure 87. Strain at Midspan for Load Path 2

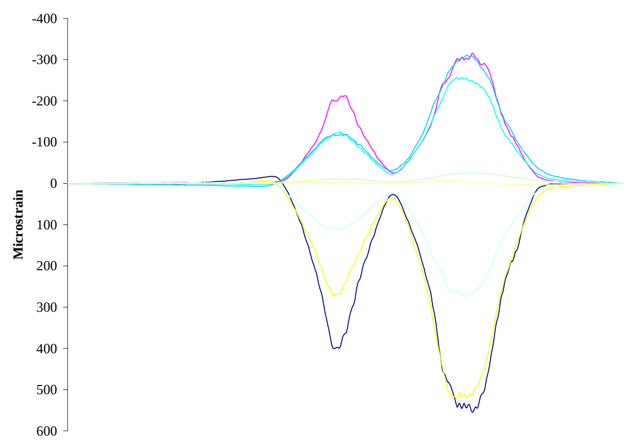

Figure 88. Strain at Midspan for Load Path 3

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 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 percent. The moisture content measurements are summarized in Table 5.

Table 15. Moisture Content Summary

Moisture Content Reading Locations and Values	
Location	%
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 & 2	14.3
Bottom of Deck Between Girders 5 & 6	13.0
Bottom of Deck Between Girders 9 & 10	13.3

Finite Element Analysis

A finite element model was developed (see Figure 89) 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.

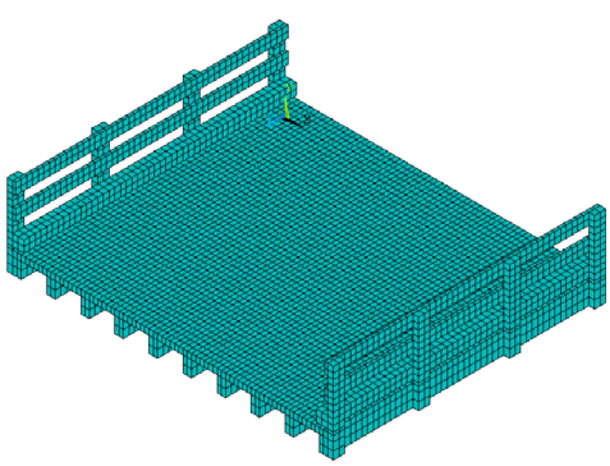

Figure 89. Finite Element Model

Figures 31 through 33 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.

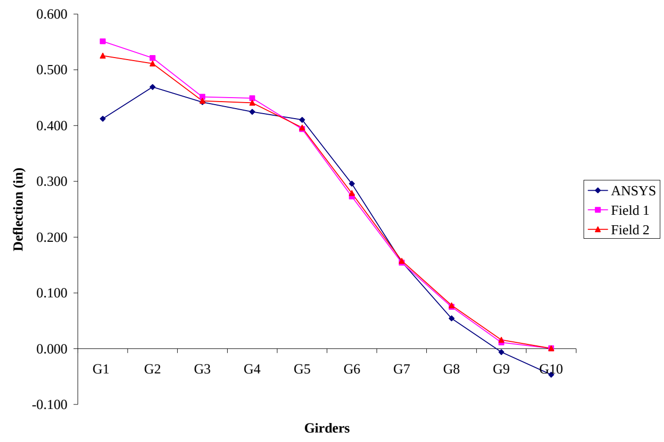

Figure 90. ANSYS Calibration Results Load Path 1

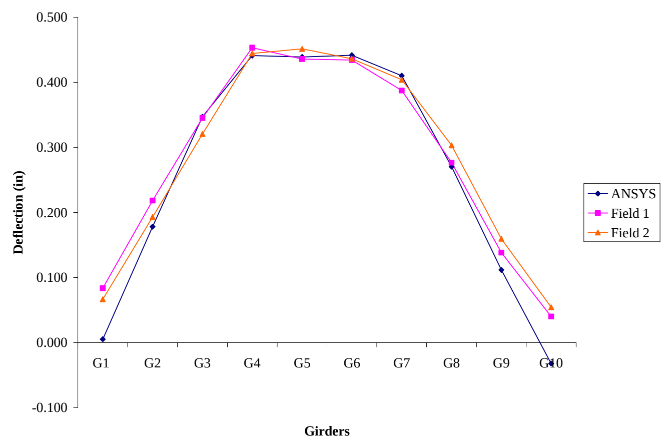

Figure 91. ANSYS Calibration Results Load Path 2

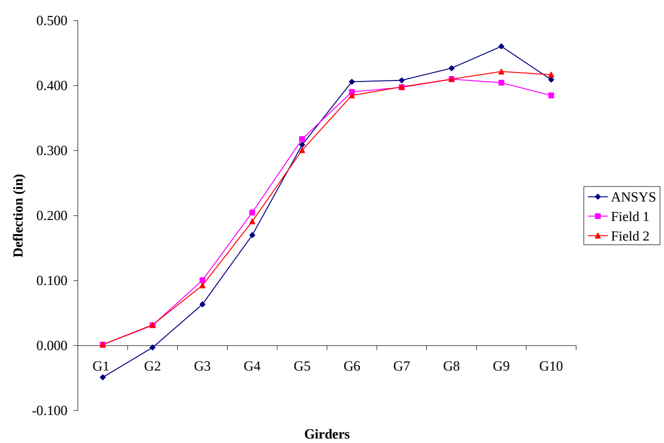

Figure 92. ANSYS Calibration Results Load Path 3

Figure 93 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.

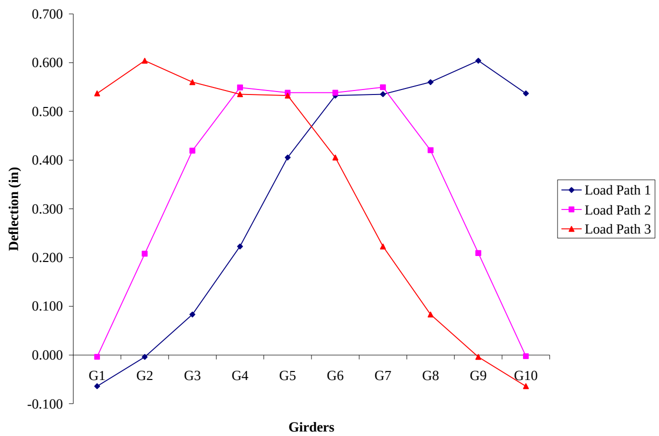

Figure 93. ANSYS Deflection Results for Each Load Path when Subjected to HS20 Tandem Axle Design Vehicle

Figure 35 shows the maximum tensile stresses at midspan due to the AASHTO HS20 tandem axle design vehicle traveled along each load path.

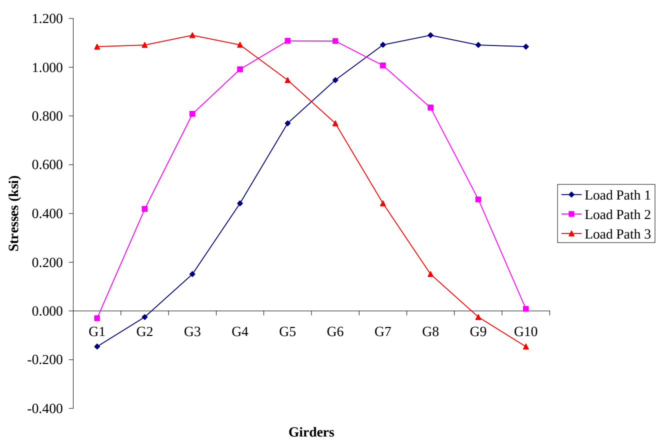

Figure 94. ANSYS Tensile Stress for Each Load Path when Subjected to HS20 Tandem Axle Design Vehicle

Discussion of Results

The following discussion is based on the results presented, including: deflections at midspan, distribution factors, 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, 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 0.022 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, i.e., 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 and limits their use for primary analysis. Nevertheless, Figures 27 through 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 619 and 581 microstrain, respectively. These strains equate to maximum stresses of 712 and 667 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 934 and 875 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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 1131 and 1108 psi, 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; 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 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 1500 psi 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.

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APPENDIX E

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 560385

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 560385

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 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.

Acknowledgements

We would like to 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.

Brett Rhindhart, North Carolina Department of Transportation employee who assisted in the static load testing

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Madison County Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 560385, hereinafter referred to as the Madison County Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 95. Madison County Bridge in North Carolina

The Madison County Bridge was built in 1962 and is located in Madison County in western North Carolina 0.1miles 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 Figure 96). 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 Figure 97).

Figure 96. Madison County Bridge

Figure 97. Madison County Bridge Substructure

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 5-3/4 in. x 11-1/2 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 4 in. x 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 asphalt wearing surface. Figure 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. A typical setup of deflection instrumentation is shown in Figure 6.

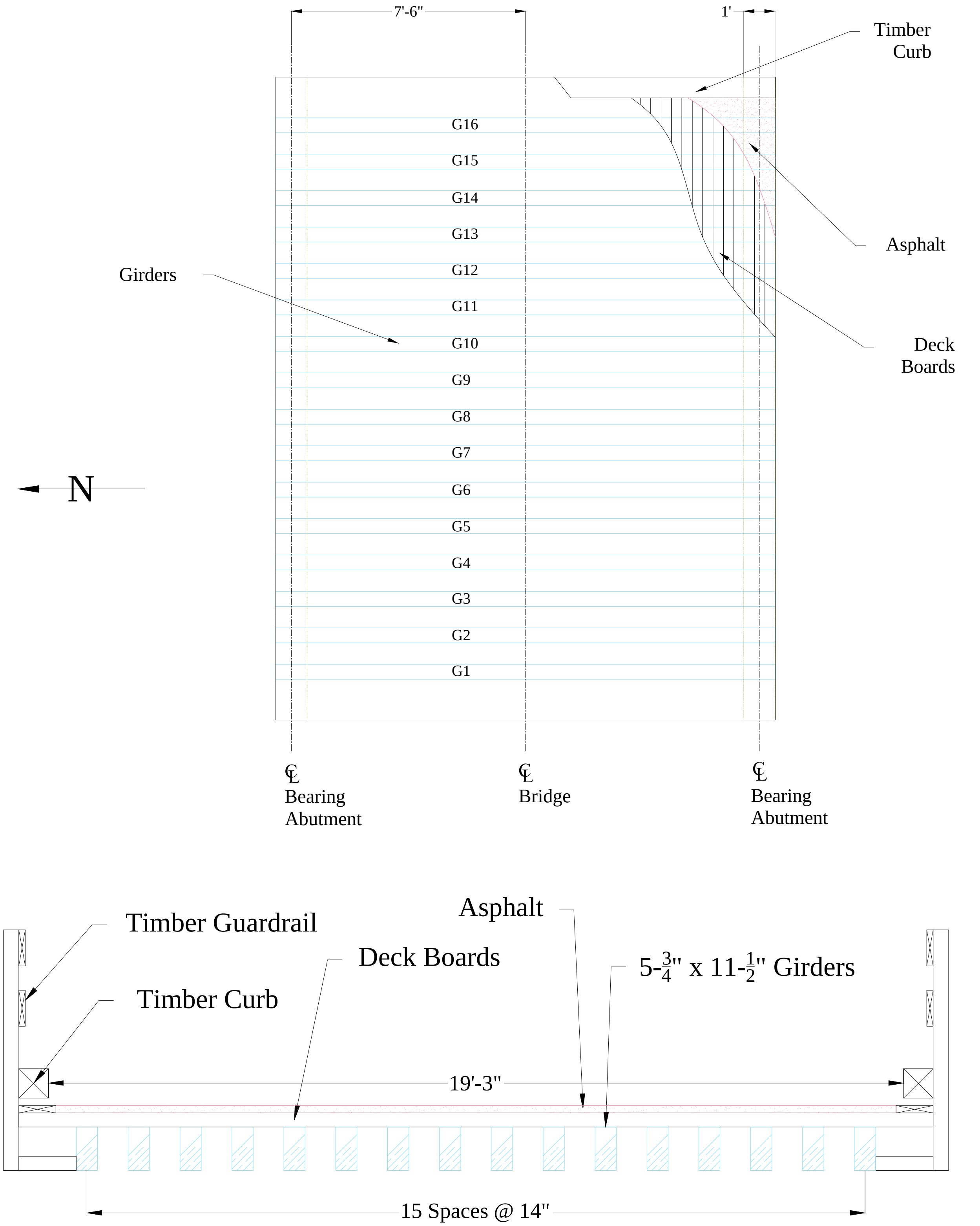

Figure 98. Plan and Profile Layout of Madison County Bridge

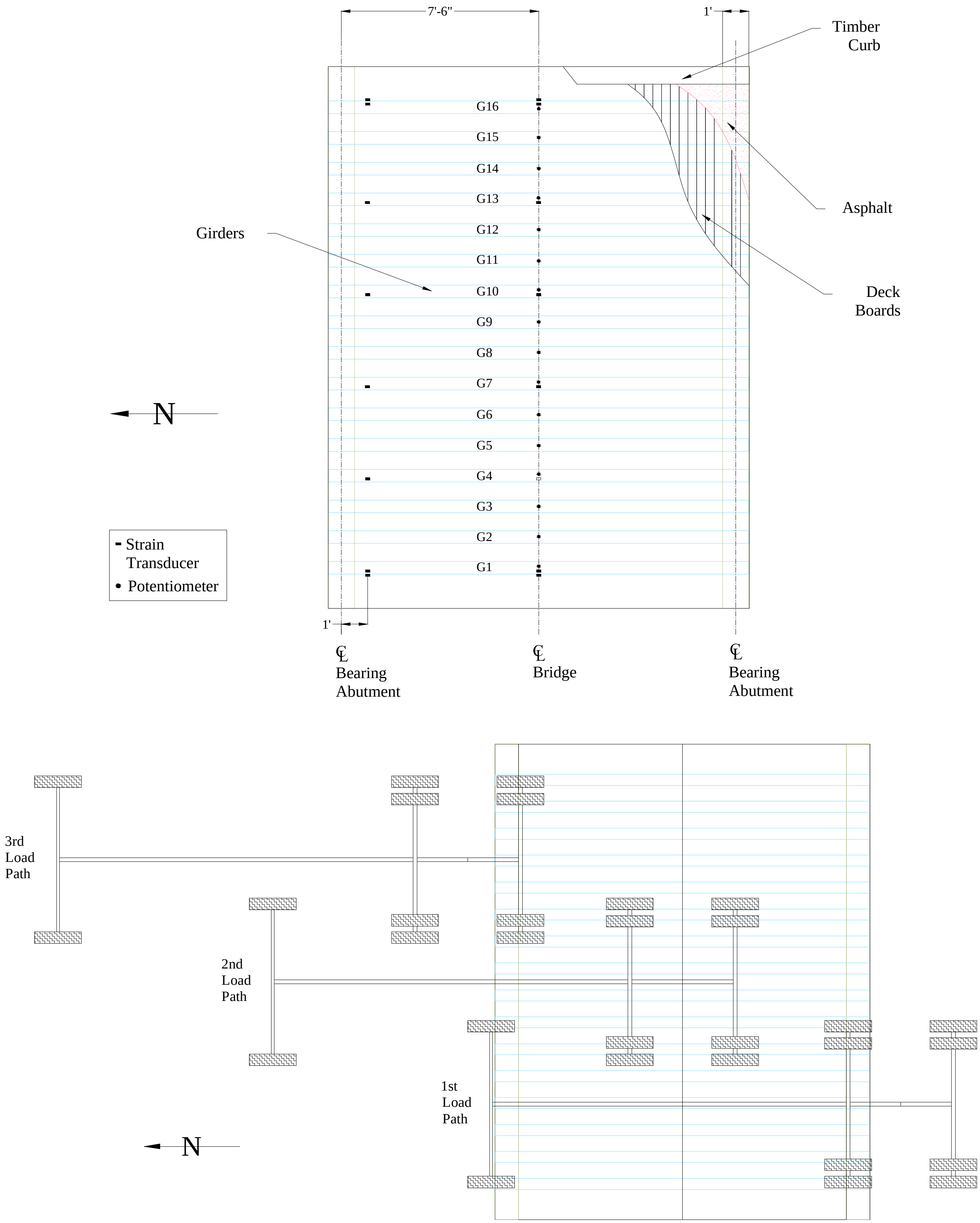

Figure 99. Instrumentation and Load Paths of Madison County Bridge

Figure 100. Typical Deflection Instrumentation

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 Figure 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 7. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 101. Outside Girder Strain Transducer Setup

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.

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 8. 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 lbs. The load over the front axle was 16,280 lbs and, assuming that the load over each rear axle was equal, the load was 19,060 lbs over each rear axle. Figure 103 shows the truck used for the load testing.

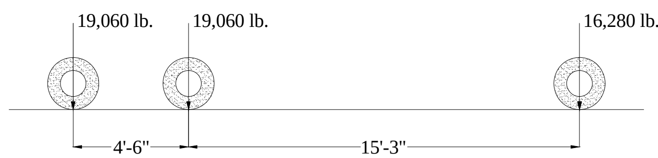

Figure 102. Truck Configuration and Axle Loads

Figure 103. Tandem Axle Load Truck

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 10 through 12.

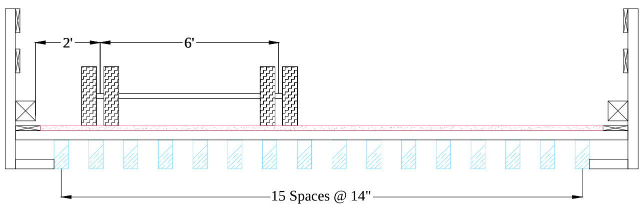

Figure 104. Transverse Truck Position - Load Path 1

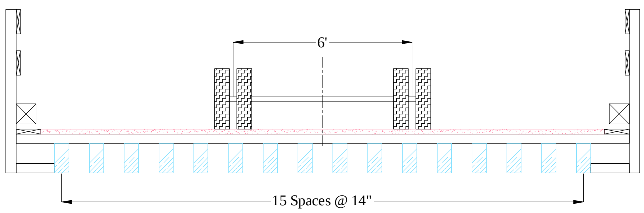

Figure 105. Transverse Truck Position - Load Path 2

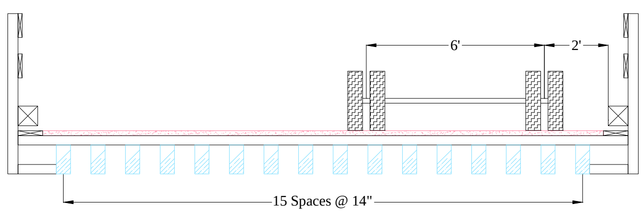

Figure 106. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 ensuing.

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 107 shows the transverse cracking.

Figure 107. Presence of Transverse Cracks

At the transition between the roadway and the bridge and at approximately 5 ft before the bridge, large cracks have formed (see Figure 108). An uneven transition could subject the bridge to unnecessary effects from dynamic loads even though slow vehicle speeds on this roadway make this unlikely.

Figure 108. Transverse Cracks at Bridge Approach

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

Figure 109. Deck Board Detachment

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 though 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 110. The girder bearing on the sill was sufficient and there is no misalignment.

Figure 110. Checking at Girder Centerline

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.

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.

Time-History Deflections

Figures 17 through 19 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

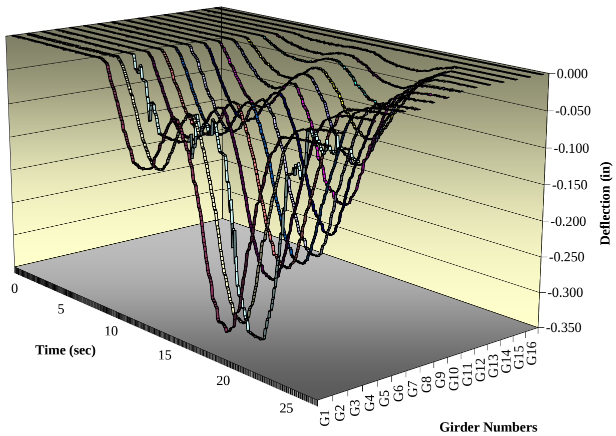

Figure 111. Deflections for Load Path 1

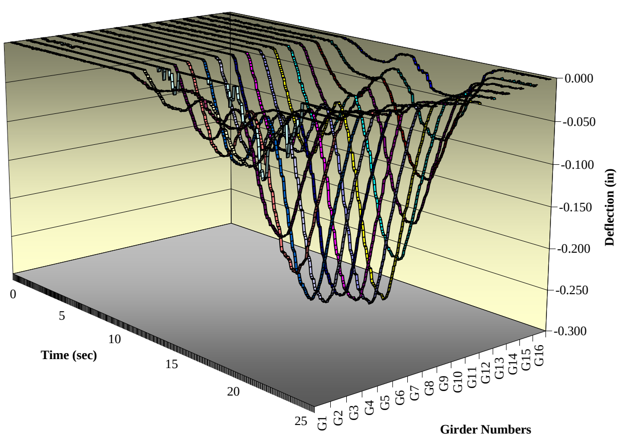

Figure 112. Deflections for Load Path 2

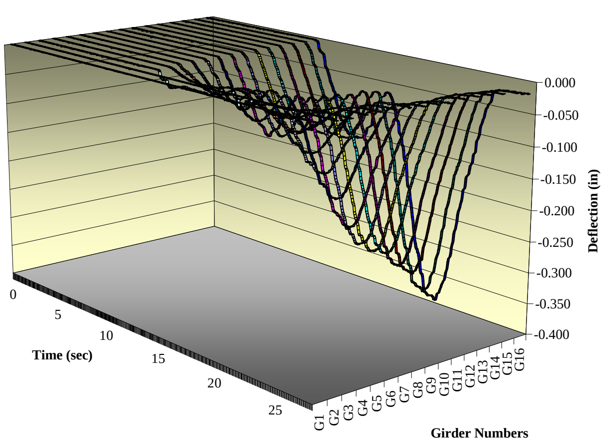

Figure 113. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 20 through 22. 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.

Table 16. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.338	0.272	0.389

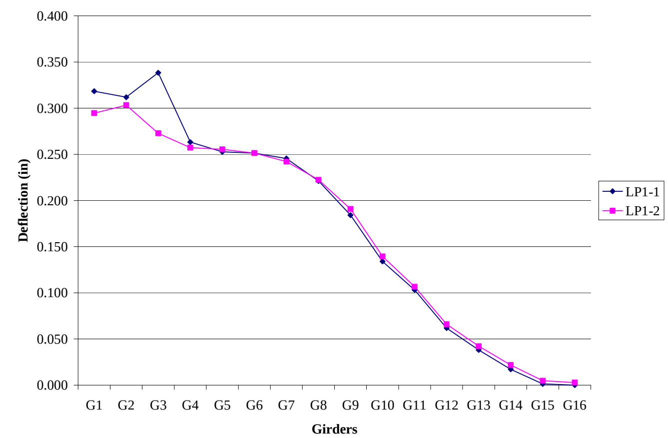

Figure 114. Maximum Deflections for Load Path 1

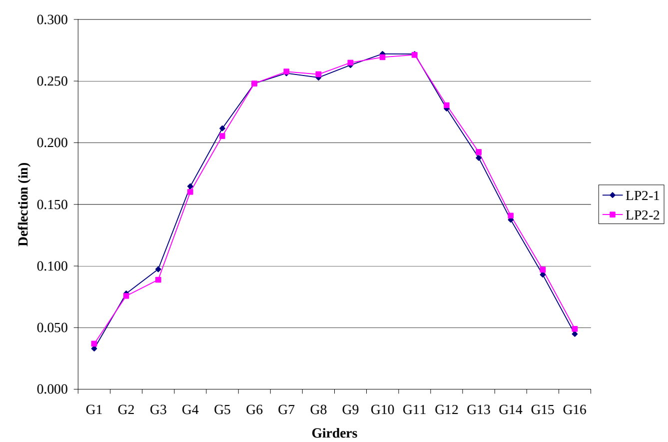

Figure 115. Maximum Deflections for Load Path 2

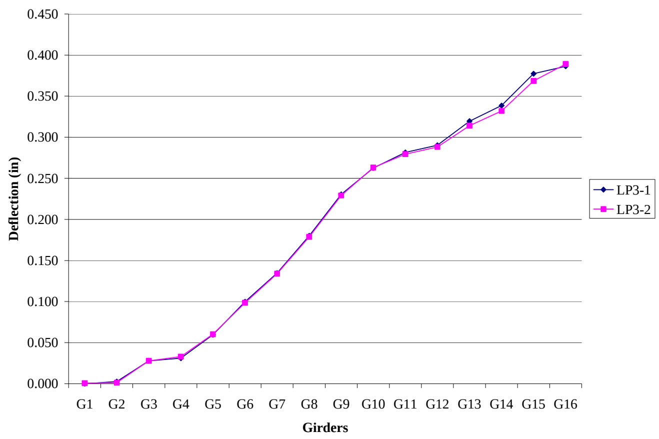

Figure 116. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 17. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 7

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 18. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
379	471	322

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 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 8

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

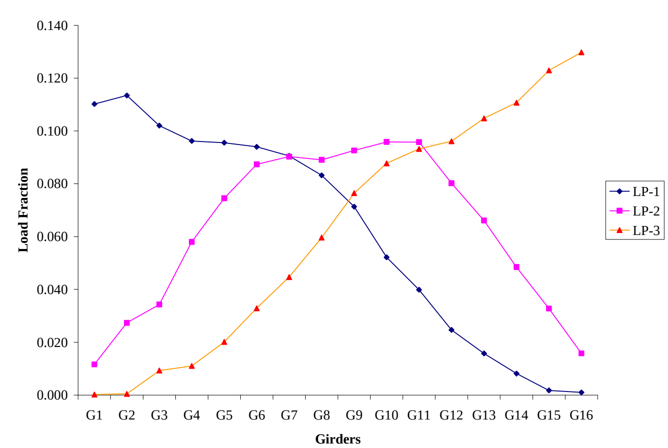

Figure 117. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

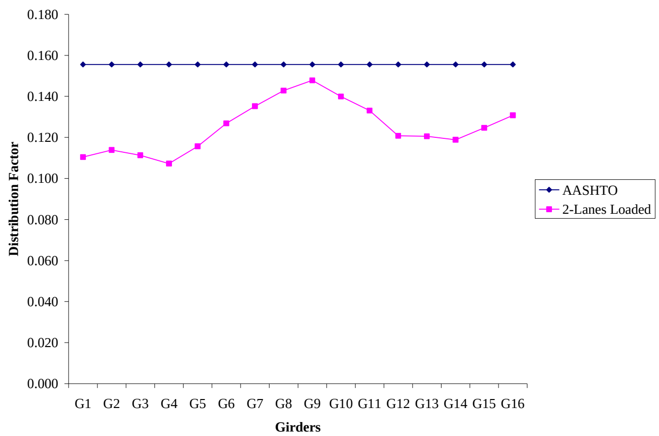

Figure 118. AASHTO Design Live Load Distribution

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. Figures 25 through 27 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 4.

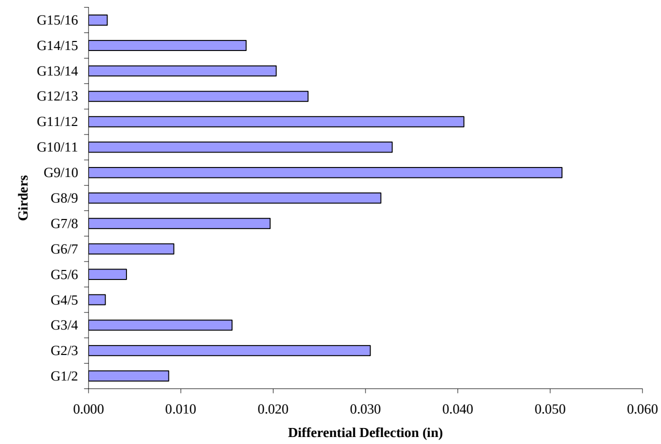

Figure 119. Differential Deflections for Load Path 1

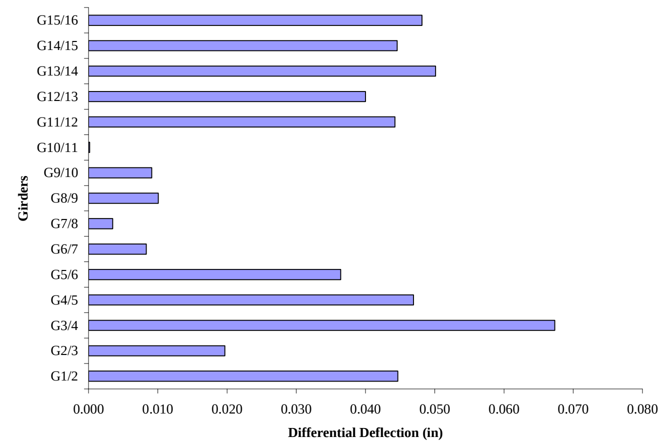

Figure 120. Differential Deflections for Load Path 2

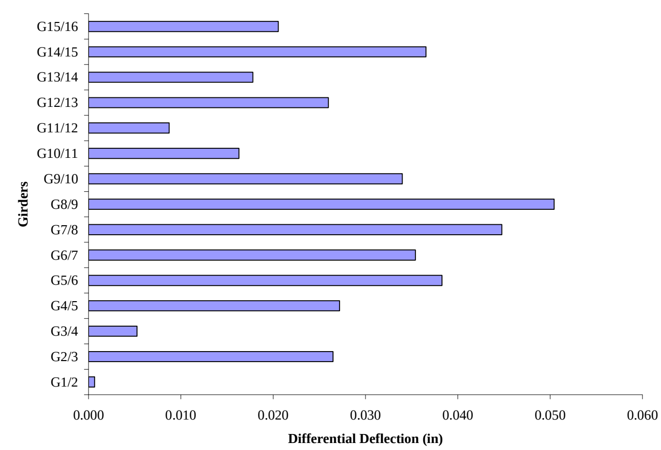

Figure 121. Differential Deflections for Load Path 3

Table 19. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.051	0.067	0.050

The maximum differential deflection of 0.067 in. occurs in load path 2. This is nearly 25 percent of the maximum deflection 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 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 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 27 through 29 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 27 through 29 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, i.e., the girders act independently of the deck when subjected to bending.

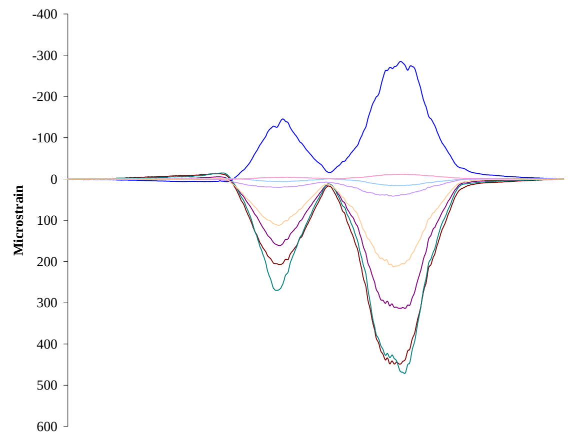

Figure 122. Strain at Midspan for Load Path 1

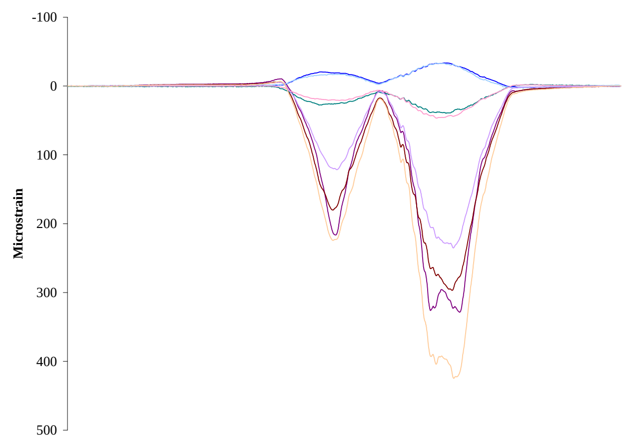

Figure 123. Strain at Midspan for Load Path 2

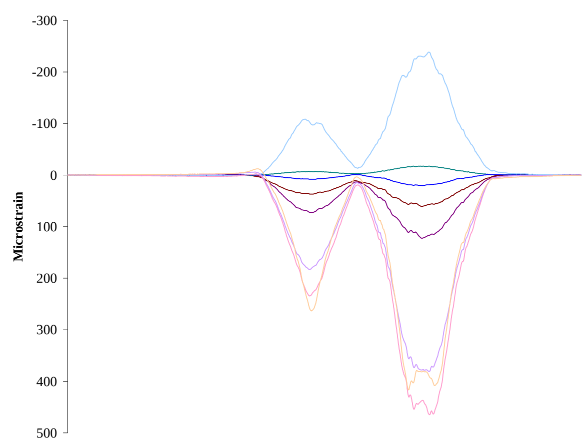

Figure 124. Strain at Midspan for Load Path 3

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, 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 percent. The moisture content measurements are summarized in Table 5.

Table 20. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	15.1
Bottom of Deck Between Girders 8 & 9	23.1
Bottom of Deck Between Girders 15 & 16	17.1

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, 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 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, i.e., 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 28 through 30 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 430 and 711 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 1500 psi 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.

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APPENDIX F

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 430351

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 430351

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 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.

Acknowledgements

We would like to 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.

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.

Rick Arrington, North Carolina Department of Transportation Employee who assisted during the load testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Dix Creek Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 430351, hereinafter referred to as the Dix Creek Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 125. Dix Creek Bridge in North Carolina

The Dix Creek Bridge was built in 1957 and is located in Haywood County in western North Carolina 100 ft west of 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 Figure 126).

Figure 126. Dix Creek Bridge Substructure

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 3.

Figure 127. Dix Creek Parapet

Girders measure 17 ft-9 in. from end to end and have a clear span of 16 ft-1 in. A total of 9 girders, spaced 24-3/4 in. center-to-center, measuring 5-3/4 in. x 11-1/2 in. in cross-section are present and are seated and toe-nailed to the 10-in. x 10-in. timber sills with spikes. The deck consists of individual 4 in. x 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 asphalt wearing surface. Figure 4 illustrates the layout of the bridge.

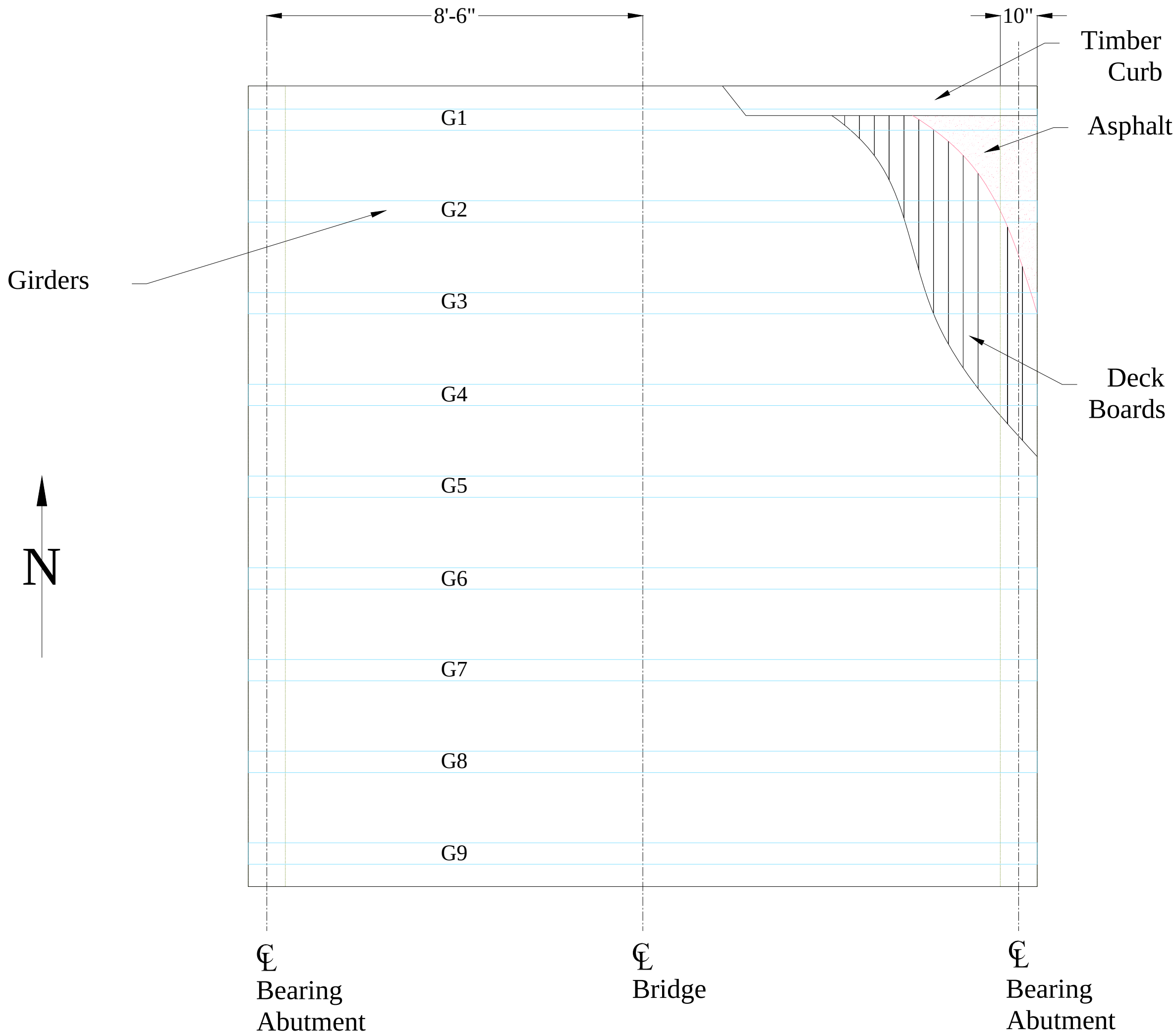
Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 9 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.



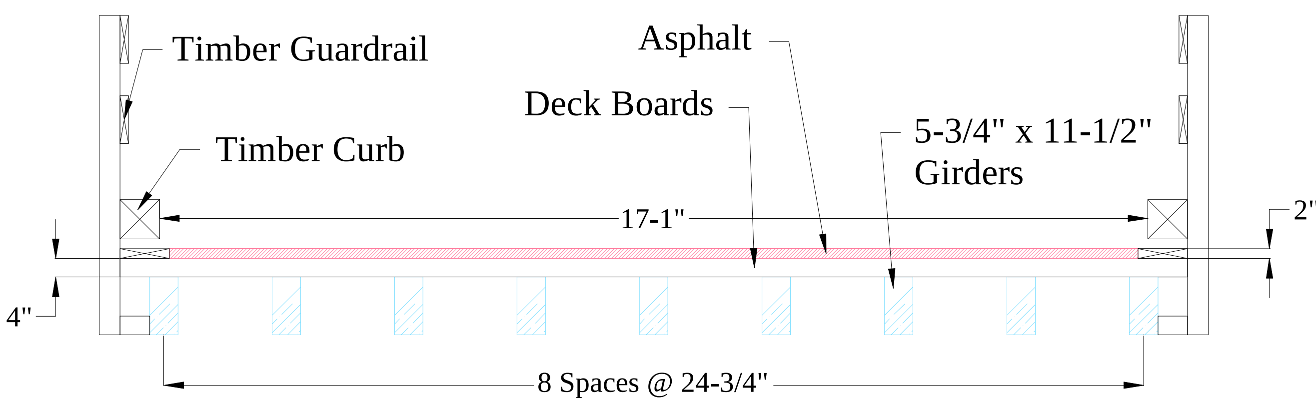
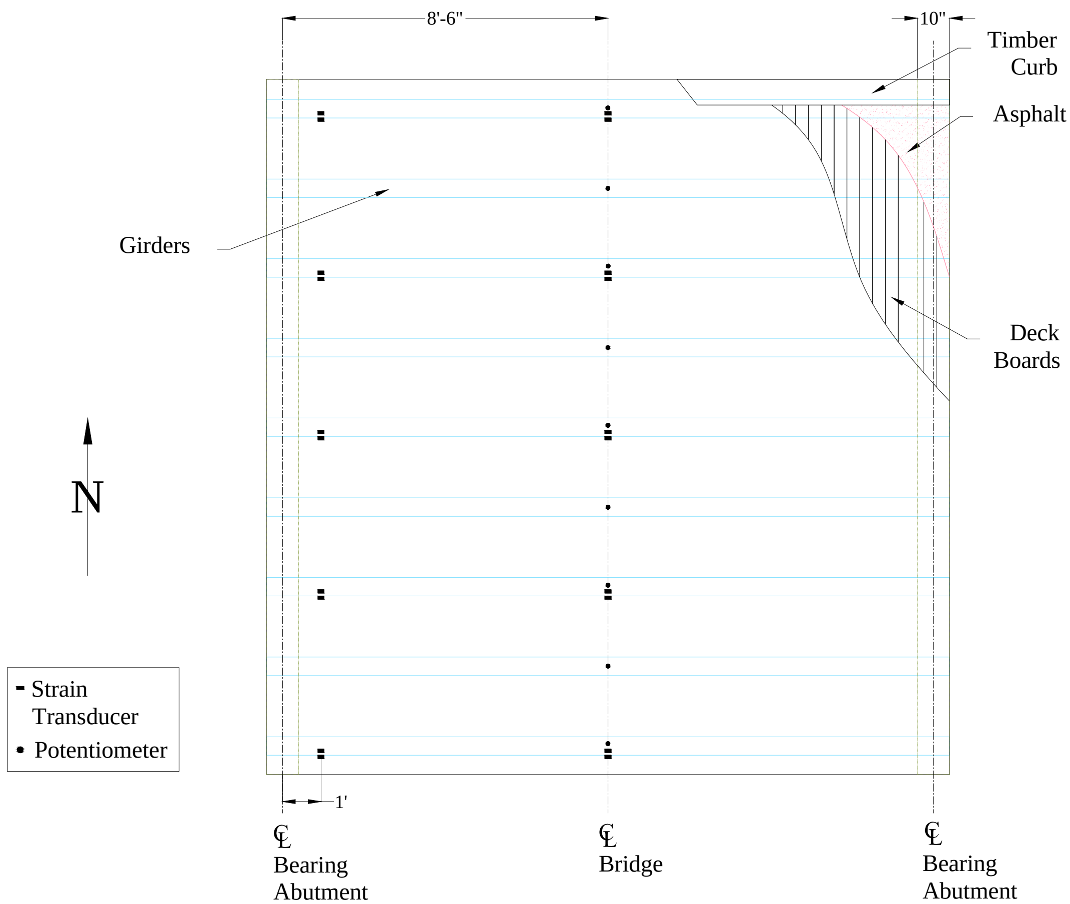


Figure 128. Plan and Profile Layout of Dix Creek Bridge



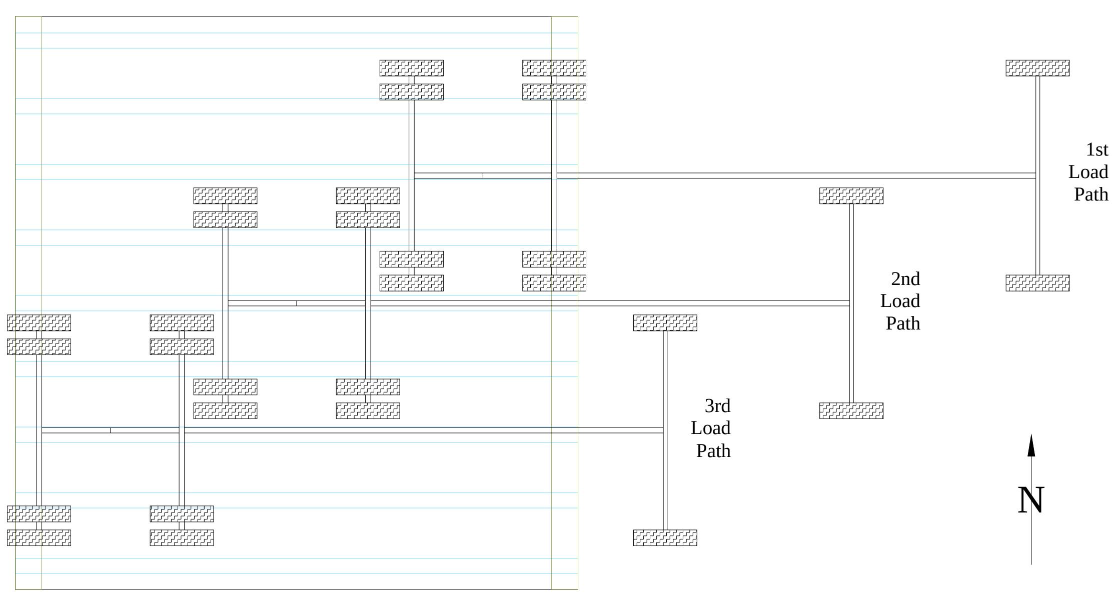


Figure 129. Instrumentation and Load Paths of Dix Creek Bridge

Figure 130. Deflection Instrumentation

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 Figure 5). 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 Figure 7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 131. Strain Transducers

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 measurments were taken at several locations throughout the girder and deck elements.

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 8. 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 lbs and it is typical that 70 percent 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 lbs and 15,996 lbs, respectively. Figure 133 shows the truck used for load testing.

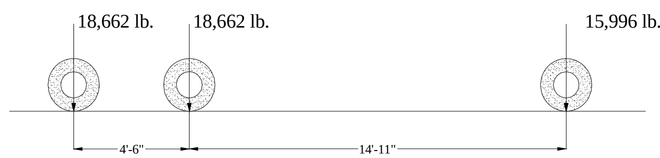


Figure 132. Truck Configuration and Axle Loads

Figure 133. Tandem Axle Load Truck

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 10 through 12 illustrate each load path.

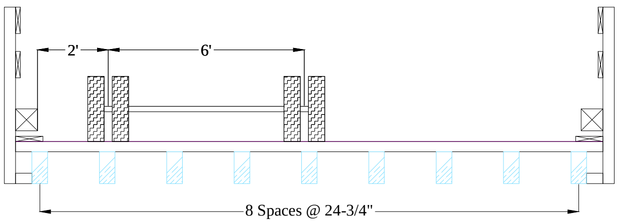


Figure 134. Transverse Truck Position - Load Path 1

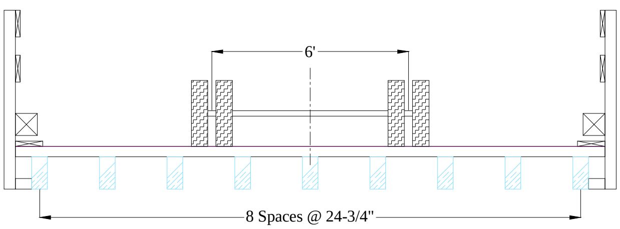


Figure 135. Transverse Truck Position - Load Path 2

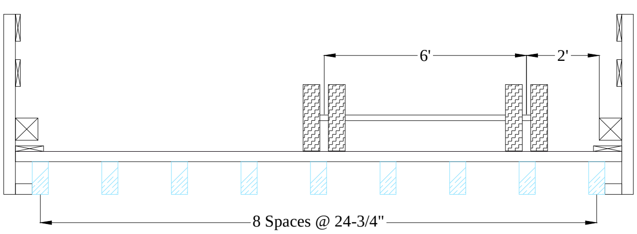


Figure 136. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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 some 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.

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 seems and are shown in Figure 13. 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 even though 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 137. Transverse Cracks in the Wearing Surface

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.

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 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 14

Figure 138. Girder End Conditions

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 139 shows a typical case of checking found throughout the bridge.

Figure 139. Checking at Girder Centerline

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, 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.

Figure 140. Parapet Condition

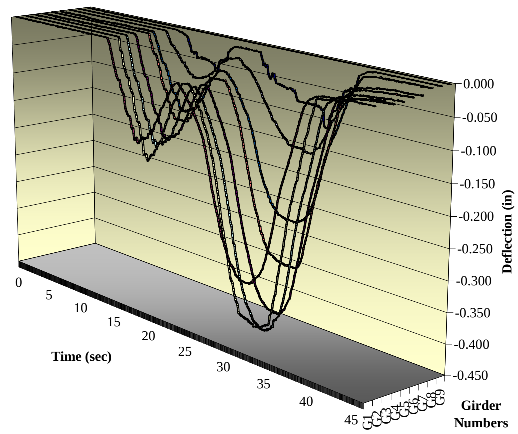


Figure 141. Deflections Load Path 1

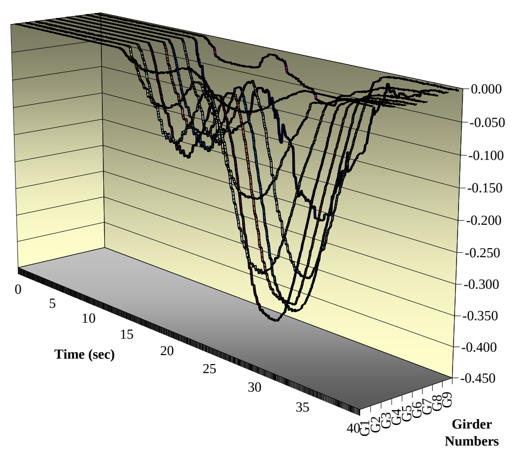


Figure 142. Deflections Load Path 2

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 16 through 18 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

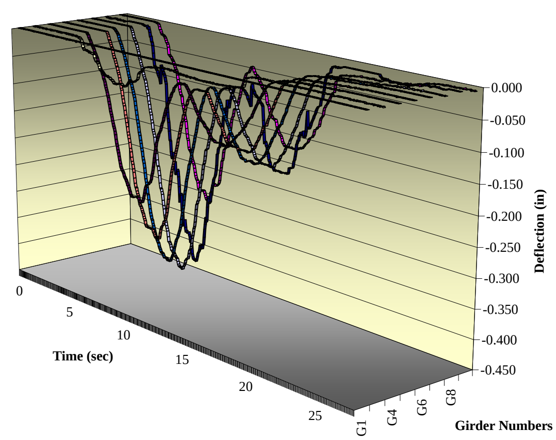


Figure 143. Deflections Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 19 through 21. 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.

Table 21. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.446	0.406	0.456

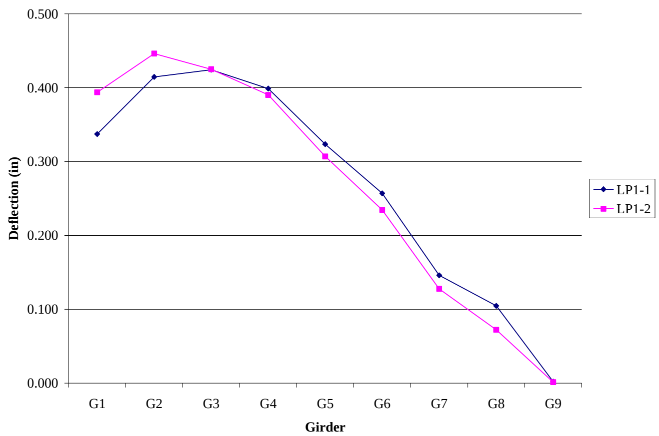


Figure 144. Maximum Deflections for Load Path 1

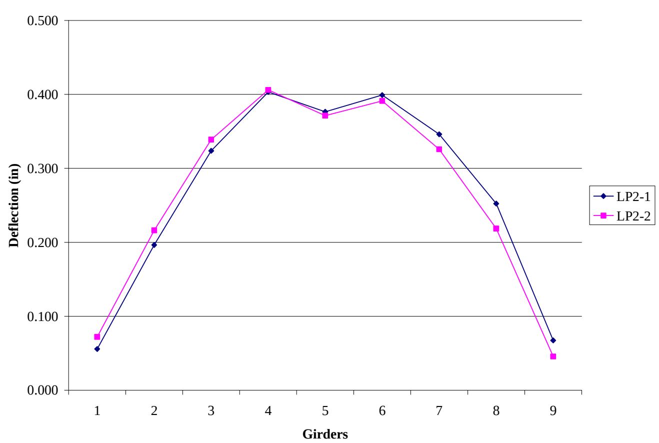


Figure 145. Maximum Deflections for Load Path 2

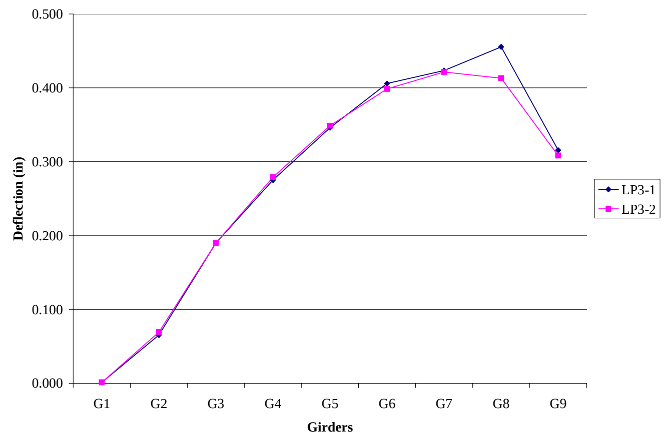


Figure 146. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 22. Live Load Deflection Limit States

Source	**n-Value**
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 9

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 23. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
323	355	316

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 of the n-values are below the recommended n-values stated in Table 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 10

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

- LF_i = load fraction of the i^{th} girder
- Δi = deflection of the i^{th} girder
- $\Sigma\Delta_i$ = sum of all girder deflections
- n = number of girders

Figure 22 shows the load fractions for each girder for each load path.

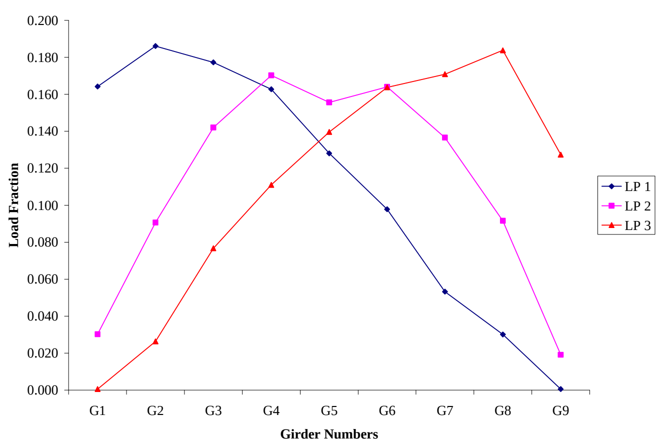


Figure 147. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

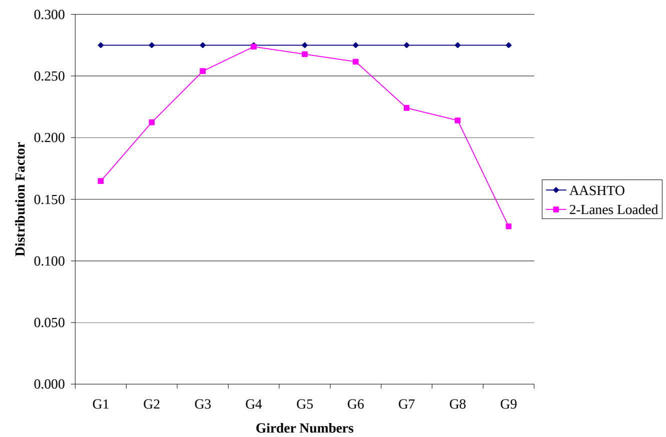


Figure 148. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

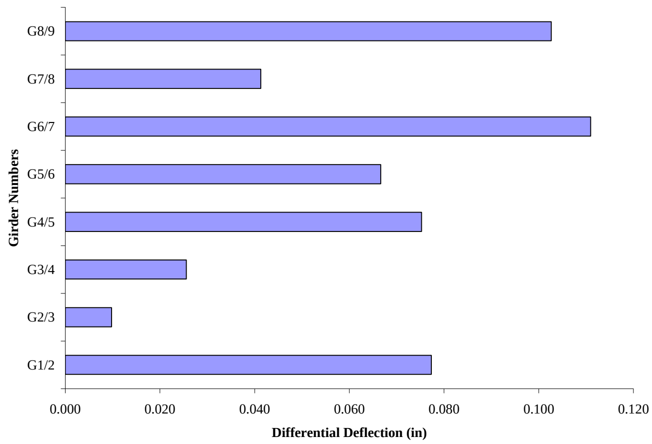


Figure 149. Differential Deflections for Load Path 1

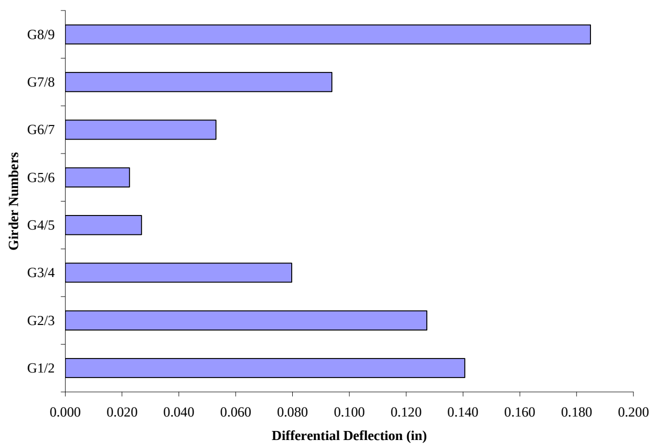


Figure 150. Differential Deflections for Load Path 2

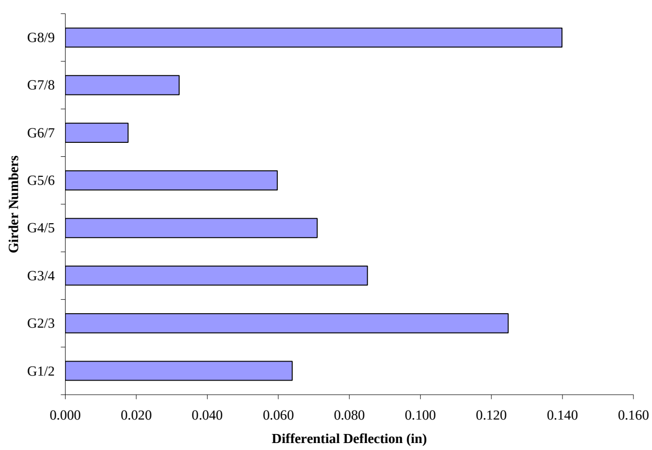


Figure 151. Differential Deflections for Load Path 3

Table 24. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.111	0.185	0.140

The maximum differential deflection of 0.185 in. occurs in load path 2. This is nearly 46 percent 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 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 28 through 30 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 28 through 30 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, i.e., the girders act independently of the deck when subjected to bending.

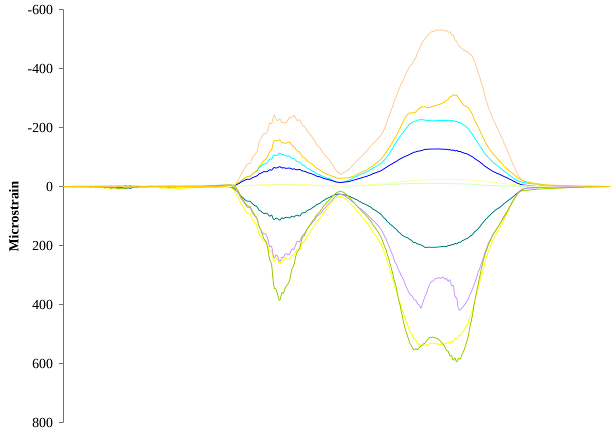


Figure 152. Strain at Midspan for Load Path 1

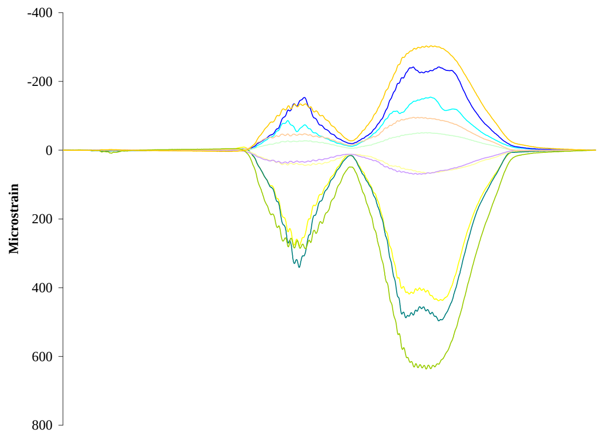


Figure 153. Strain at Midspan for Load Path 2

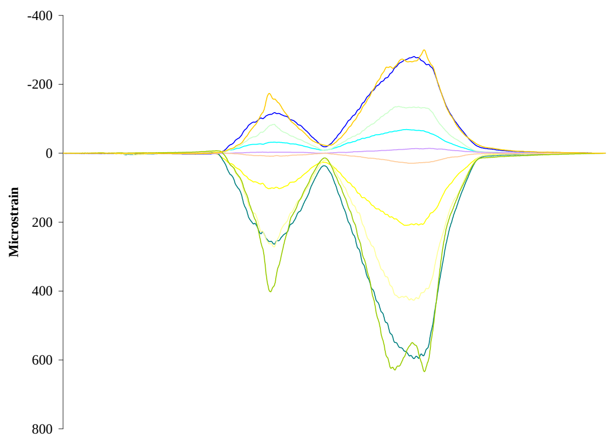


Figure 154. Strain at Midspan for Load Path 3

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 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 30+ percent. Overall, significant moisture content differences were not found throughout the bridge except for between girders 8 and 9. The moisture content measurements are summarized in Table 5.

Table 25. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	18.2
Bottom of Deck Between Girders 5 & 6	26.0
Bottom of Deck Between Girders 8 & 9	30 +

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, i.e., 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 28 though 30 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 530 and 636 microstrain, respectively. These strains equate to maximum stresses of 610 and 731 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 817 and 980 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 1500 psi 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.

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APPENDIX G

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 430306

United States
Department of
Agriculture

Forest Service

Forest Products
Laboratory

Iowa State
University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 430306

Terry Wipf
Brent Phares
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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 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.

Acknowledgements

We would like to 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.

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.

Rick Arrington, North Carolina Department of Transportation employee who assisted during the load testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Bald Creek Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 460306, hereinafter referred to as the Bald Creek Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 155. Bald Creek Bridge in North Carolina

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 degree 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 Figure 156).

Figure 156. Bald Creek Bridge Substructure

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 157.

Figure 157. Bald Creek Bridge Parapet

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 in. x 11-1/2 in. in cross-section are present and are seated and toe-nailed to the 10-in. x 10-in. timber sills with spikes. The deck consists of individual 4 in. x 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 asphalt wearing surface. Figure 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.

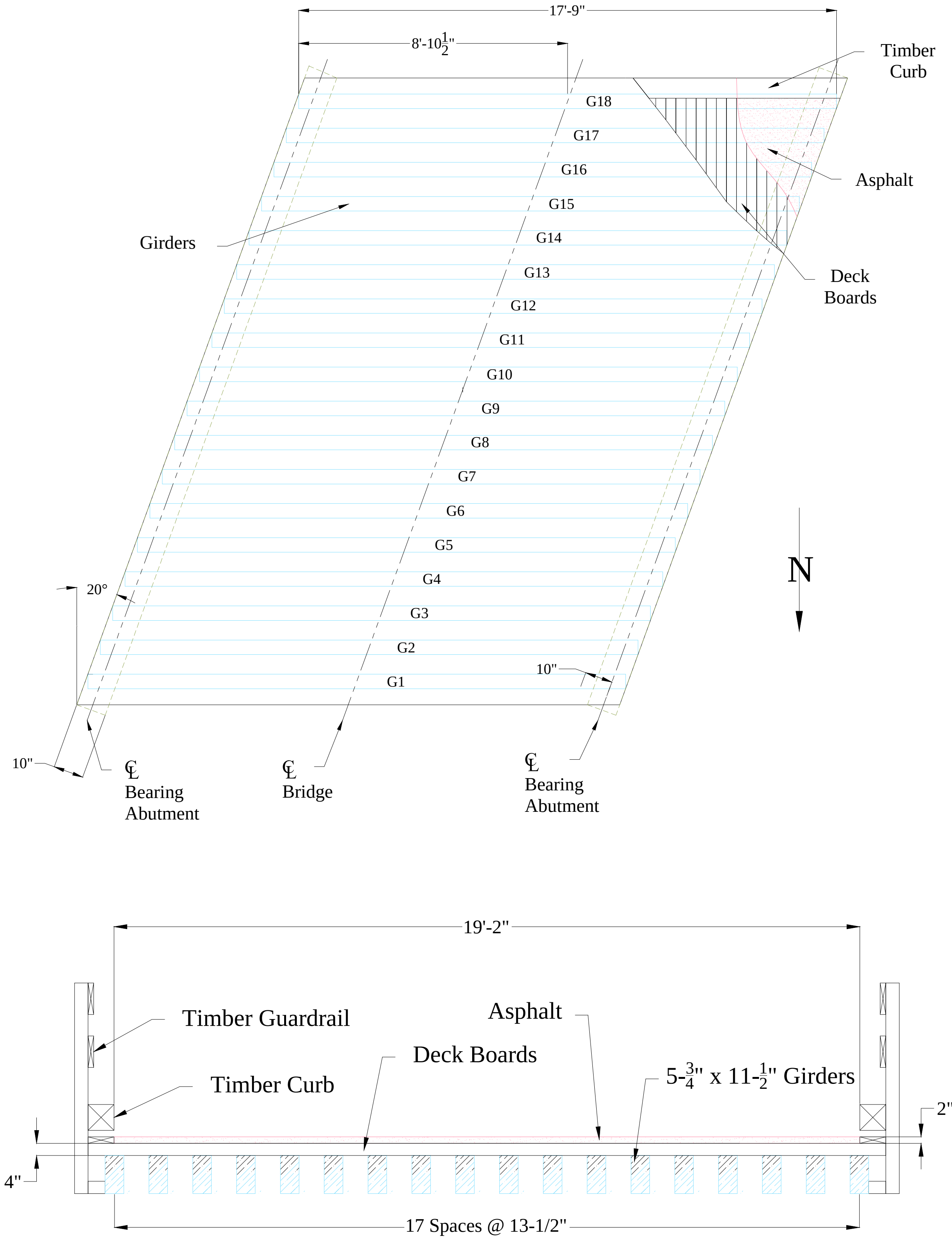

Figure 158. Plan and Profile Layout of Bald Creek Bridge

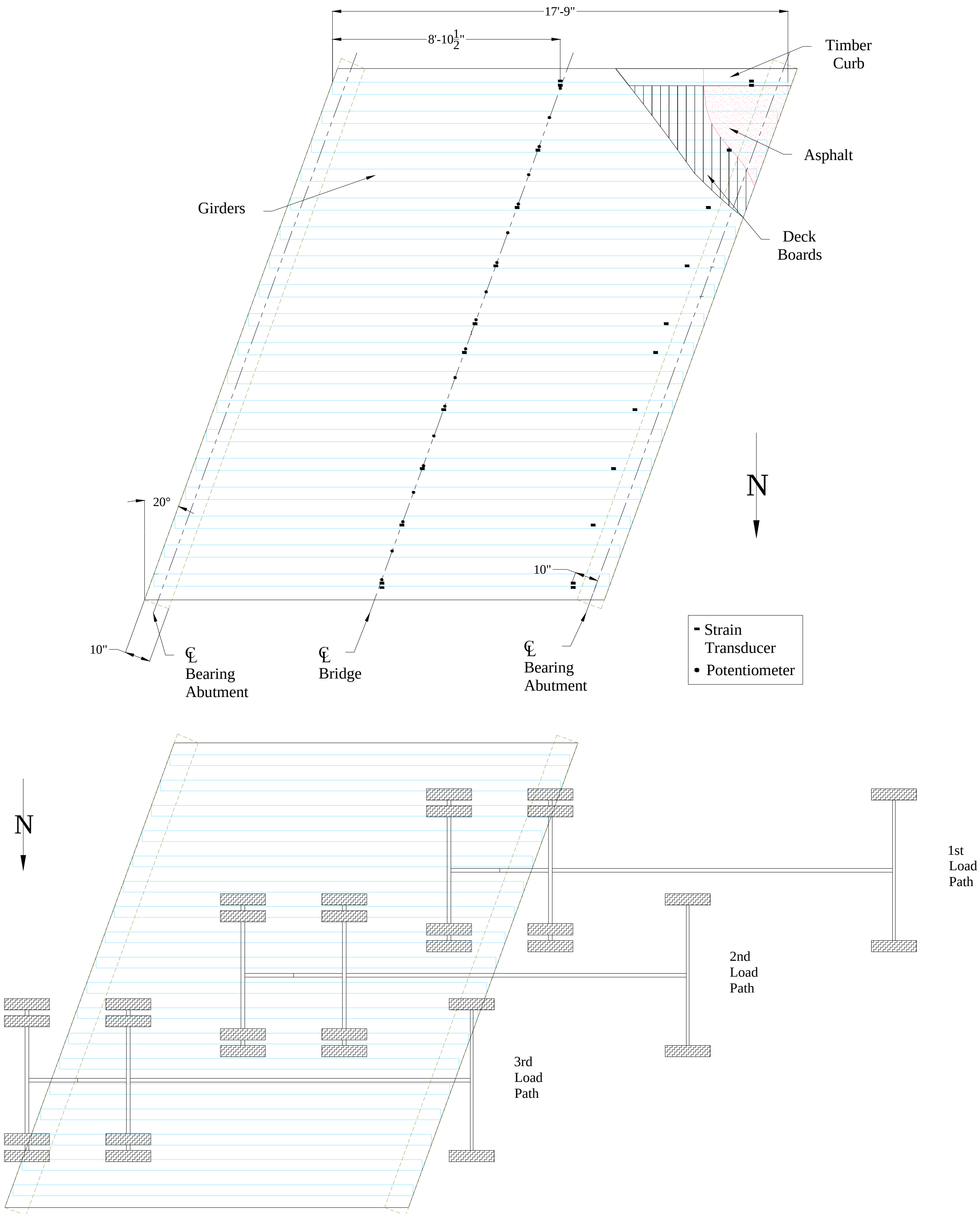

Figure 159. Instrumentation and Load Paths of Bald Creek Bridge

Figure 160. Deflection Instrumentation

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 Figure 5). 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 161 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.

Figure 161. Outside Girder Strain Transducer Setup

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 measurments were taken at several locations throughout the girder and deck elements.

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 8. 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 lbs and typically, 70 percent 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 lbs and 15,996 lbs, respectively. Figure 163 shows the truck used for the load testing.

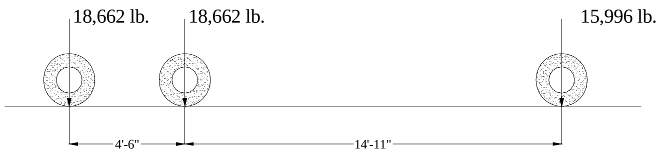

Figure 162. Truck Configuration and Axle Loads

Figure 163. Tandem Axle Load Truck

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 10 through 12 illustrate each load path.

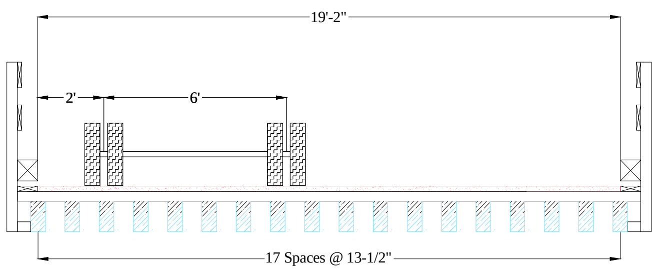

Figure 164. Transverse Truck Position - Load Path 1

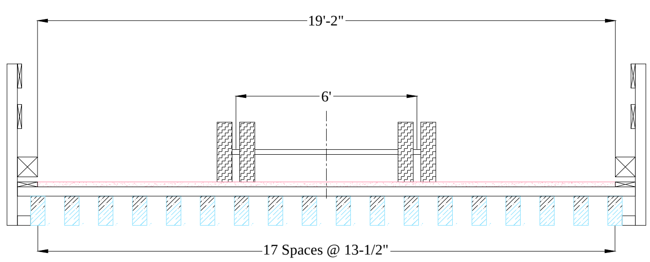

Figure 165. Transverse Truck Position - Load Path 2

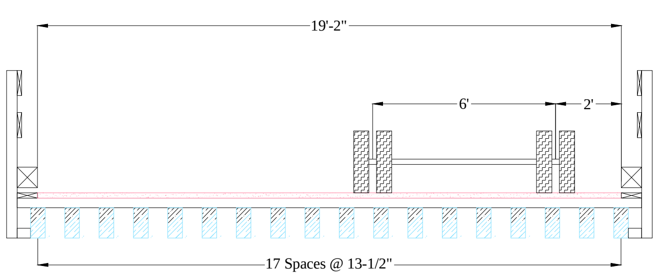

Figure 166. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 seems (see Figure 13). 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.

Figure 167. Minor Transverse Cracks

Deck

The deck appeared to be in good condition and 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.

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 168 shows the typical condition of the superstructure.

Figure 168. Typical Superstructure Condition

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 besides 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 Figure 15).

Figure 169. Advanced Degradation throughout Curbs

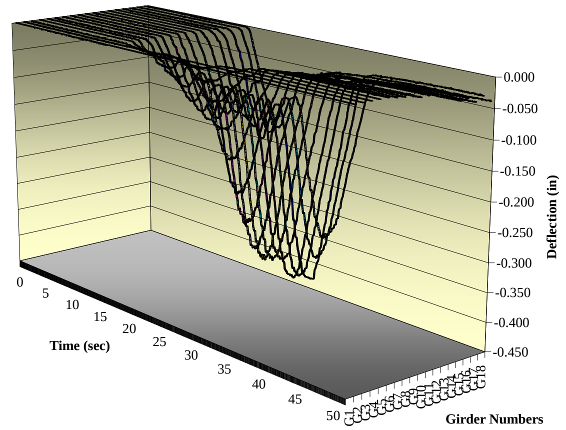

Figure 170. Deflections Load Path 1

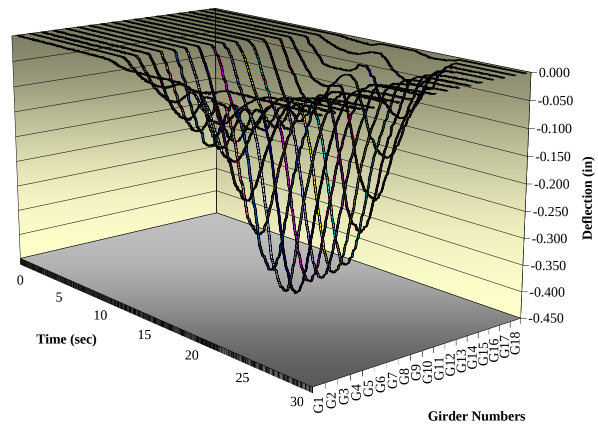

Figure 171. Deflections Load Path 2

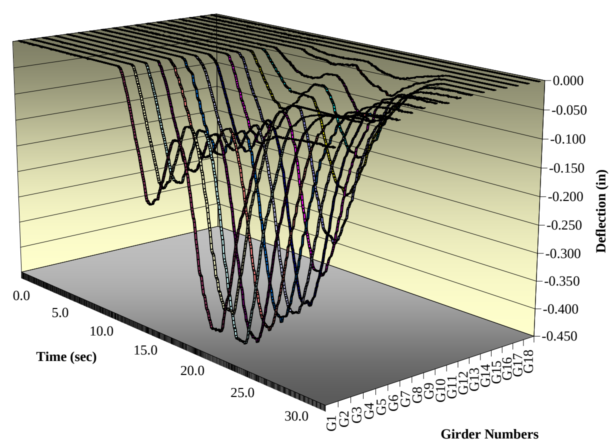

Figure 172. Deflections Load Path 3

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 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 16 through 18 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. Looking to the above mentioned figures this two wave relationship is quite evident 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 1. Each passing of the three load paths is illustrated in Figures 19 through 21. 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.

Table 26. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.411	0.412	0.448

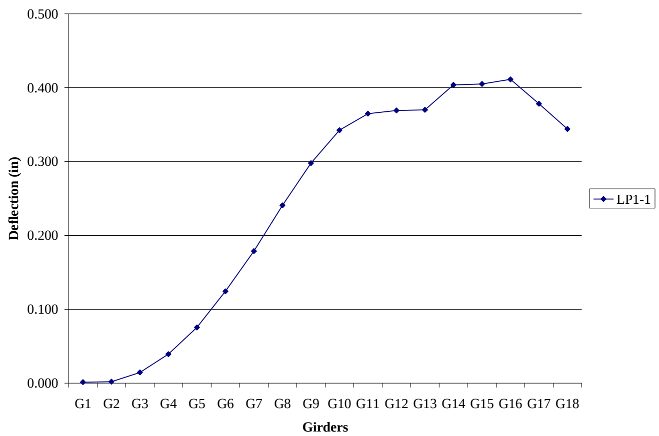

Figure 173. Maximum Deflections for Load Path 1

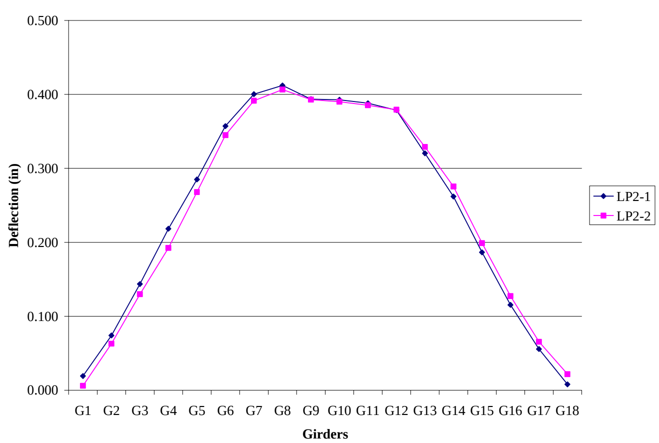

Figure 174. Maximum Deflections for Load Path 2

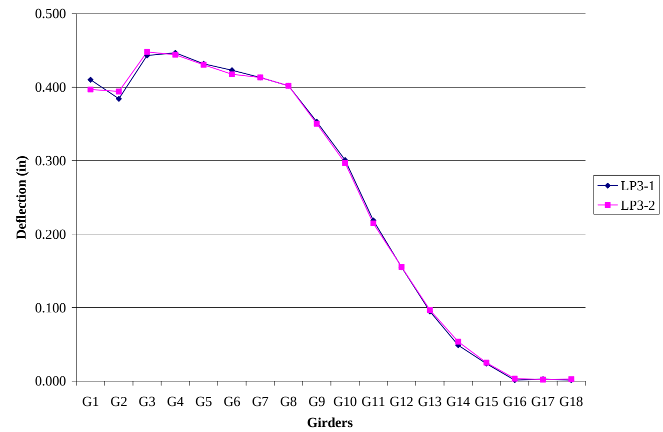

Figure 175. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 27. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 11

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 28. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
349	349	321

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 of the n-values are below the recommended n-values stated in Table 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 12

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

- LF_i = load fraction of the i^{th} girder
- Δi = deflection of the i^{th} girder
- $\Sigma\Delta_i$ = sum of all girder deflections
- n = number of girders

Figure 22 shows the load fractions for each girder for each load path.

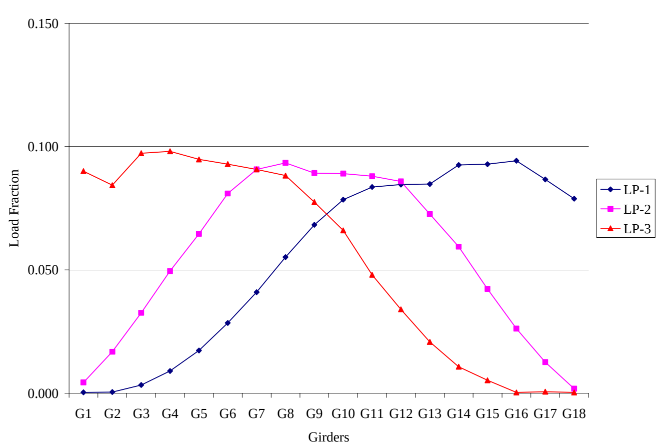

Figure 176. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

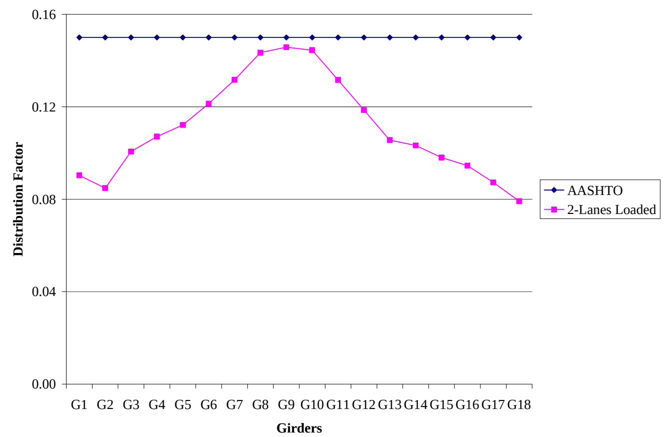

Figure 177. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

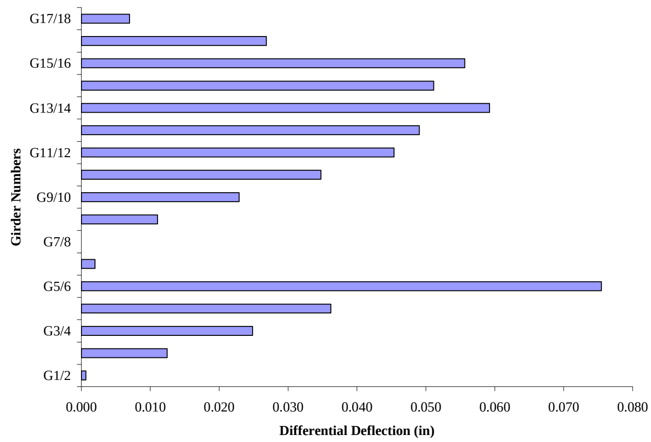

Figure 178. Differential Deflections for Load Path 1

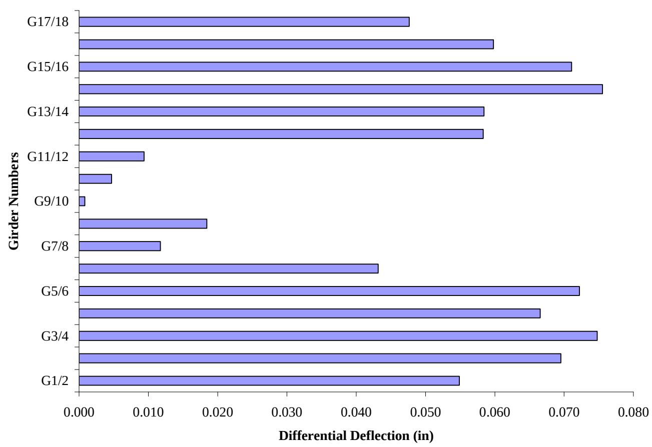

Figure 179. Differential Deflections for Load Path 2

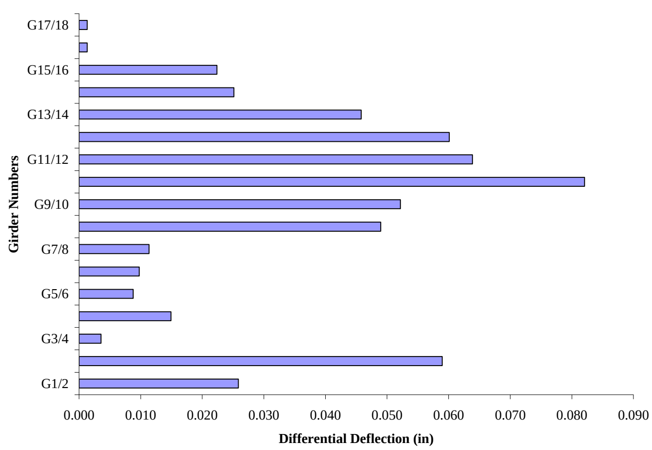

Figure 180. Differential Deflections for Load Path 3

Table 29. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.075	0.077	0.082

The maximum differential deflection of 0.082 in. occurs in load path 3. This is approximately 18 percent 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 and 631 and 648 microstrain, respectively. The strain plot at midspan is shown in Figures 27 through 29 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 27 through 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, i.e., the girders act independently of the deck when subjected to bending.

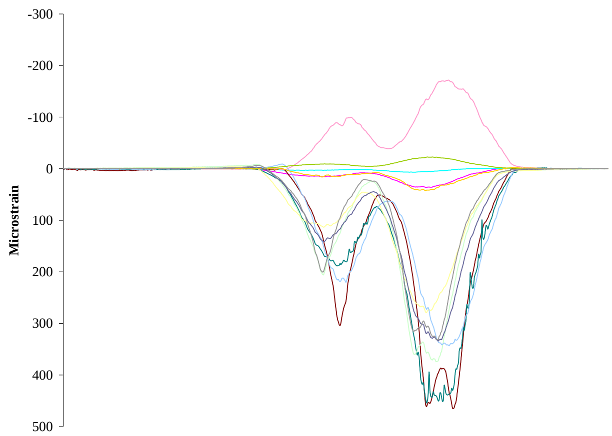

Figure 181. Strain at Midspan for Load Path 1

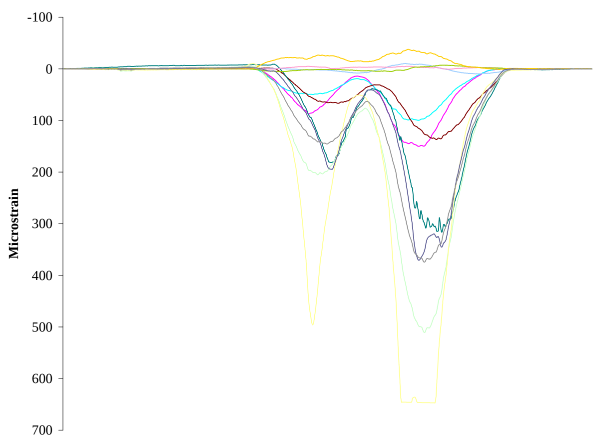

Figure 182. Strain at Midspan for Load Path 2

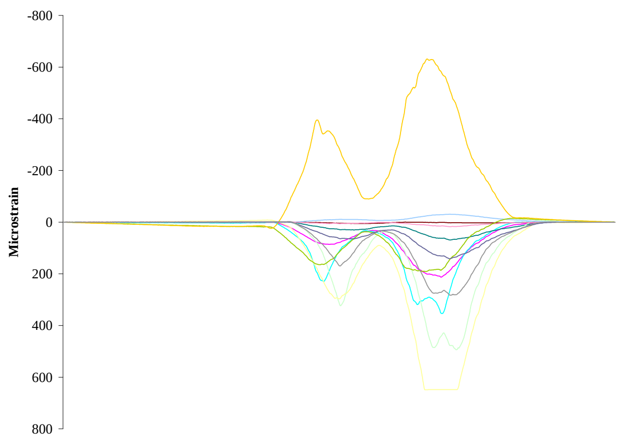

Figure 183. Strain at Midspan for Load Path 3

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, 9, and 18 at midspan and at the east abutment. The bottom of the deck between girders 1 and 2, 9 and 10, and 17 and 18 was measured at midspan. Measurements ranged from 14.5 to 22.2 percent. Overall, significant moisture content differences were not found throughout the girders and the deck measurement of 14.5 percent was the exception. The moisture content measurements are summarized in Table 5.

Table 30. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	22.2
Bottom of Deck Between Girders 9 & 10	14.5
Bottom of Deck Between Girders 17 & 18	21.2

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, 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, i.e., 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 27 though 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 631 and 648 microstrain, respectively. These strains equate to maximum stresses of 726 and 745 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 973 and 998 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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. It is evident however that the bridge has been recently rehabilitated as the overall wearing surface, deck, and superstructure appears 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 1500 psi 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.

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APPENDIX H

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 990163

United States
Department of
Agriculture

Forest Service

Forest Products
Laboratory

Iowa State
University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 990163

Terry Wipf
Brent Phares
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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.

Acknowledgements

We would like to 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.

Dennis Burleson, North Carolina Department of Transportation employee who operated the load truck during testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Doebag Creek Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 990163, hereinafter referred to as the Doebag Creek Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 184. Bridge Location

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 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 degree 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 Figure 185).

Figure 185. Substructure Sill and Posts

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 2.

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 one helper, spaced 1 ft-9 in. center-to-center, measuring 6 in. x 12 in. in cross-section are present and are seated and toe-nailed to the 10-in. x 10-in. timber sills with spikes. The deck consists of individual 4 in. x 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 4 illustrates the layout of the bridge.

Figure 186. Doebag Creek Parapet

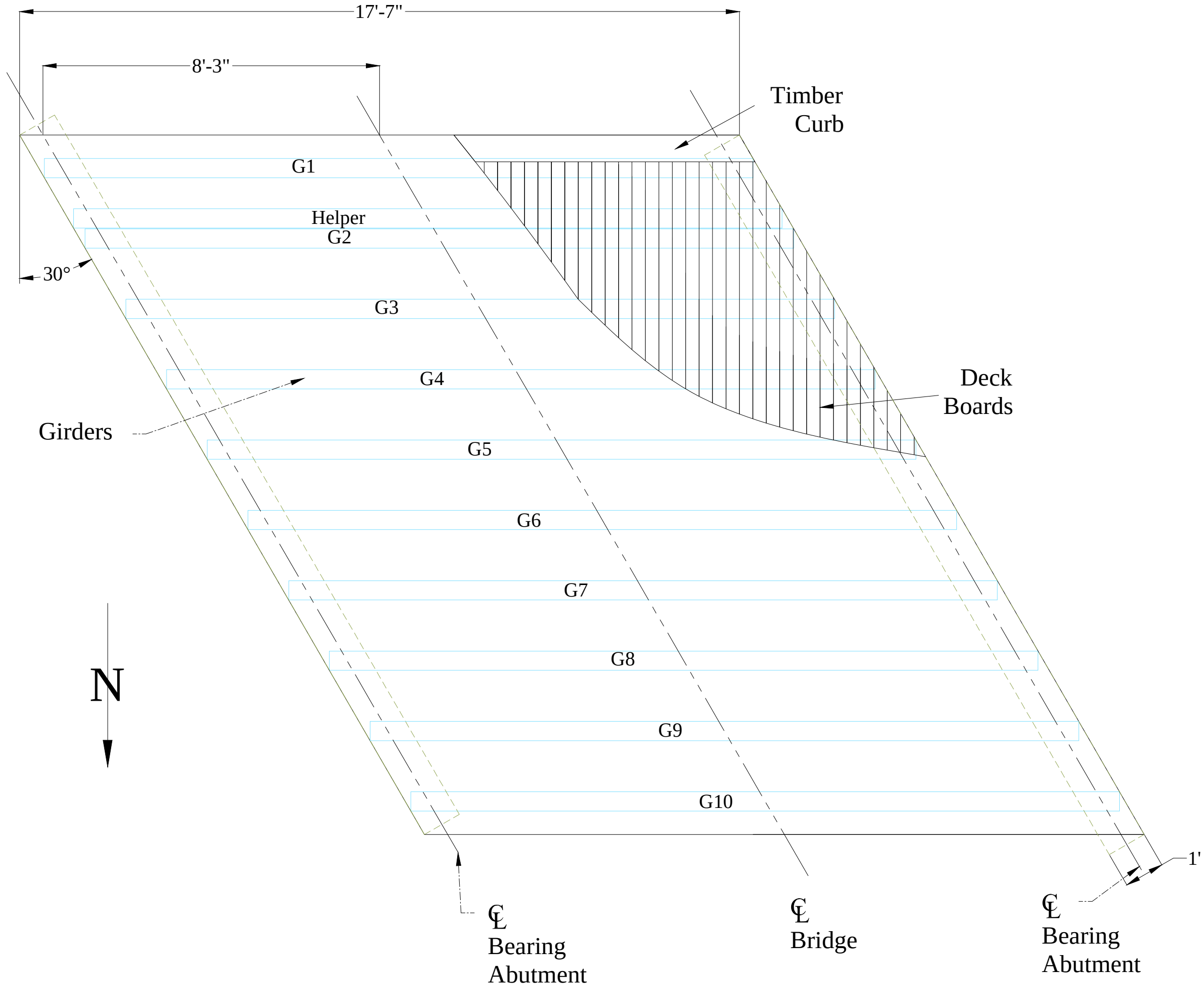
Evaluation Methodology

The 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 was collected through the use of ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 188. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.



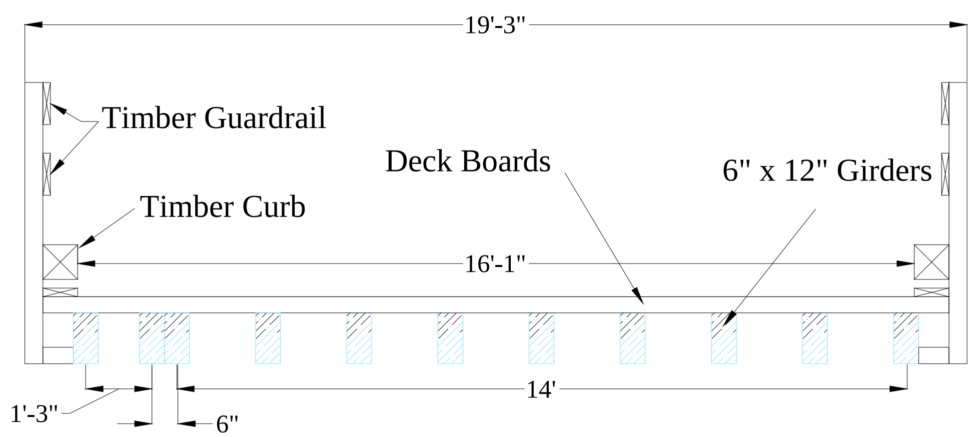
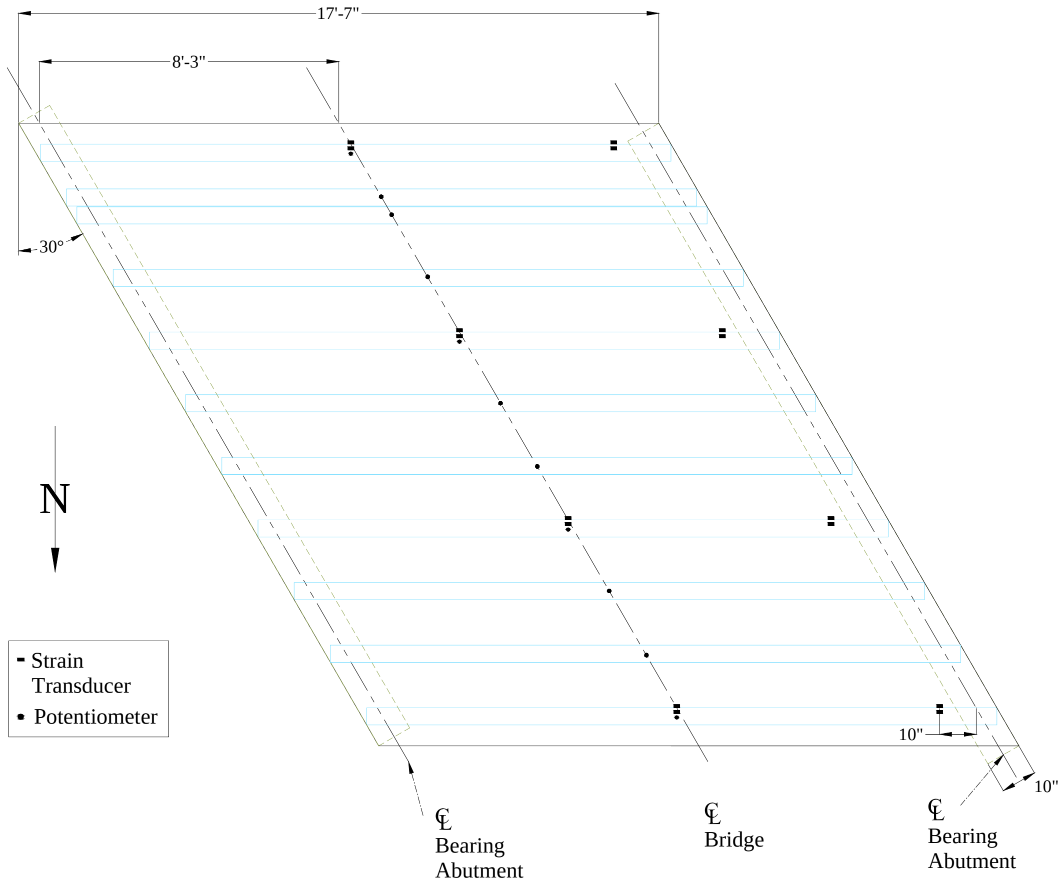


Figure 187. Plan and Profile Layout of Doebag Creek Bridge



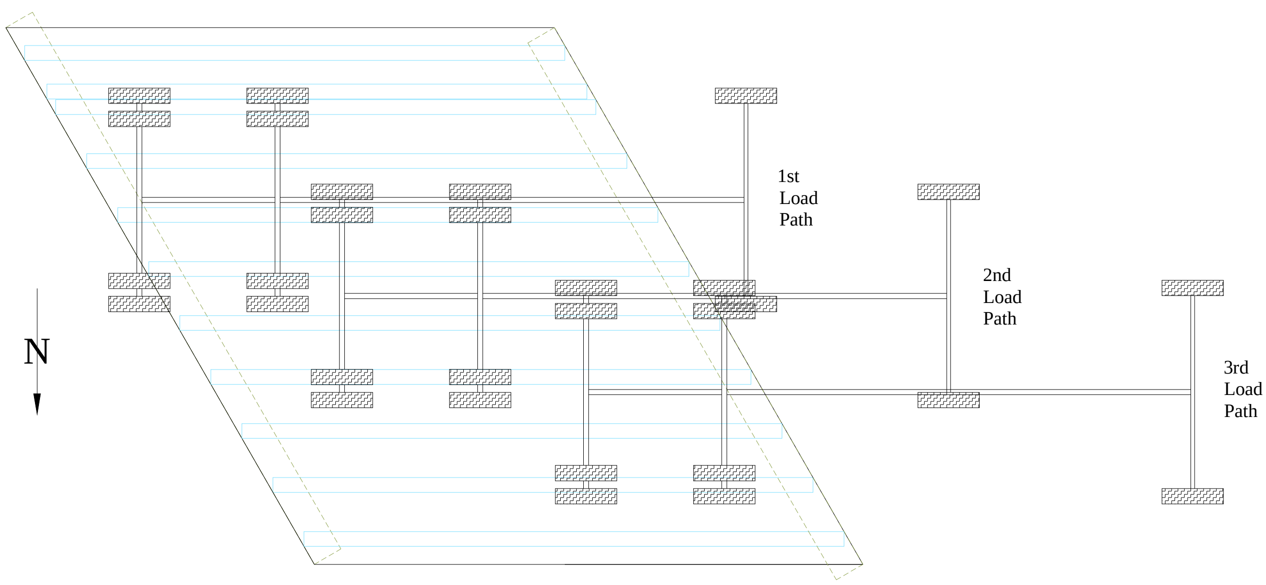


Figure 188. Instrumentation and Load Paths of Doebag Creek Bridge

Figure 189. Deflection Instrumentation

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 Figure 188). 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 Figure 190). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 190. Strain Transducers

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.

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 8. 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 lbs and the total weight on each rear axle and the front axle is 19,330 and 16,660 lbs, respectively.

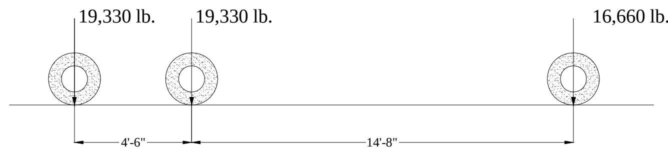


Figure 191. Truck Configuration and Axle Loads

Figure 192. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

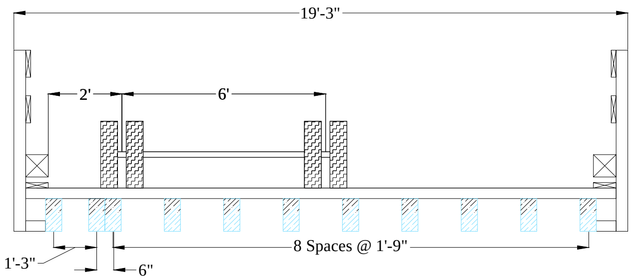


Figure 193. Transverse Truck Position - Load Path 1

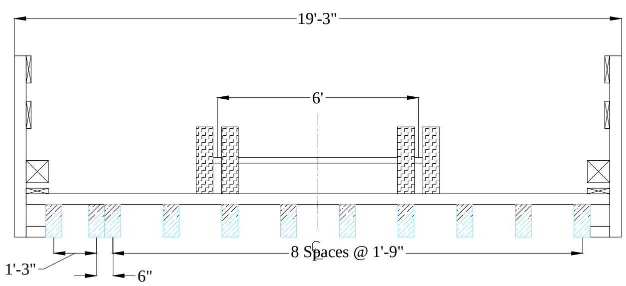


Figure 194. Transverse Truck Position - Load Path 2

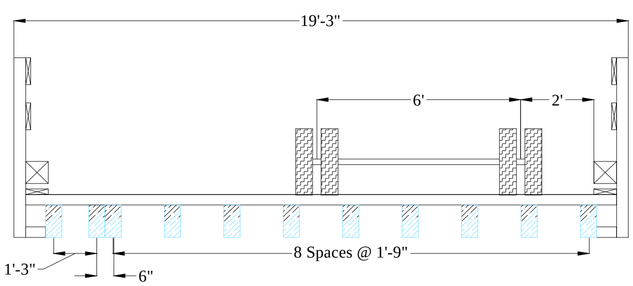


Figure 195. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 Figure 196) and scattered fungus on the underside of the deck was 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.

Figure 196. Decay in the Ends of the Deck Boards

Superstructure

NC-DOT 2003 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 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. Figure 197 through Figure 200 show the condition of the superstructure. The bearing of the girders on the sill was sufficient and there is no misalignment.

Figure 197. Girder and Deck Condition

Figure 198. Fungal Growth throughout Superstructure

Figure 199. Mechanical Damage to Girder

Figure 200. Checking in the Girders

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 Figure 201). There were minor checks in the parapet and parapet curb.

Figure 201. Foliage Overgrowth

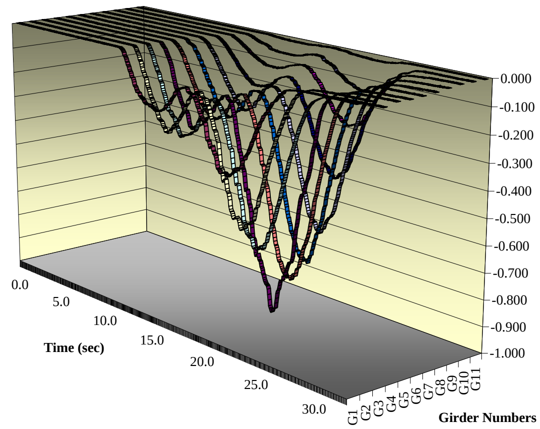


Figure 202. Deflections Load Path 1

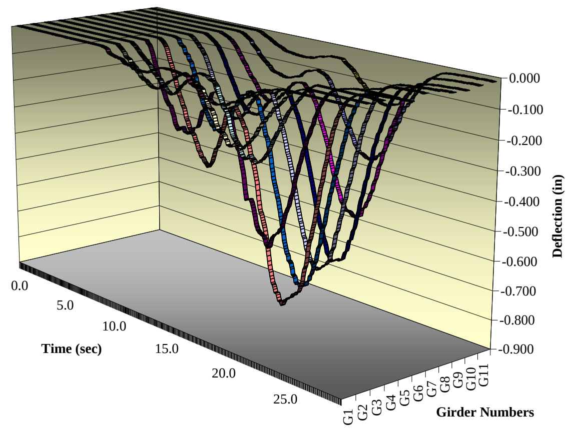


Figure 203. Deflections Load Path 2

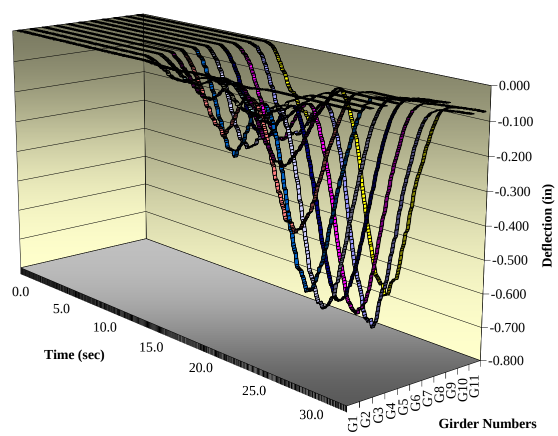


Figure 204. Deflections Load Path 3

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 19 through 21 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. Looking to the above mentioned figures this two wave relationship is quite evident 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 1. Each passing of the three load paths is illustrated in Figures 22 through 24. 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.

Table 31. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in)		
Load Path 1	Load Path 2	Load Path 3
0.908	0.814	0.784

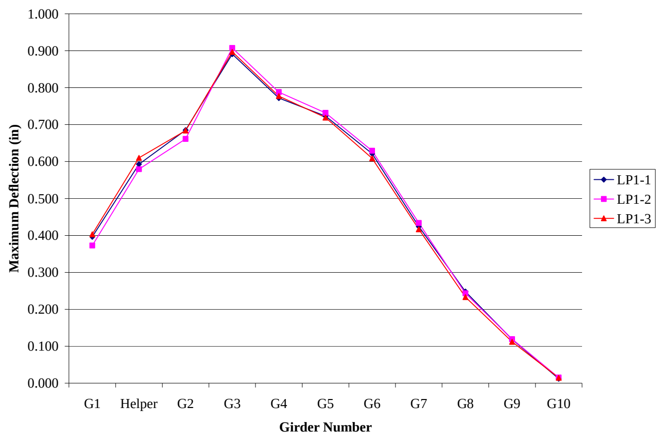


Figure 205. Maximum Deflections for Load Path 1

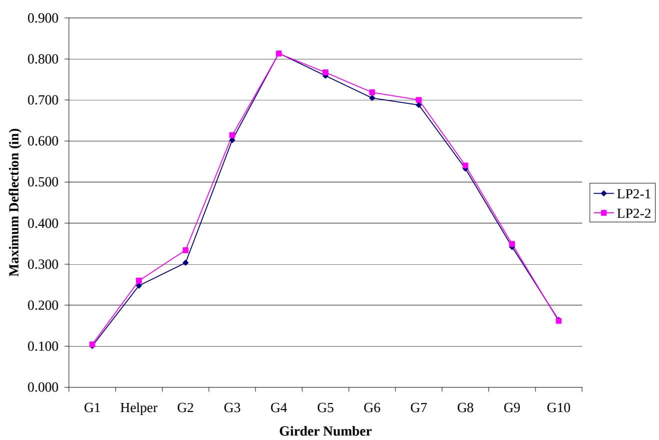


Figure 206. Maximum Deflections for Load Path 2

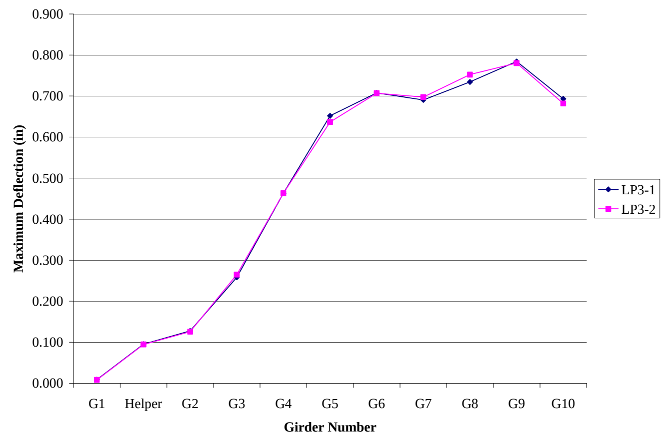


Figure 207. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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. Though this particular bridge does not have a wearing surface the recommended live load deflection limit states are included nevertheless.

Table 32. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 13

$$n = \frac{Length}{Deflection \times \frac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 33. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
163	181	188

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 of the n-values are below the recommended n-values stated in Table 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 14

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i = distribution factor of the ith girder
Δi = deflection of the ith girder
$\Sigma\Delta_i$ = sum of all girder deflections
n = number of girders

Figure 22 shows the load fractions for each girder for each load path.

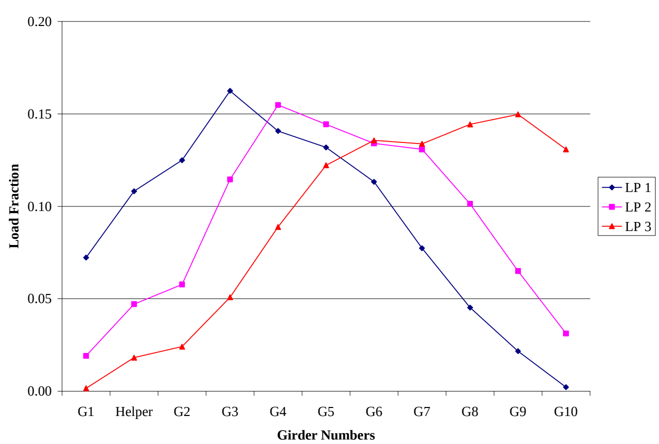


Figure 208. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

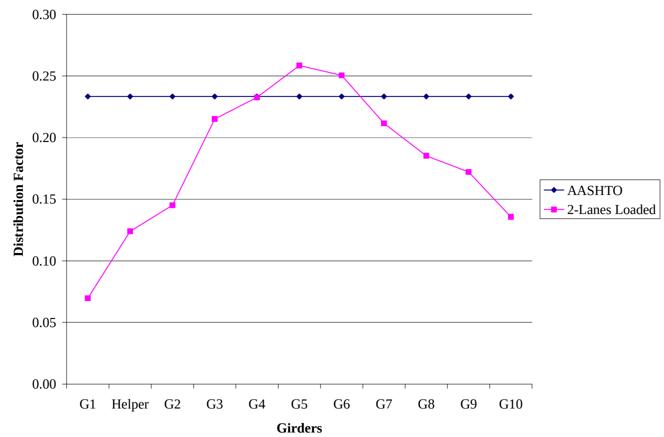


Figure 209. AASHTO Design Live Load Distribution

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 affects 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. Again, it should be noted that both of these criterion 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 27 through 29 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 4.

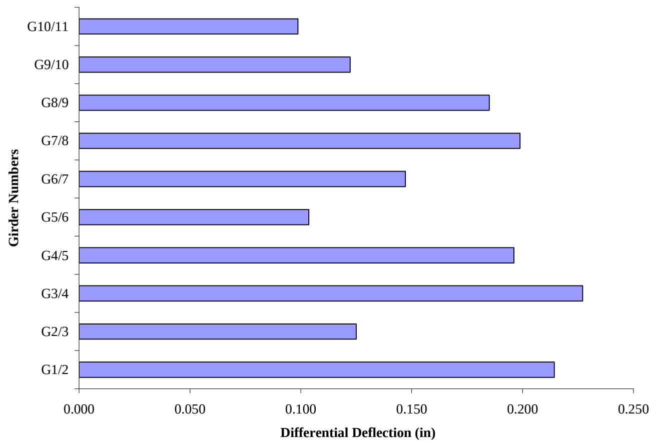


Figure 210. Differential Deflections for Load Path 1

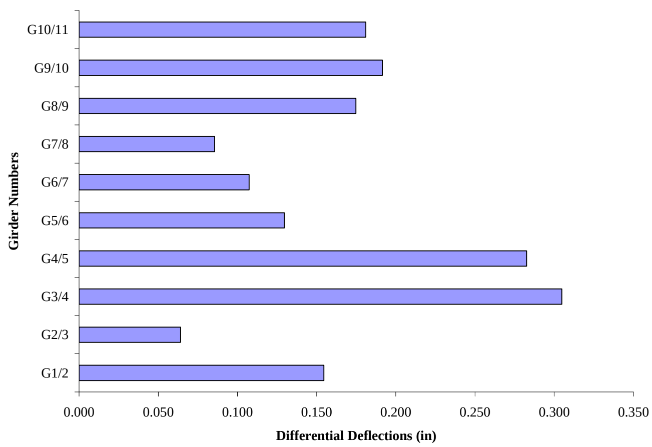


Figure 211. Differential Deflections for Load Path 2

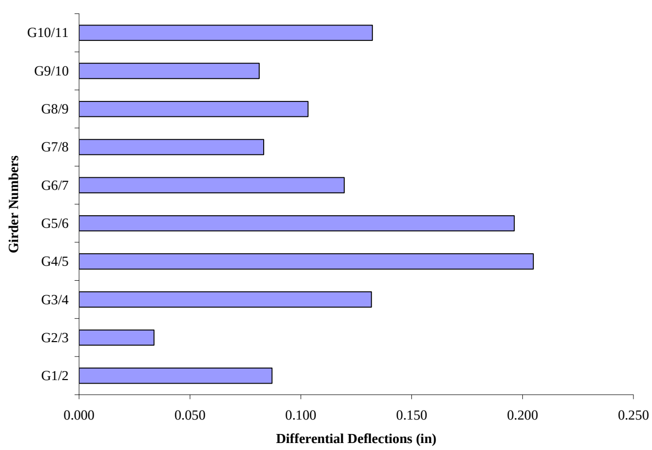


Figure 212. Differential Deflections for Load Path 3

Table 34. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in)		
Load Path 1	Load Path 2	Load Path 3
0.227	0.305	0.205

The maximum differential deflection of 0.305 in occurs in load path 2. This is nearly 38 percent 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 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 30 through 32 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 30 through 32 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, i.e. the girders act independently of the deck when subjected to bending.

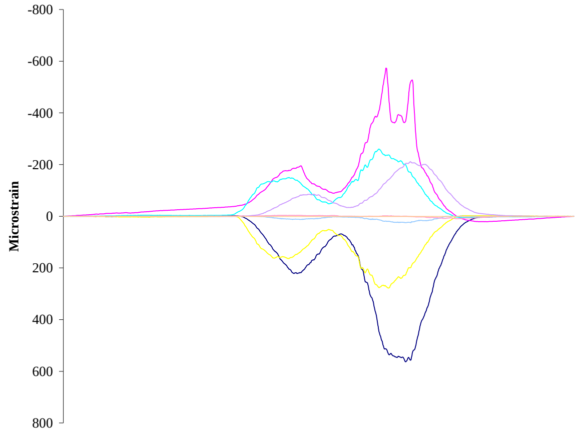


Figure 213. Truck Position versus Strain at Midspan for Load Path 1

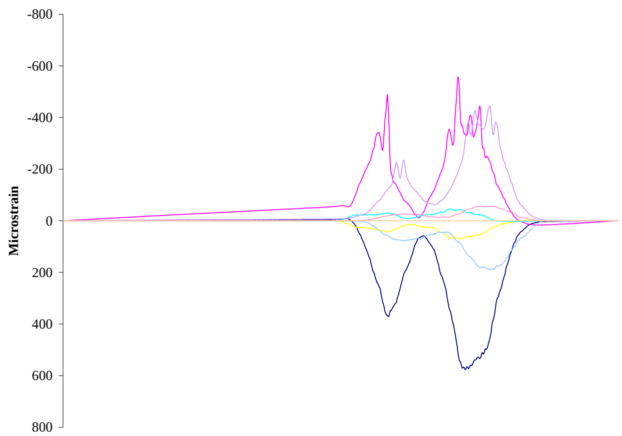


Figure 214. Truck Position versus Strain at Midspan for Load Path 2

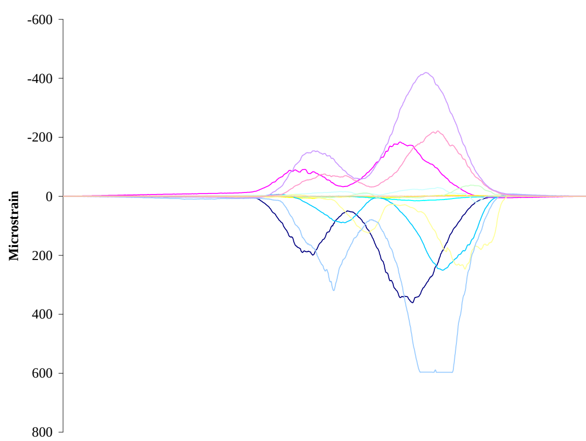


Figure 215. Truck Position versus Strain at Midspan for Load Path 3

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 percent. Overall, significant moisture content was found throughout the bridge. The moisture content measurements are summarized in Table 5.

Table 35. Moisture Content Summary

Moisture Content Reading Locations and Values	
Location	%
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 & 2	23.4
Bottom of Deck Between Girders 5 & 6	30+
Bottom of Deck Between Girders 9 & 10	25.2

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.

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.

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 limits their use for primary analysis. Nevertheless, Figures 30 through 32 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 851 and 888 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 1500 psi 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.

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APPENDIX I

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 580125

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NORTH CAROLINA BRIDGE NO. 580125

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 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.

Acknowledgements

We would like to 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.

Jerry Smith, Marion Bridge Maintenance employee who operated the load truck during testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the McDowell County Bridge in western North Carolina.

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, (i.e., 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.

Background

The location of North Carolina state bridge number 580125, hereinafter referred to as the McDowell County Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 216. McDowell County Bridge in North Carolina

The McDowell County Bridge was built in 1958 and is located in McDowell County in western North Carolina 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 degree 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 Figure 217).

Figure 217. McDowell County Bridge Substructure

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 2.

Figure 218. McDowell County 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 in. x 12-1/4 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 4 in. x 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 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 6.

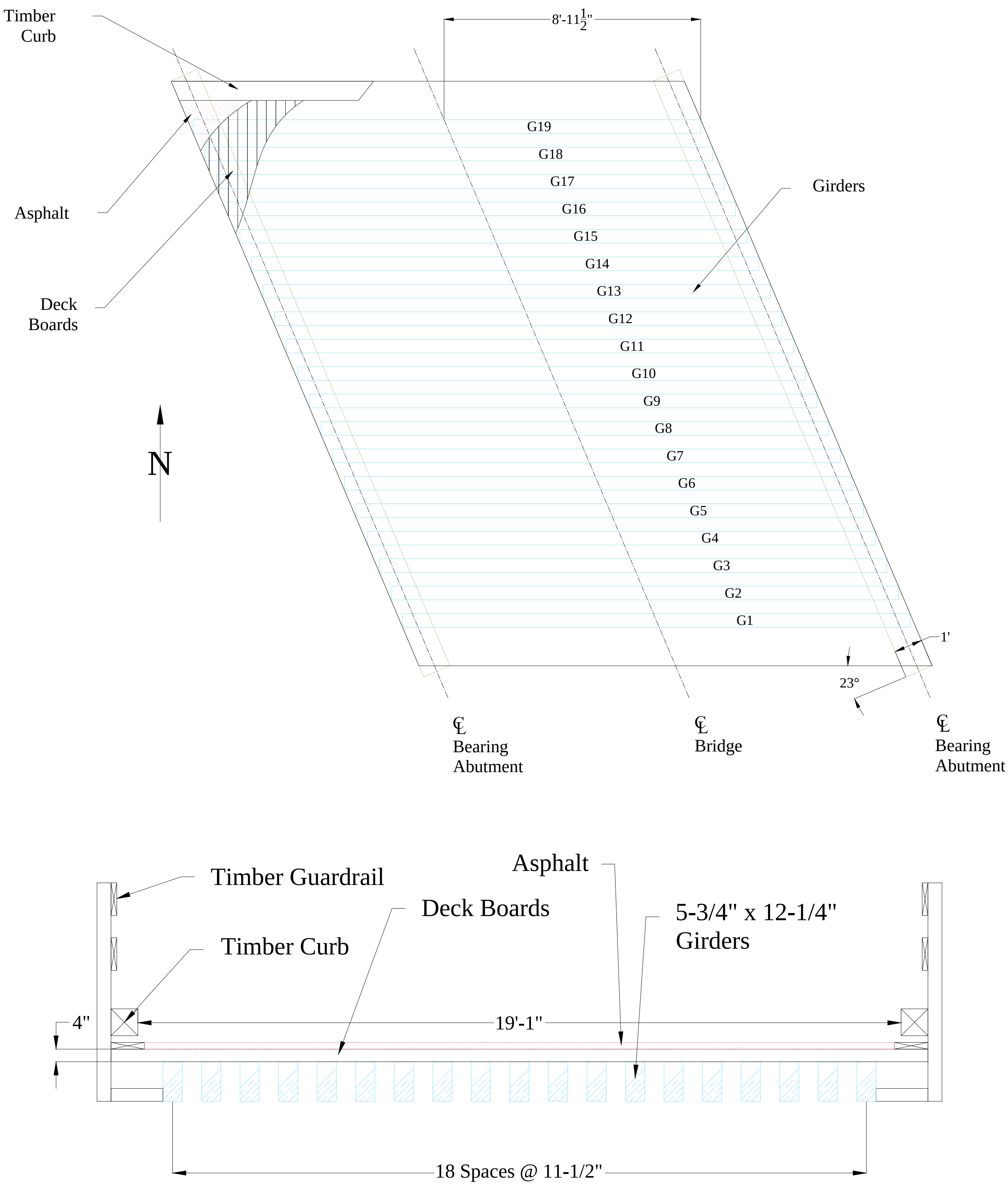

Figure 219. Plan and Profile Layout of McDowell County Bridge

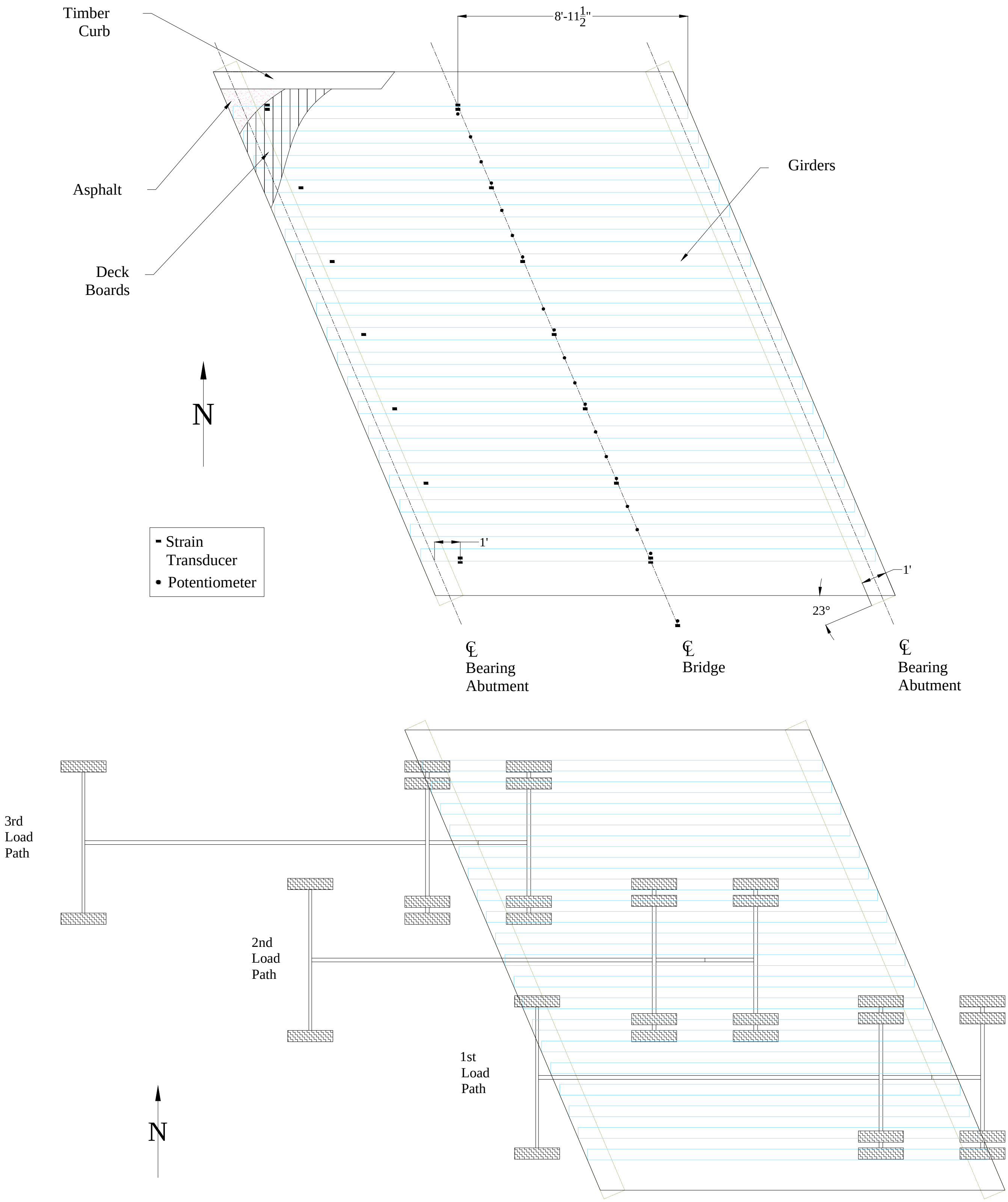

Figure 220. Instrumentation and Load Paths of McDowell County Bridge

Figure 221. Deflection Instrumentation

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 Figure 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 7. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Figure 222. Strain Transducers

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.

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 8. 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 lbs. Typically, 70 percent 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 lbs and 17,118 lbs, respectively. The truck used for load testing is shown in Figure 224.

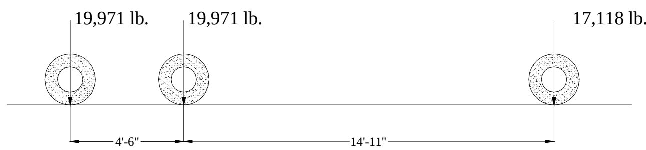

Figure 223. Truck Configuration and Axle Loads

Figure 224. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figure 227). 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.

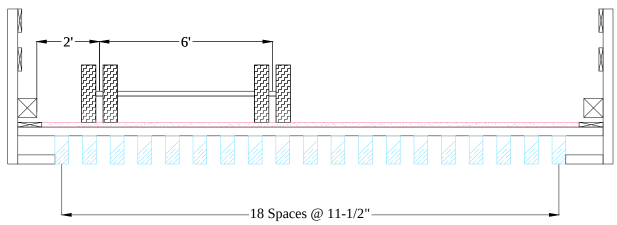

Figure 225. Transverse Truck Position - Load Path 1

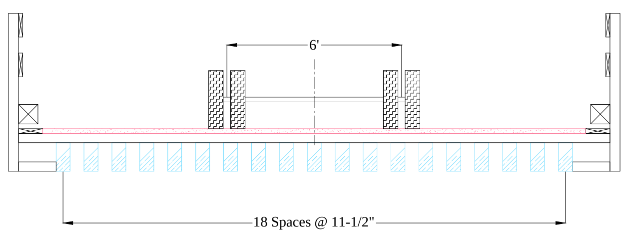

Figure 226. Transverse Truck Position - Load Path 2

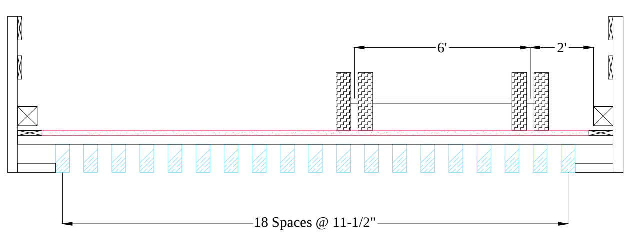

Figure 227. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 228. The asphalt pavement generally looked to be in good condition aside from the transverse cracking.

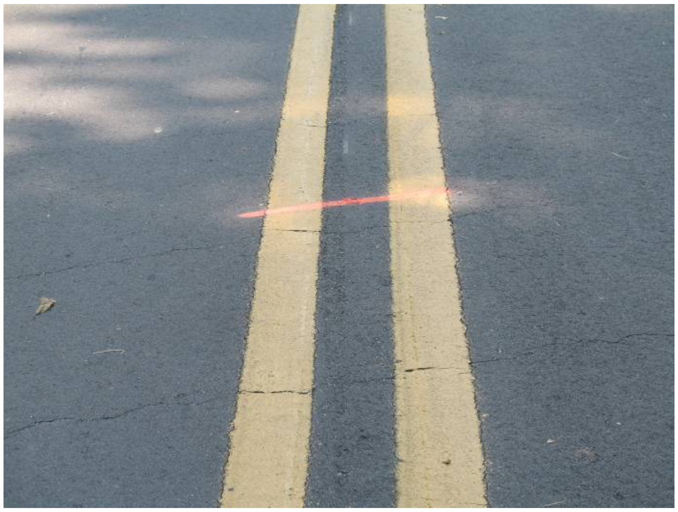

Figure 228. Observed Transverse Cracks

Deck

The deck appeared to be in good condition and there was no visible detachment of the deck boards from the girders and all deck boards were securely fastened. Some very minor 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 the past the girder face (see Figure 229). Very minor checking was present near the centerline of some girders 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.

Figure 229. Moss Growth on Girder 19

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 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 Figure 230). 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.

Figure 230. Wingwall Weathering and Decay

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 16 through 18 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

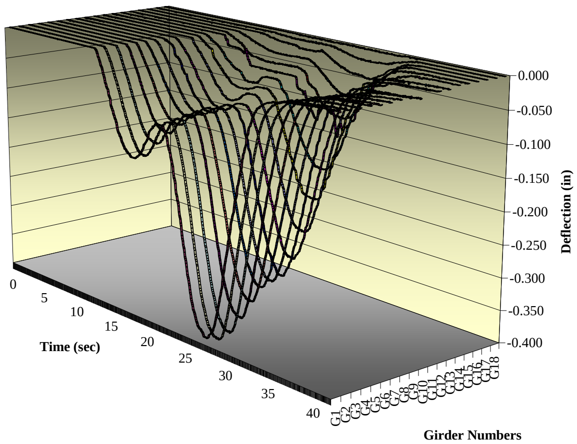

Figure 231. Deflections Load Path 1

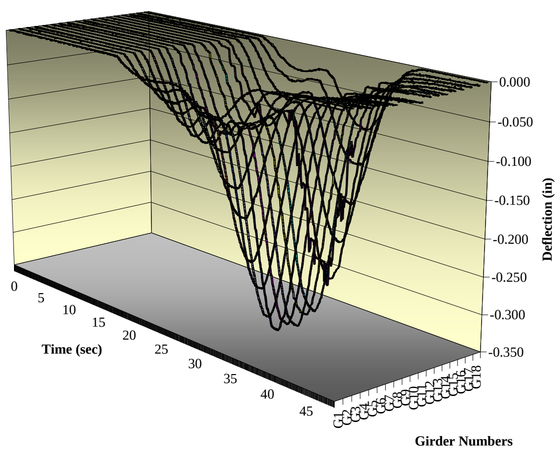

Figure 232. Deflections Load Path 2

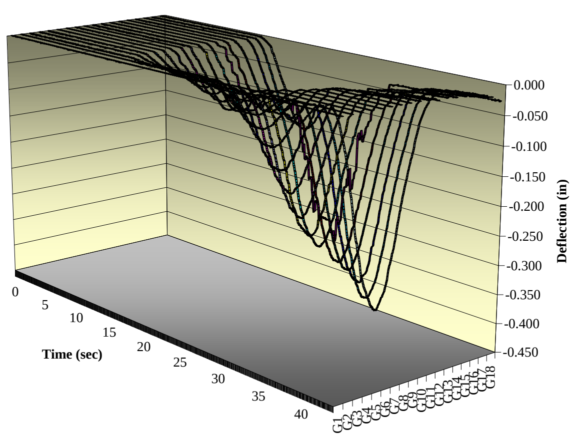

Figure 233. Deflections Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 19 through 21. 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.

Table 36. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.403	0.347	0.448

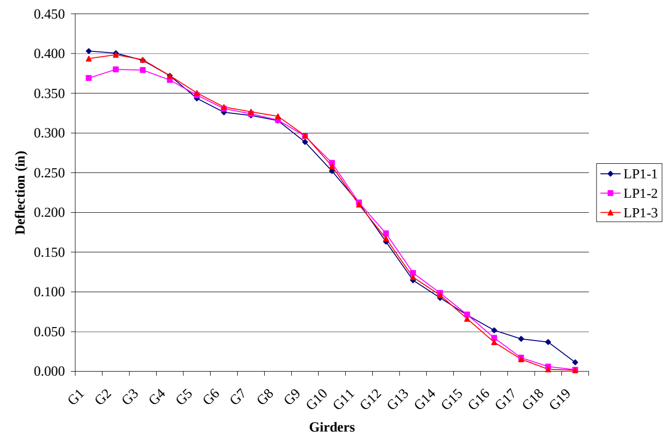

Figure 234. Maximum Deflections for Load Path 1

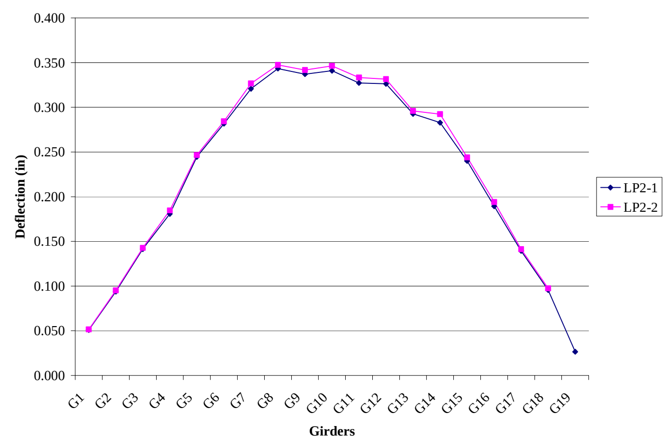

Figure 235. Maximum Deflections for Load Path 2

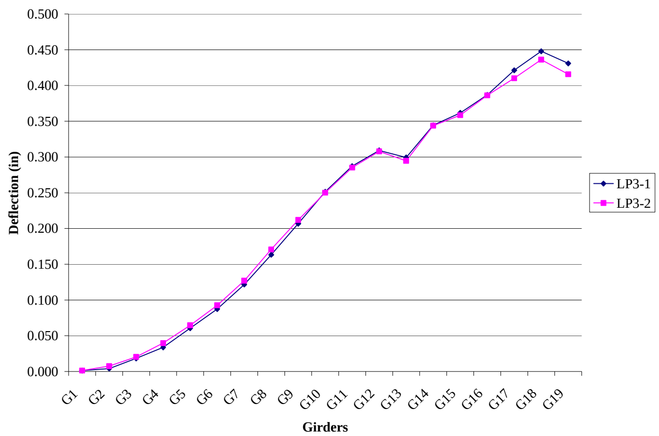

Figure 236. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 37. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 15

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 38. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
379	435	337

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 38. Most Critical n-ValuesTable 3. The possible reasons for deflections greater than those recommended will be discussed later.

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 Equation 2.

Equation 16

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

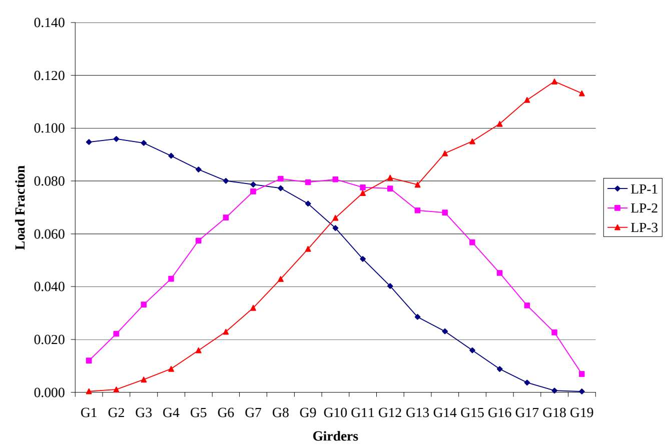

Figure 237. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

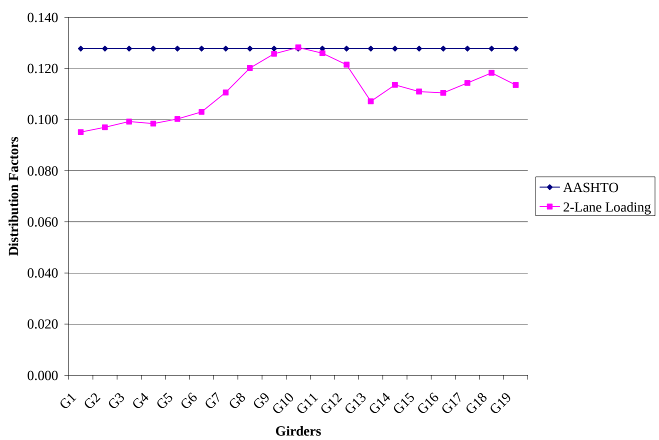

Figure 238. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

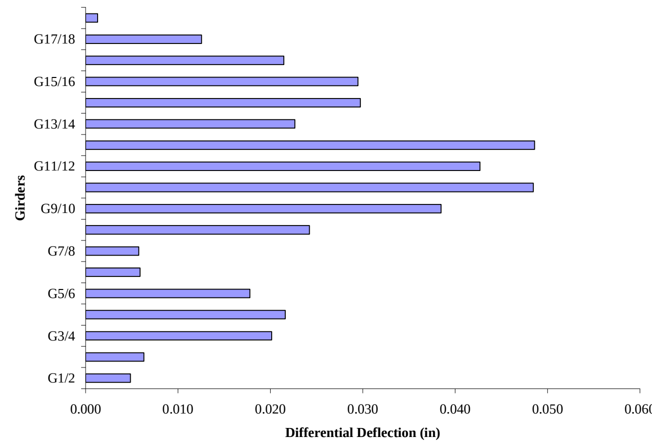

Figure 239. Differential Deflections for Load Path 1

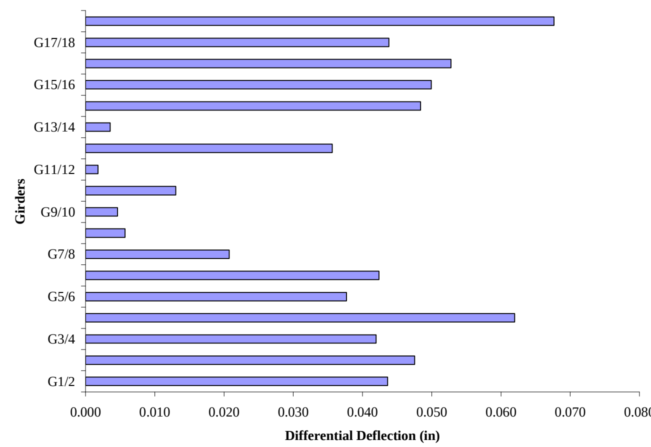

Figure 240. Differential Deflections for Load Path 2

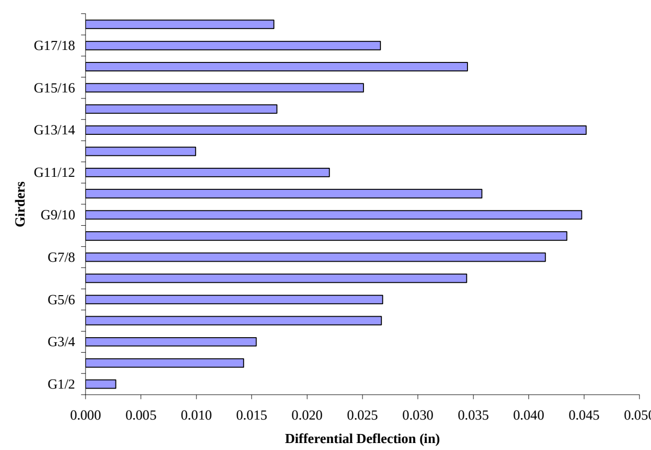

Figure 241. Differential Deflections for Load Path 3

Table 39. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.049	0.068	0.045

The maximum differential deflection of 0.068 in. occurs in load path 2. This is nearly 20 percent 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.

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 27 through 29 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 through 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, i.e., the girders act independently of the deck when subjected to bending.

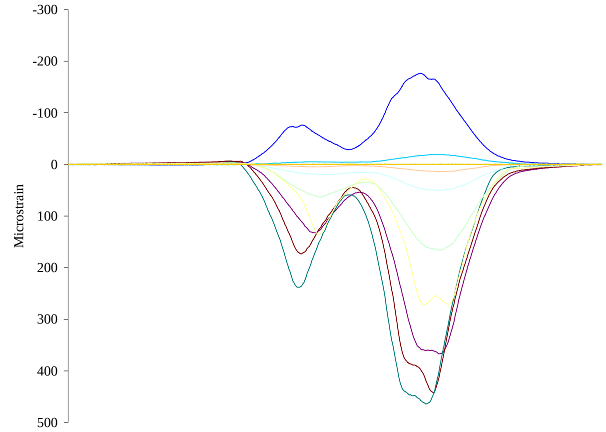

Figure 242. Strain at Midspan for Load Path 1

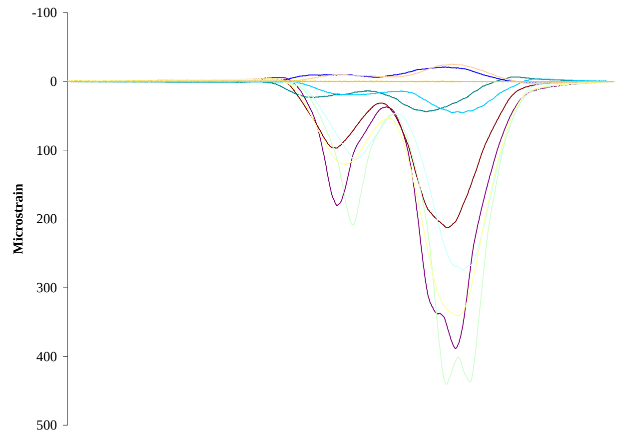

Figure 243. Strain at Midspan for Load Path 2

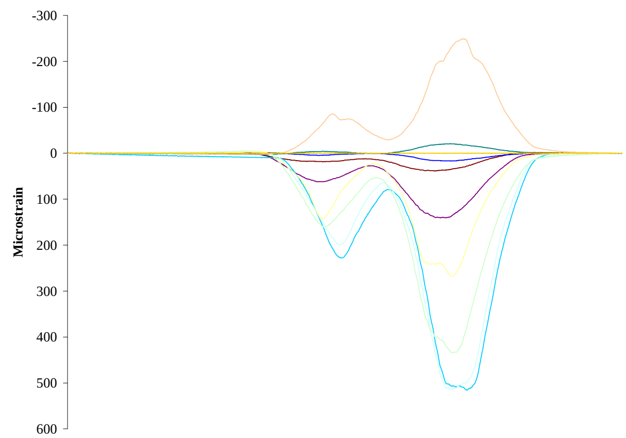

Figure 244. Strain at Midspan for Load Path 3

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, 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 percent. The moisture content measurements are summarized in Table 5.

Table 40. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
Girder 1, West Abutment	18.0
Girder 1, Midspan	16.1
Girder 10, West Abutment	16.8
Girder 10, Midspan	19.3
Girder 19, West Abutment	21.0
Girder 19, Midspan	22.7
Bottom of Deck Between Girders 1 & 2	23.8
Bottom of Deck Between Girders 9 & 10	24.2
Bottom of Deck Between Girders 18 & 19	18.3

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, 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 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, i.e., 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 27 though 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 250 and 516 microstrain, respectively. These strains equate to maximum stresses of 289 and 593 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 362 and 742 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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. 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 these 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 psi 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.

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APPENDIX J

PERFORMANCE REPORT

LARIMER COUNTY, CO BRIDGE NO. LR5J-0.2-70

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

LARIMER COUNTY, CO BRIDGE NO. LR5J-0.2-70

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 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.

Acknowledgements

We would like to 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.

Dale Miller, Larimer County Road and Bridge employee who organized the load testing

Jim Graves, Larimer County employee who operated the load vehicle during live load testing

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Larimer County Bridge.

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, (i.e., 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.

Background

The location of Colorado state bridge LR5J-0.2-70, hereinafter referred to as the Larimer County Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

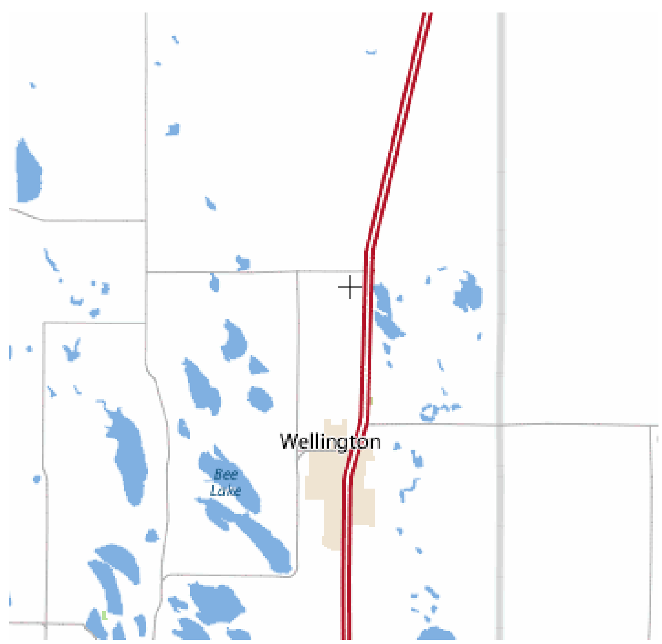

Figure 245. Larimer County Bridge Location

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).

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 Figure 2).

Figure 246. Bridge Substructure

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 2.

Figure 247. Larimer County Bridge Parapet Support

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 in. x 17-1/2 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 2 in. x 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 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.

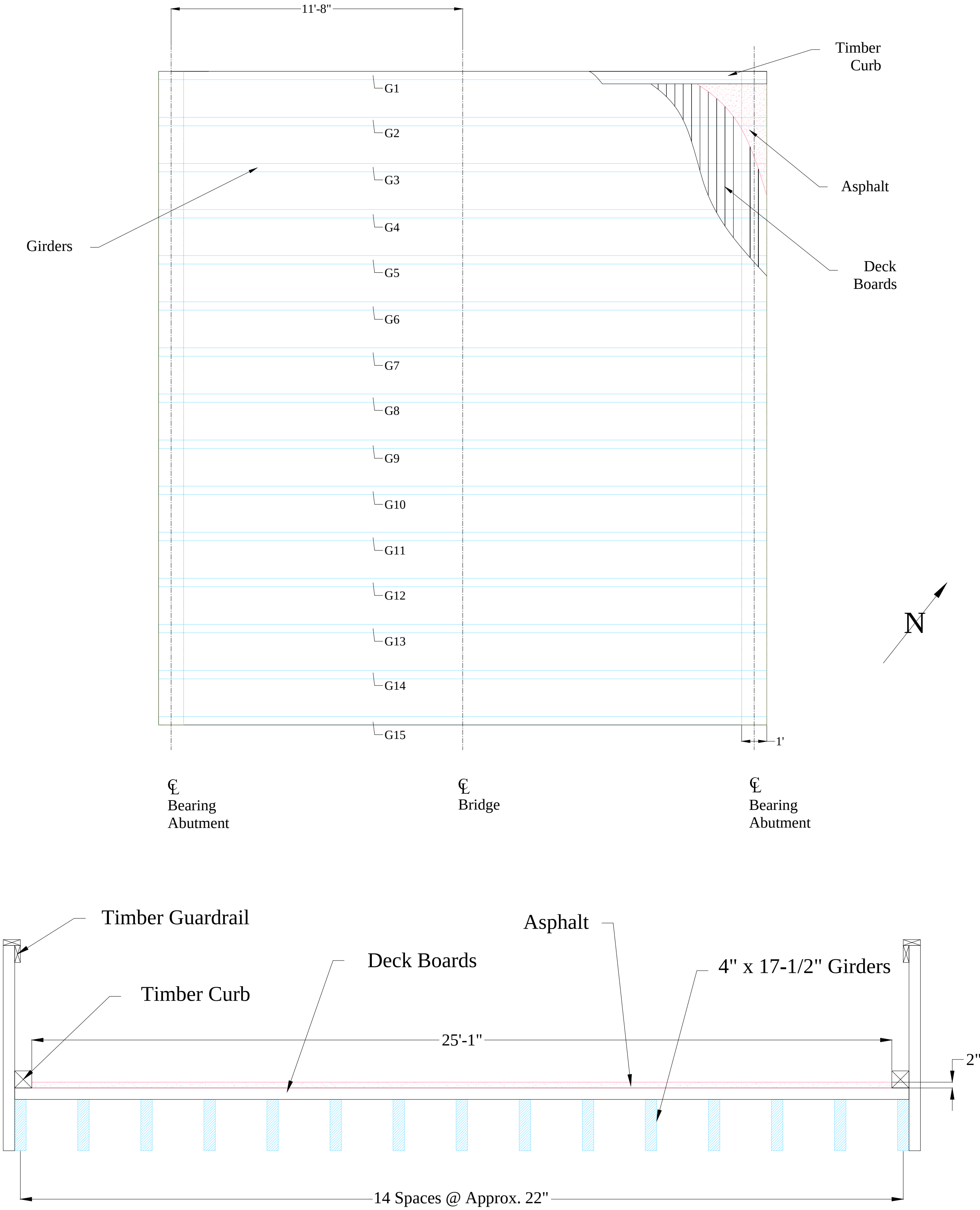

Figure 248. Plan and Profile Layout of Larimer County Bridge

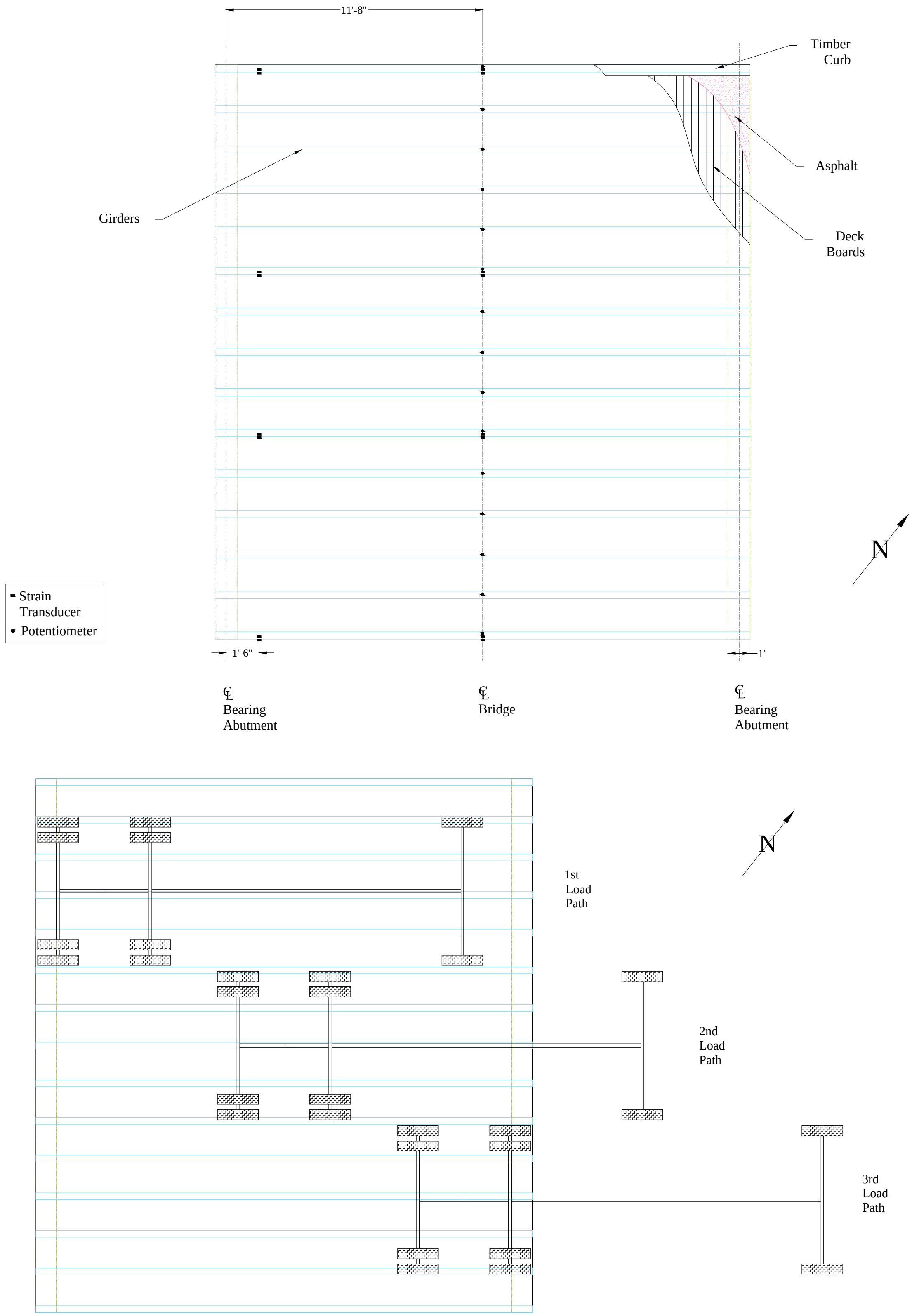

Figure 249. Instrumentation and Load Paths of Larimer County Bridge

Figure 250. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near the girder ends.

Figure 251. Strain Transducers

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.

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 8. 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 lbs. Assuming that 70 percent of the vehicle weight was distributed over the rear axles, the weight over the front axle and back axles was 16,200 lbs and 18,900 lbs, respectively. The assumed axle weights are shown in Figure 8 and the truck used for load testing is shown in Figure 253.

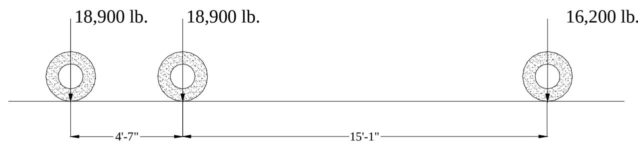

Figure 252. Truck Configuration and Axle Loads

Figure 253. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

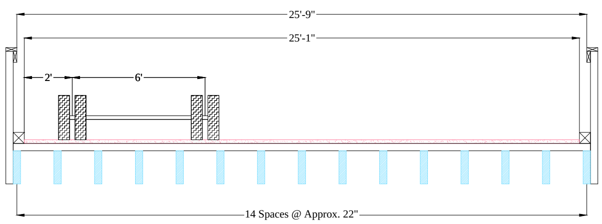

Figure 254. Transverse Truck Position - Load Path 1

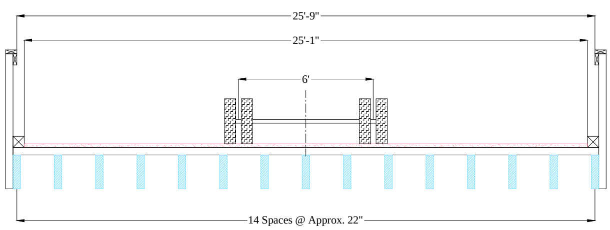

Figure 255. Transverse Truck Position - Load Path 2

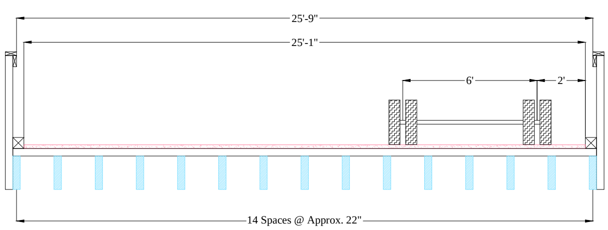

Figure 256. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 257 shows some of the cracking observed.

Aside from the transverse cracking, the wearing surface looked to be in satisfactory condition, though much of the bridge wearing surface is uneven.

Figure 257. Transverse Wearing Surface Cracking

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 and 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 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 258 shows the general superstructure condition, while Figure 259 shows the exterior girder condition.

Figure 258. General Superstructure Condition

Figure 259. 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 on the bearing seats in 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 ultraviolet degradation. The substructure also showed signs of minor checking. 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 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 16 through 18 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. Looking to the above mentioned figures this two wave relationship is evident and the deflections represent the difference in load from the front axle to the back axles.

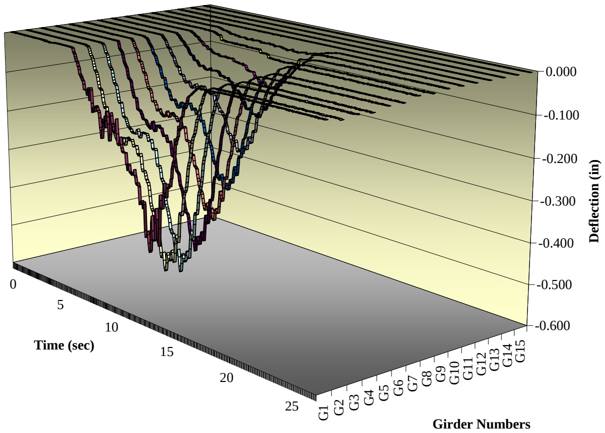

Figure 260. Deflections for Load Path 1

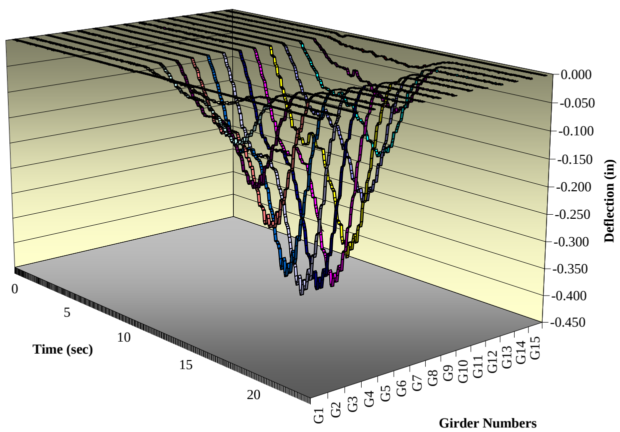

Figure 261. Deflections for Load Path 2

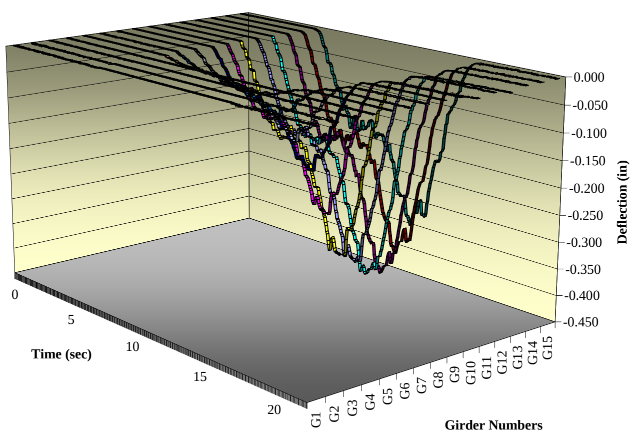

Figure 262. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 19 through 21. 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.

Table 41. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.505	0.407	0.425

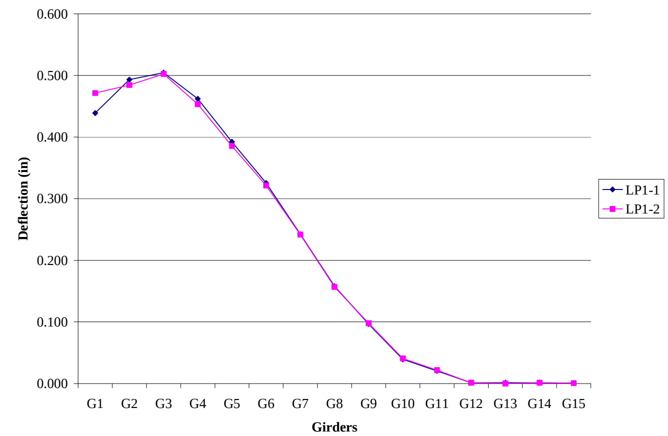

Figure 263. Maximum Deflections for Load Path 1

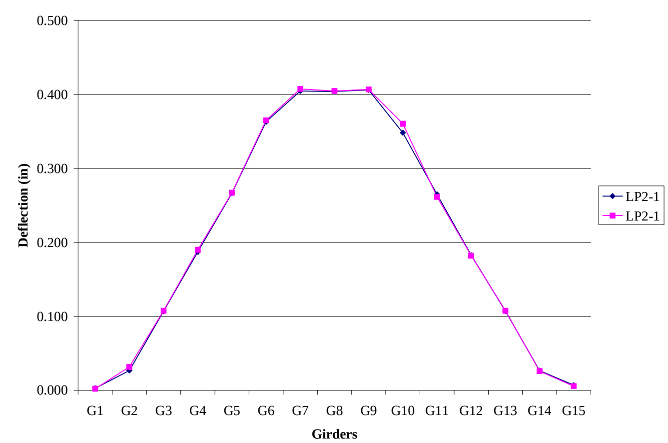

Figure 264. Maximum Deflections for Load Path 2

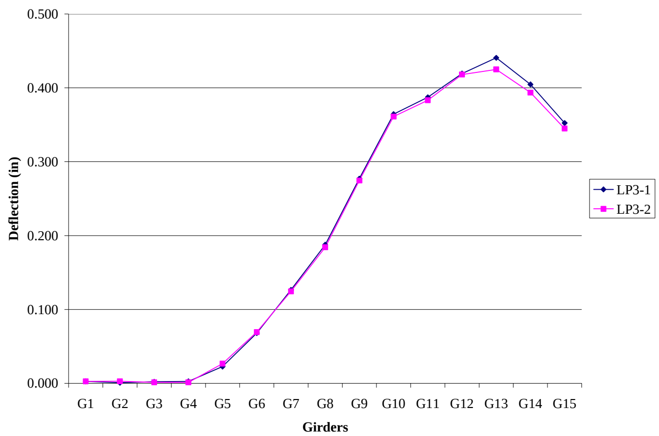

Figure 265. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 42. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 17

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 43. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
400	496	475

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 greater than the two of the recommended n-values stated in Table 3.

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 Equation 2.

Equation 18

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

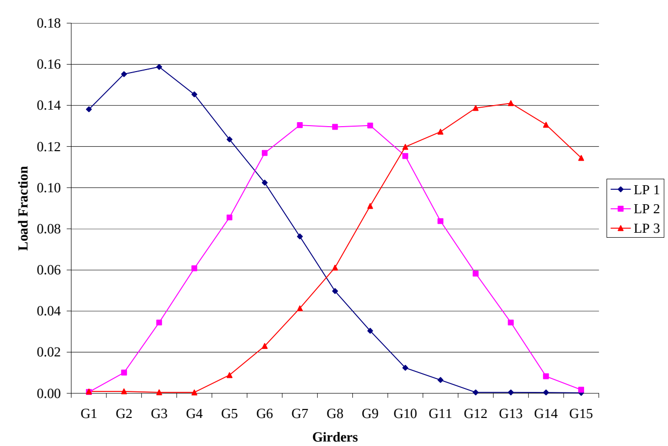

Figure 266. Load Fractions for Each Load Path

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, and S is equal to 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 23 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 267. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

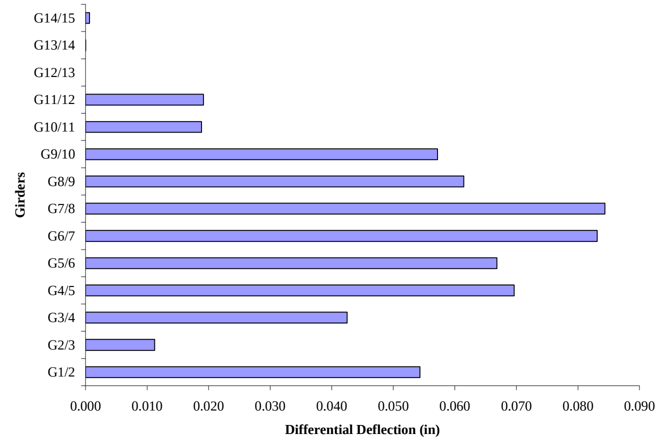

Figure 268. Differential Deflections for Load Path 1

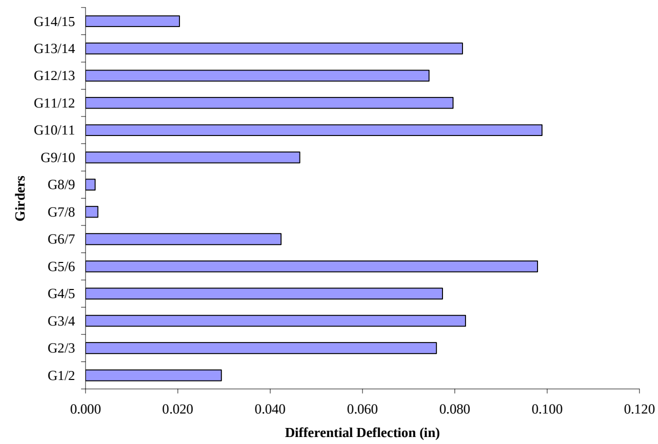

Figure 269. Differential Deflections for Load Path 2

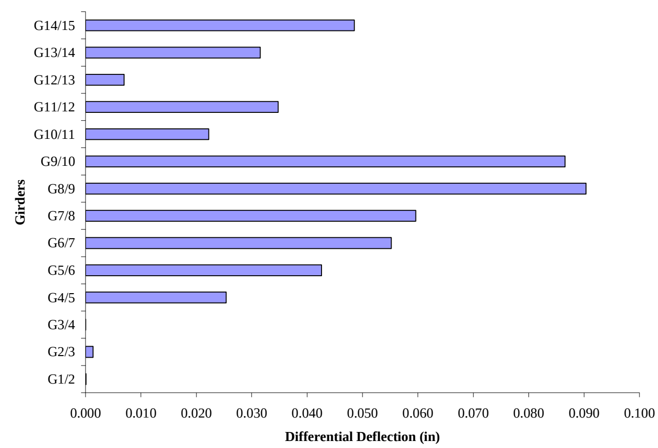

Figure 270. Differential Deflections for Load Path 3

Table 44. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.084	0.099	0.090

The maximum differential deflection of 0.099 in. occurs in load path 2 and this is nearly 25 percent 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 shown in Figures 26 through 28 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 26 through 28 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, i.e., the girders act independently of the deck when subjected to bending.

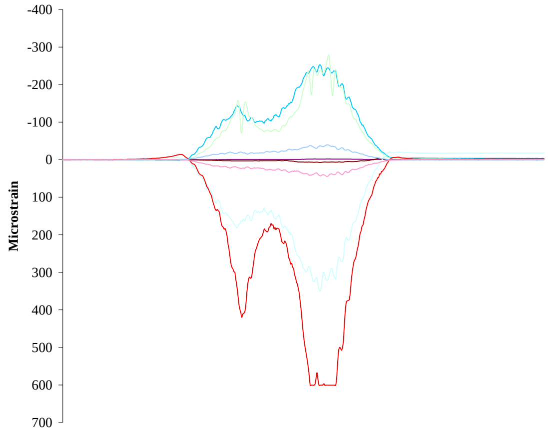

Figure 271. Strain at Midspan for Load Path 1

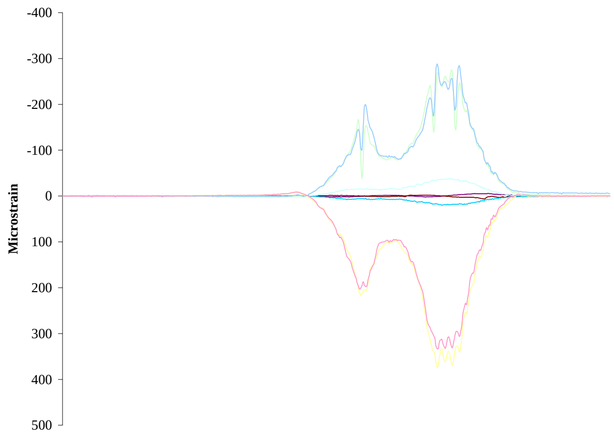

Figure 272. Strain at Midspan for Load Path 2

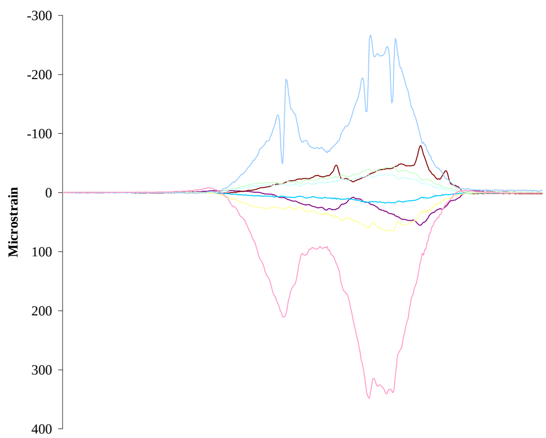

Figure 273. Strain at Midspan for Load Path 3

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 percent. The moisture content measurements are summarized in Table 5.

Table 45. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	10.6
Bottom of Deck Between Girders 7 & 8	8.6
Bottom of Deck Between Girders 14 & 15	10.4

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, 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. 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 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 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, i.e., 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 26 though 28 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 438 and 914 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 1500 psi 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.

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APPENDIX K

PERFORMANCE REPORT

COLORADO BRIDGE NO. P-19-F MINOR

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

COLORADO BRIDGE NO. P-19-F MINOR

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.

Acknowledgements

We would like to 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.

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.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Colorado Bridge.

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, (i.e., 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.

Background

The location of Colorado state bridge P-19-F Minor, hereinafter referred to as the Colorado Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 274. Colorado Bridge Location

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

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 Figure 275).

Figure 275. Bridge Substructure

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 2.

Figure 276. Colorado Bridge Parapet Support

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 in. x 18 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 3 in. x 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 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.

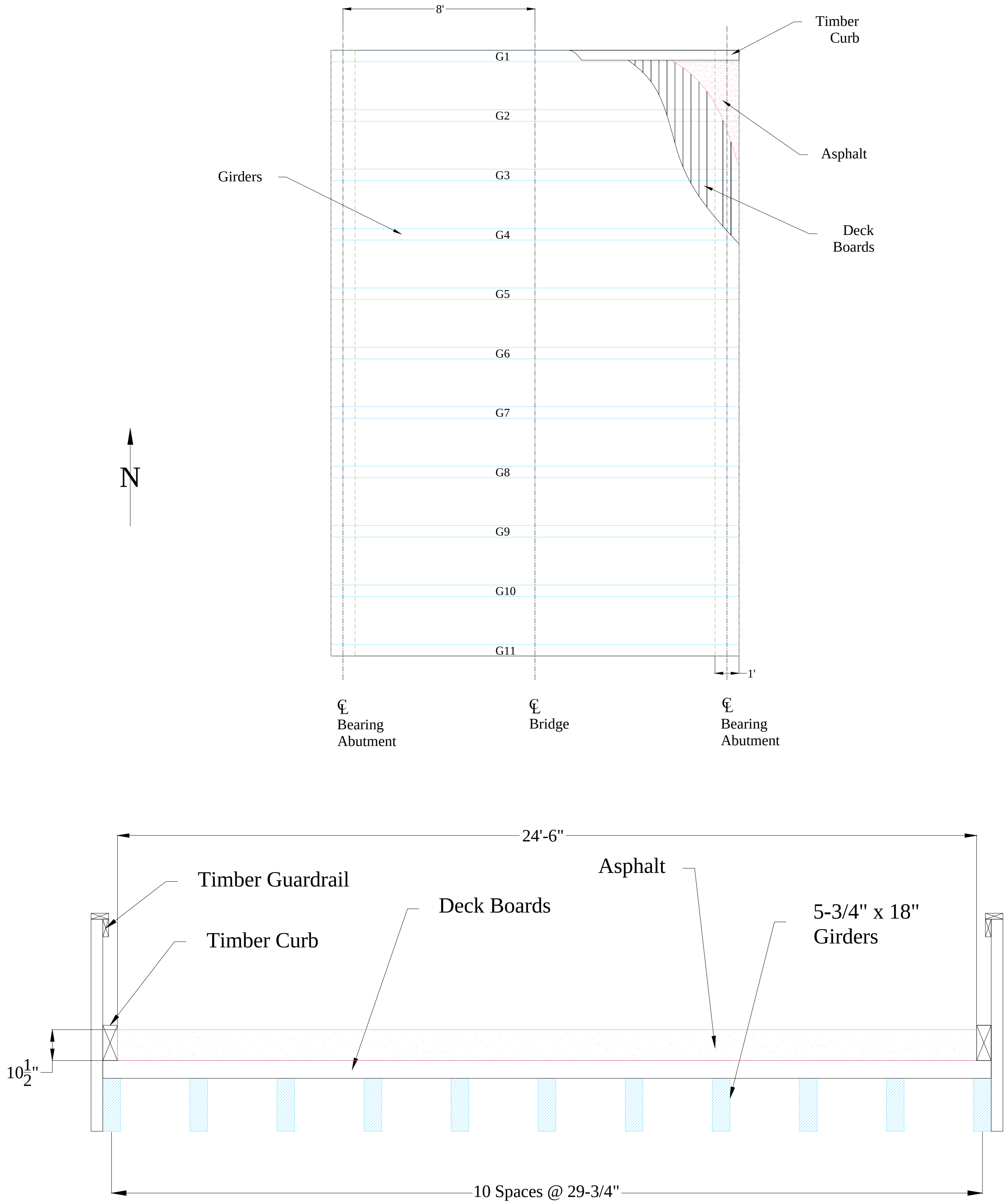
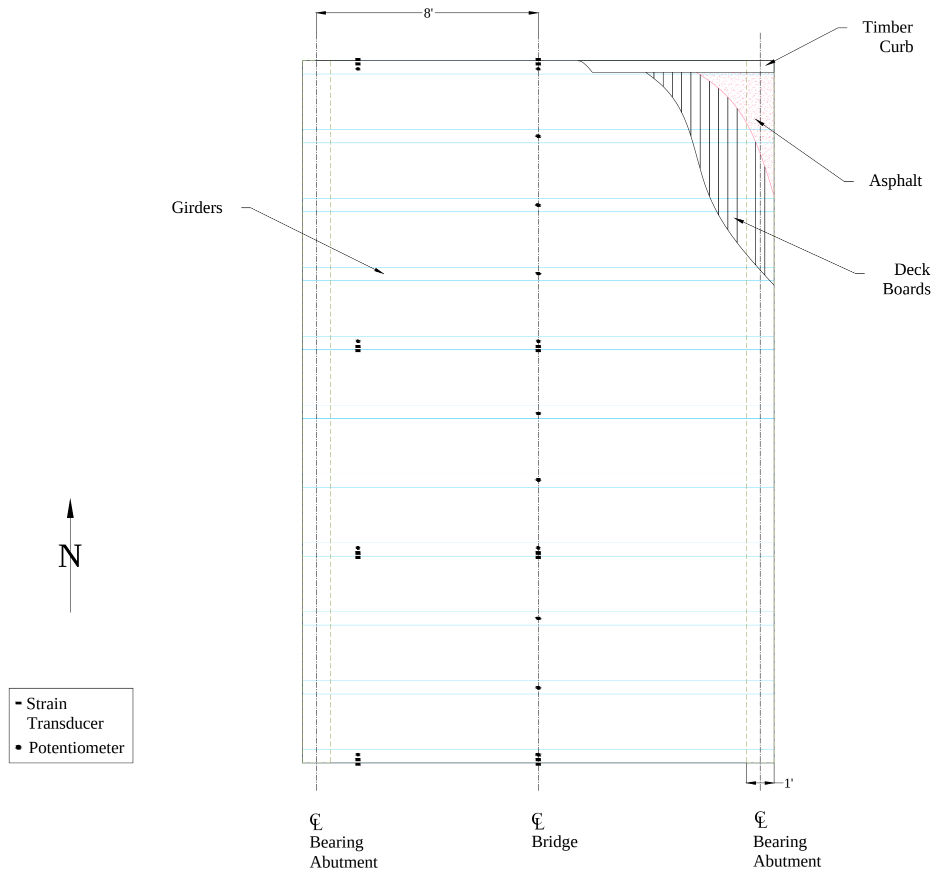


Figure 277. Plan and Profile Layout of Colorado Bridge



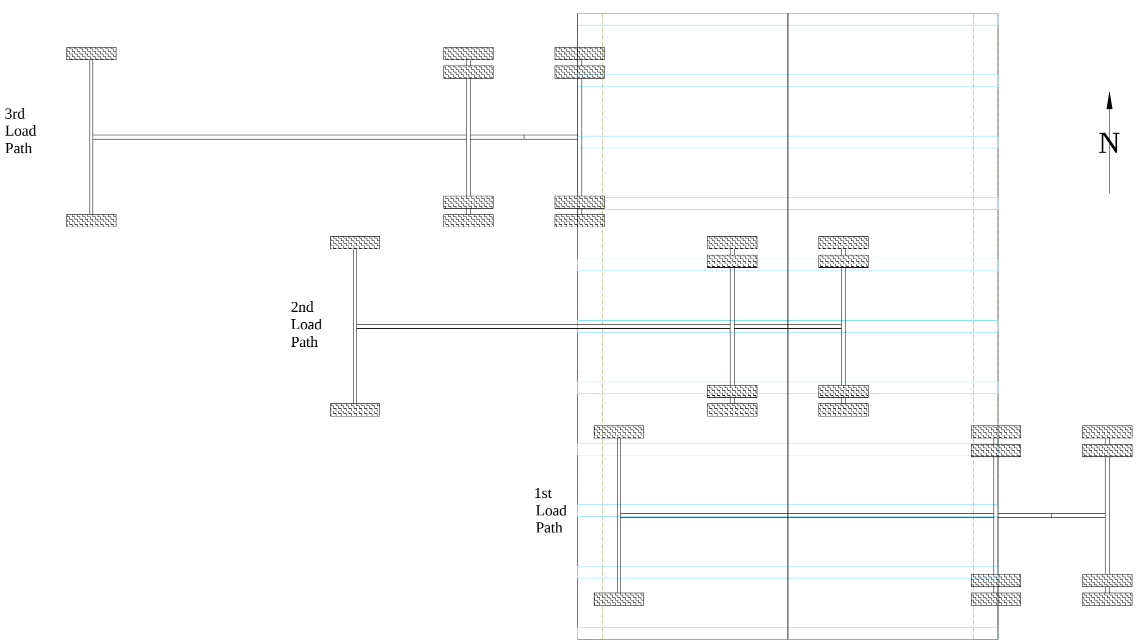


Figure 278. Instrumentation and Load Paths of Colorado Bridge

Figure 279. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near the girder ends.

Figure 280. Strain Transducers

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.

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 8. 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 lbs where the total rear weight equaled 32,620 lbs and the front axle weight was 16,500 lbs. Assuming equal weights on each rear axle, the rear axles weighed 16,810 lbs. The axle weights are shown in Figure 8 and the truck used in the load testing is shown in Figure 282.

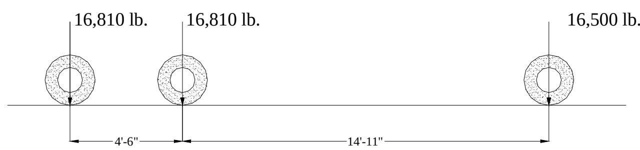


Figure 281. Truck Configuration and Axle Loads

Figure 282. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

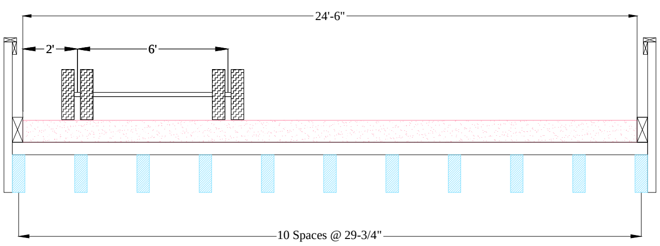


Figure 283. Transverse Truck Position - Load Path 1

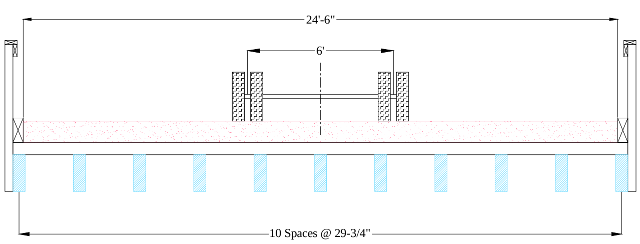


Figure 284. Transverse Truck Position - Load Path 2

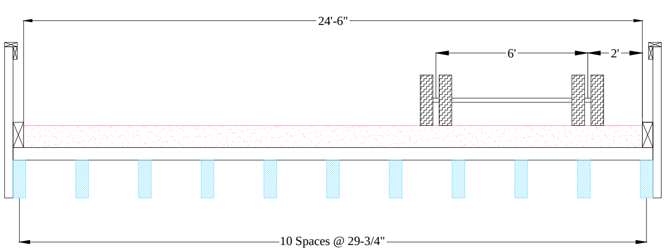


Figure 285. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 257 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.

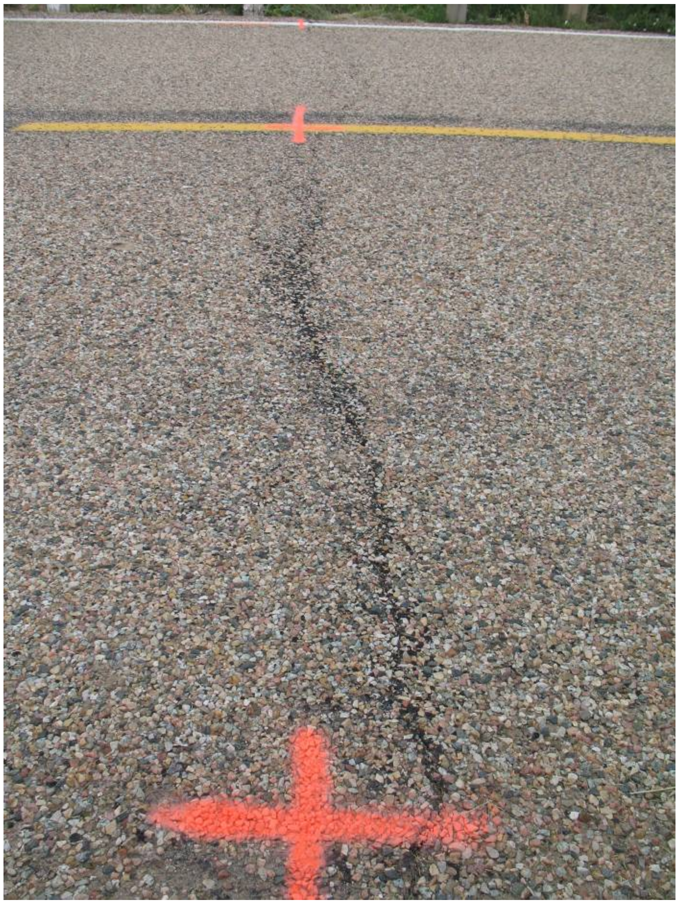


Figure 286. Transverse Wearing Surface Cracking

Deck

According to the CDOT 2004 report, white fungus was growing on the untreated bottom of the deck. This condition 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 and 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 258 shows the typical checking in the exterior girders. The girder bearing on the sill was sufficient and no misalignment was observed.

Figure 287. Checking in Exterior Girders

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 as 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 15 through 17 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

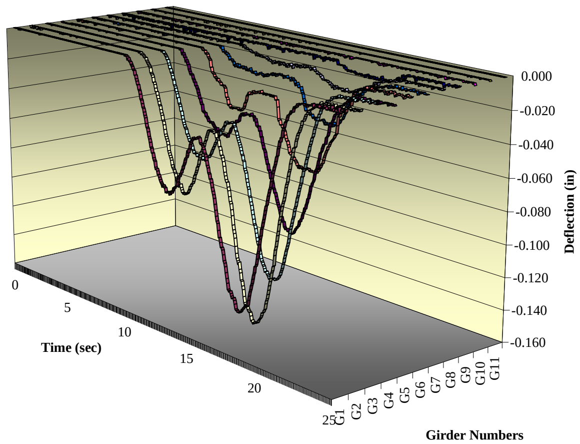


Figure 288. Deflections for Load Path 1

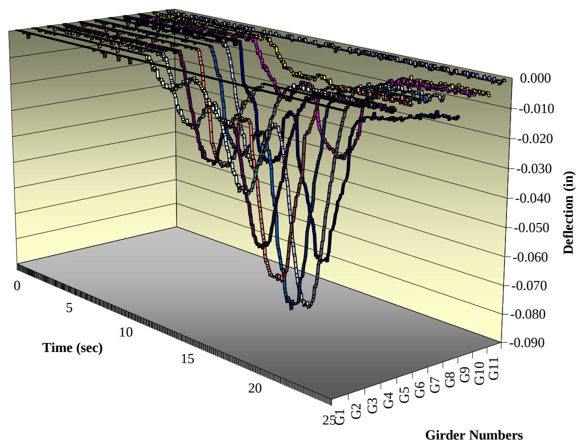


Figure 289. Deflections for Load Path 2

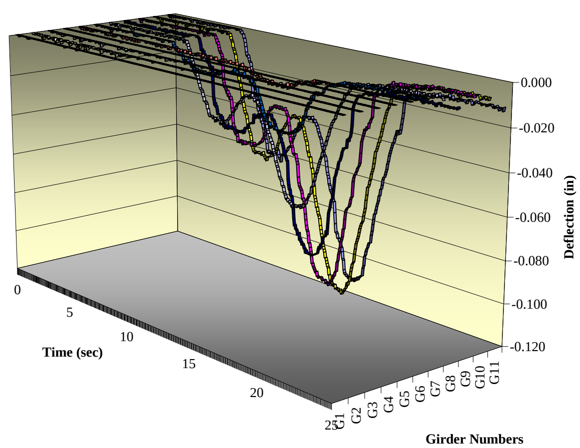


Figure 290. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 18 through 20. 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.

Table 46. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.145	0.087	0.115

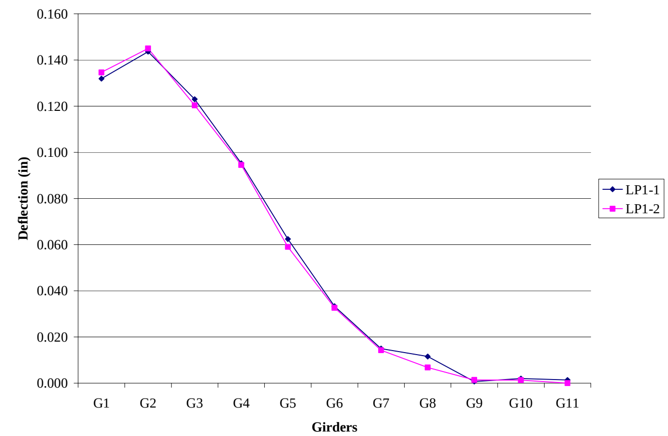


Figure 291. Maximum Deflections for Load Path 1

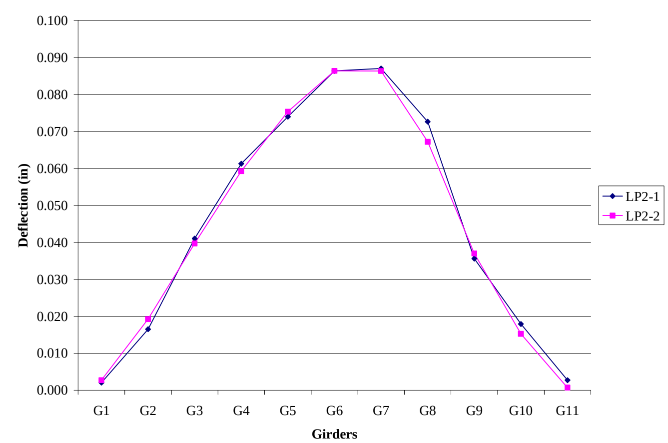


Figure 292. Maximum Deflections for Load Path 2

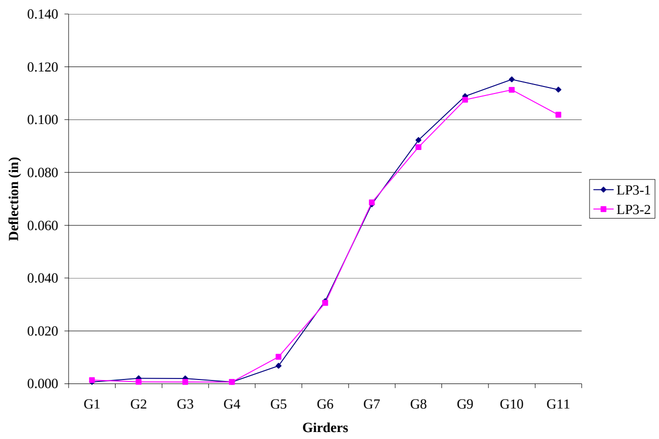


Figure 293. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 47. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 19

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 48. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
810	1350	1019

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 of the n-values are greater than the recommended n-values stated in Table 3.

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 Equation 2.

Equation 20

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

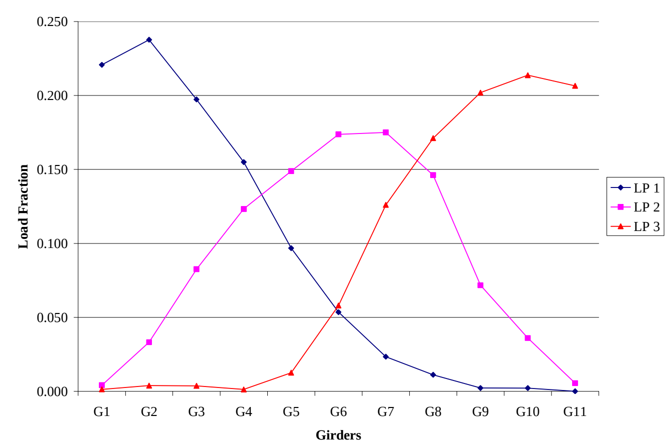


Figure 294. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

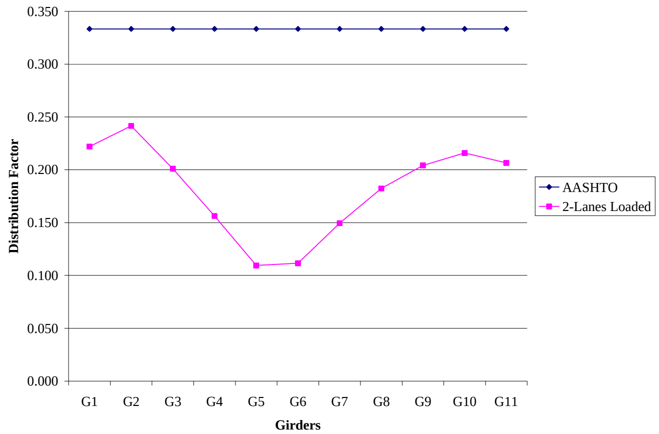


Figure 295. AASHTO Design Live Load Distribution

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. Figures 23 through 25 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 4.

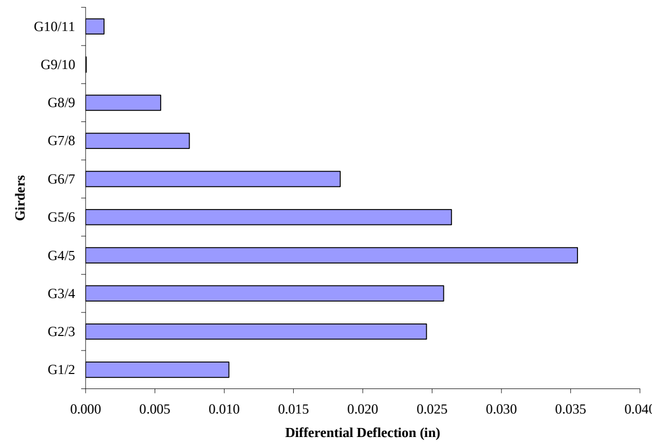


Figure 296. Differential Deflections for Load Path 1

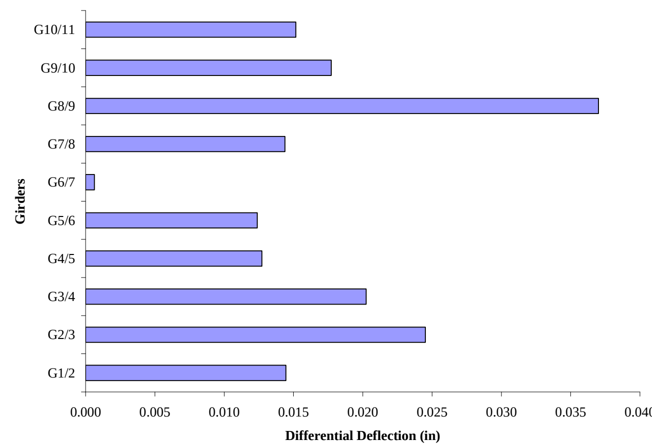


Figure 297. Differential Deflections for Load Path 2

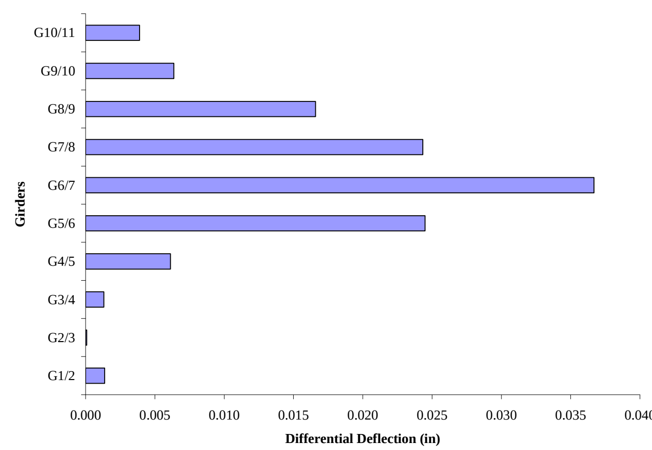


Figure 298. Differential Deflections for Load Path 3

Table 49. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.035	0.036	0.037

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 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 0.04 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 206 and 202 microstrain, respectively. The strain plot at midspan is shown in Figures 26 through 28 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 26 through 28 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, i.e., the girders act independently of the deck when subjected to bending.

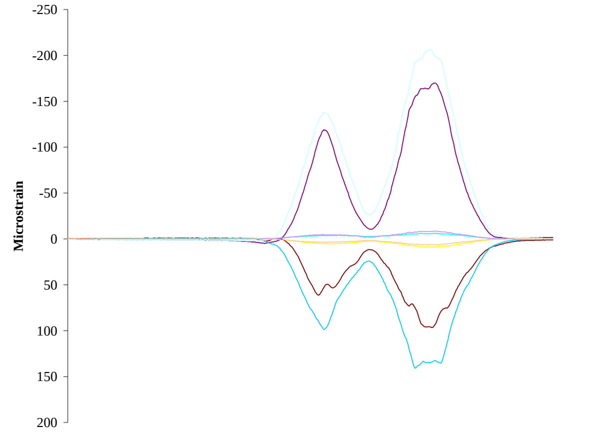


Figure 299. Strain at Midspan for Load Path 1

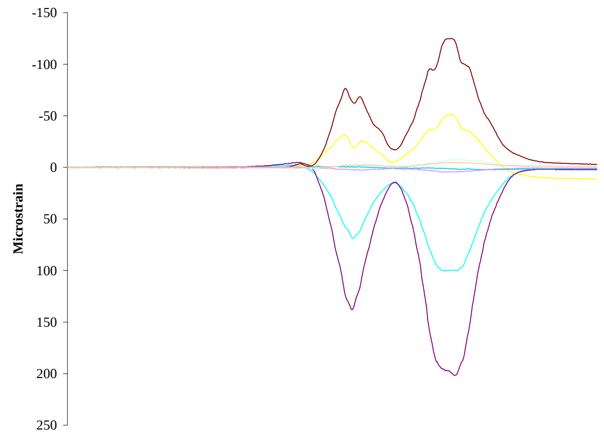


Figure 300. Strain at Midspan for Load Path 2

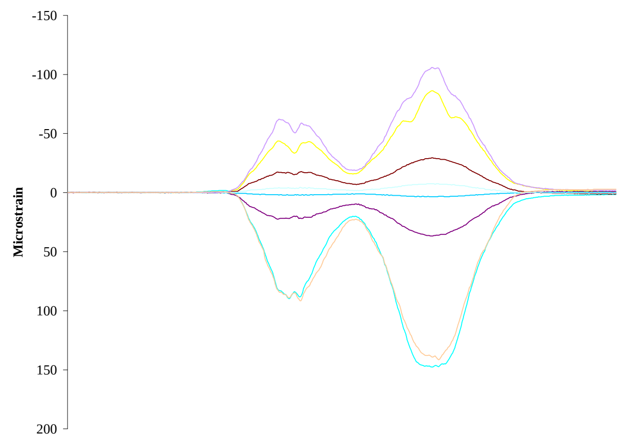


Figure 301. Strain at Midspan for Load Path 3

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 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 percent. The moisture content measurements are summarized in Table 5.

Table 50. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	19.1
Bottom of Deck Between Girders 5 & 6	25.7
Bottom of Deck Between Girders 10 & 11	18.2

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, 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. 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 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 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, i.e., 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 22 where it is seen that the girders opposite the truck path carry minimal load.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures 26 though 28 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 353 and 348 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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 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 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; 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 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 1500 psi 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.

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APPENDIX L

PERFORMANCE REPORT

COLORADO BRIDGE NO. P-19-AS

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

COLORADO BRIDGE NO. P-19-AS

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.

Acknowledgements

We would like to 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.

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.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Colorado Bridge.

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, (i.e., 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.

Background

The location of Colorado state bridge P-19-AS, hereinafter referred to as the Colorado Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 302. Colorado Bridge Location

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.

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 back-wall. 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 Figure 303).

Figure 303. Bridge Substructure

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 2.

Figure 304. Colorado Bridge Parapet Support

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. center-to-center, measuring 6 in. x 18 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 3 in. x 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 4 illustrates the layout of the bridge.

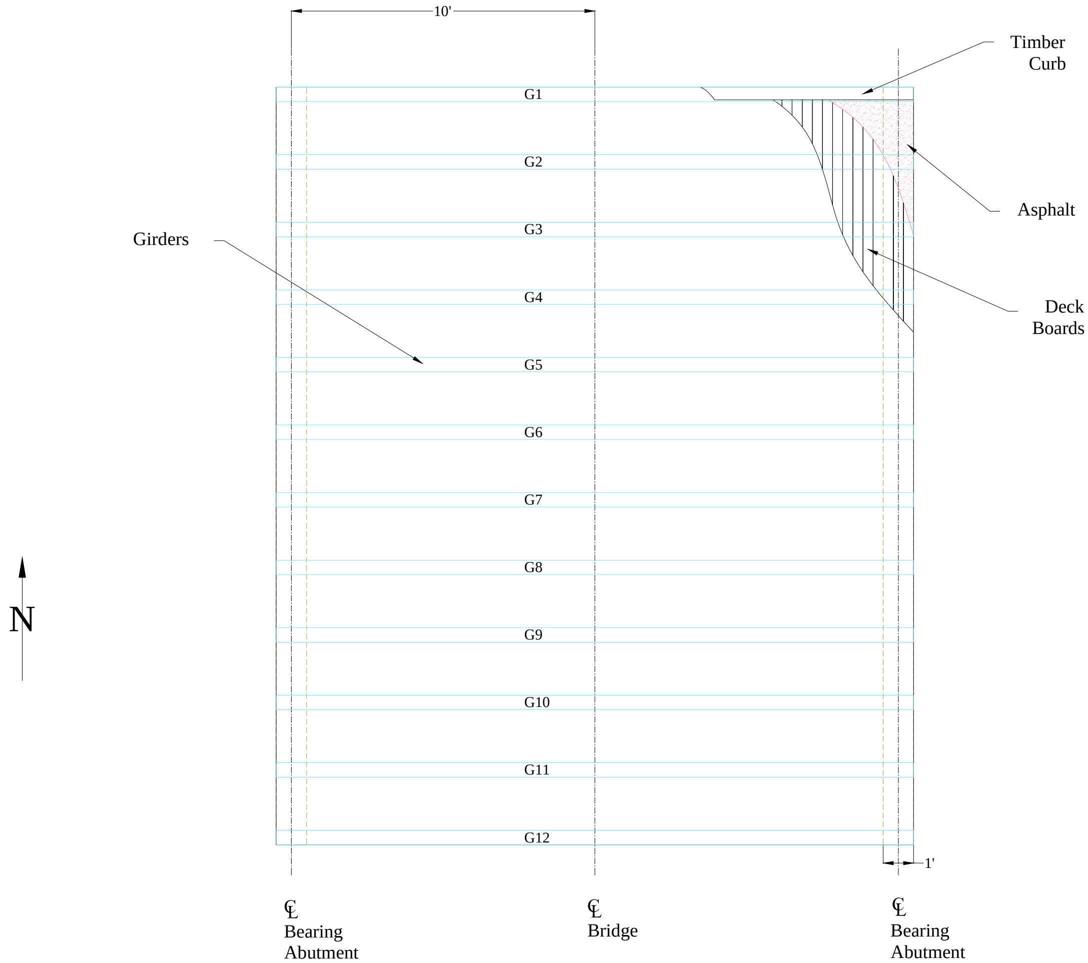
Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.



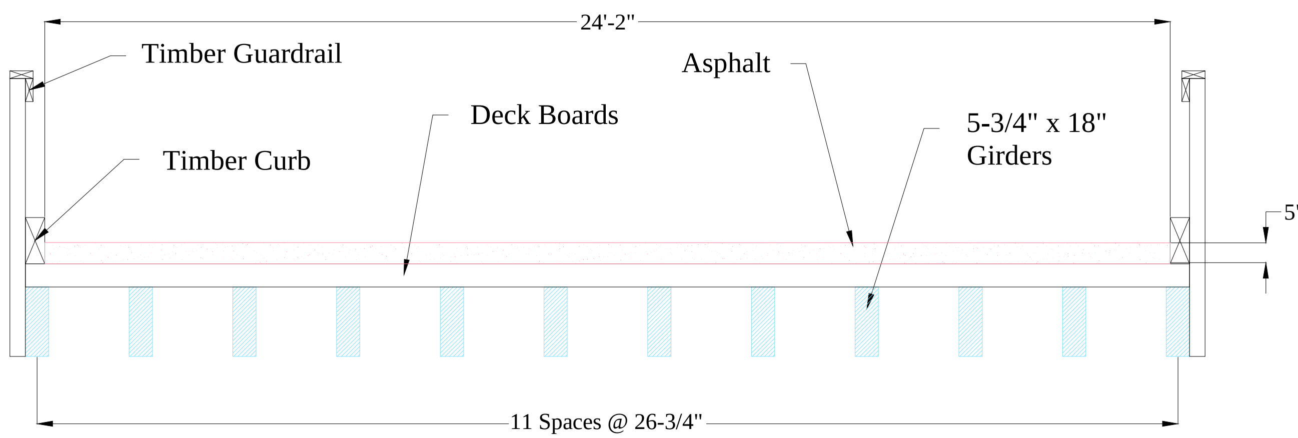


Figure 305. Plan and Profile Layout of Colorado Bridge

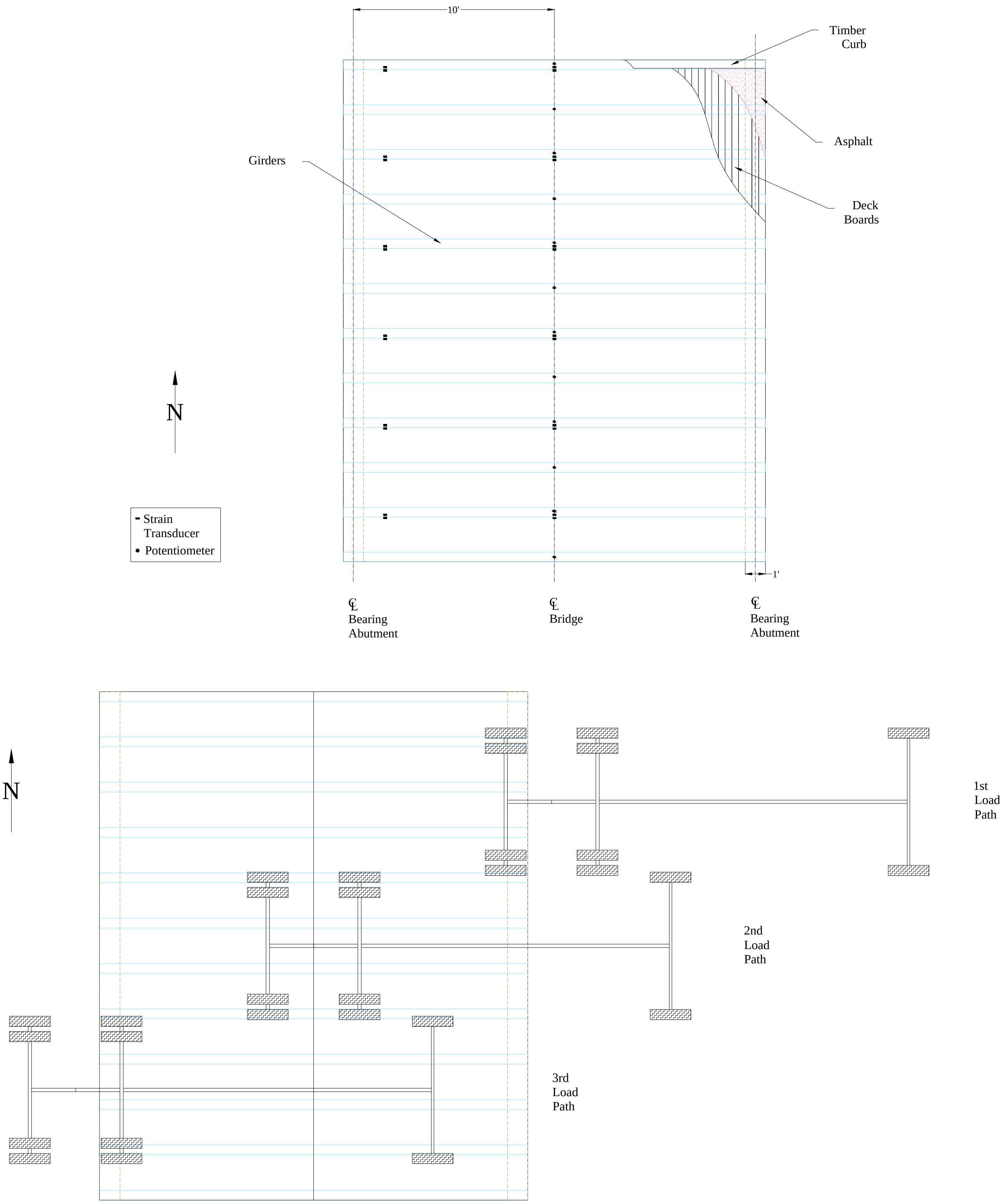


Figure 306. Instrumentation and Load Paths of Colorado Bridge

Figure 307. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near the girder ends.

Figure 308. Strain Transducers

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.

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 8. 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 lbs where the total rear weight equaled 32,620 lbs and the front axle weight was 16,500 lbs. Assuming equal weights on each rear axle, the rear axles weighed 16,810 lbs. The axle weights are shown in Figure 8. The truck used for the load testing is shown in Figure 310.

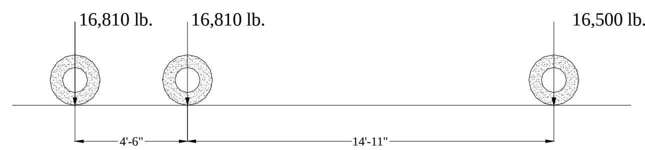


Figure 309. Truck Configuration and Axle Loads

Figure 310. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 and 12). 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.

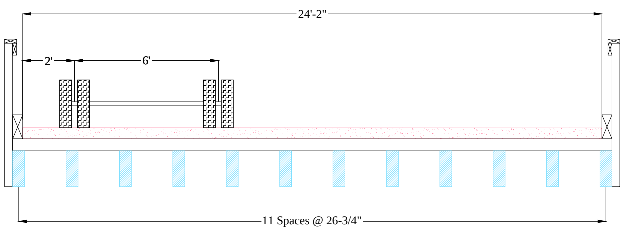


Figure 311. Transverse Truck Position - Load Path 1

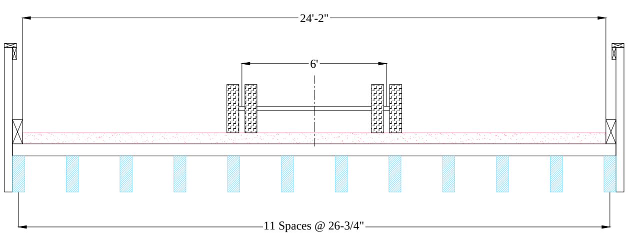


Figure 312. Transverse Truck Position - Load Path 2

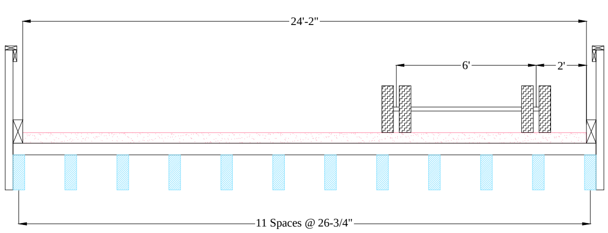


Figure 313. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the 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.

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 257 shows some of the cracking observed. Aside from the transverse cracking, the wearing surface looked to be in good condition.

Figure 314. Transverse Wearing Surface Cracking

Figure 315. 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 and 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 258 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. 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 as 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 15 through 17 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

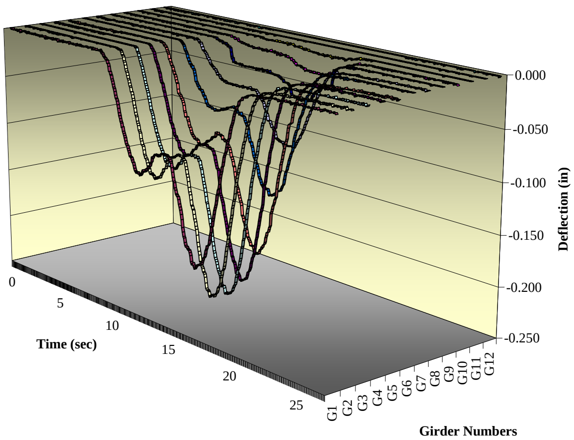


Figure 316. Deflections for Load Path 1

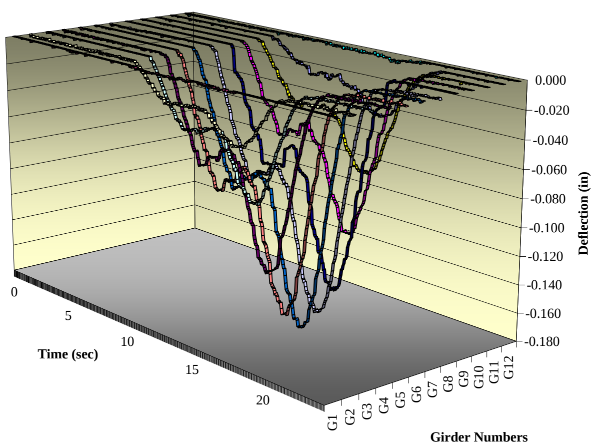


Figure 317. Deflections for Load Path 2

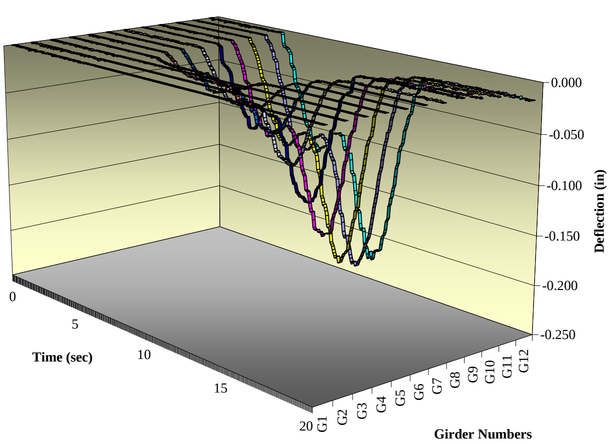


Figure 318. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 18 through 20. 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.

Table 51. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.222	0.180	0.224

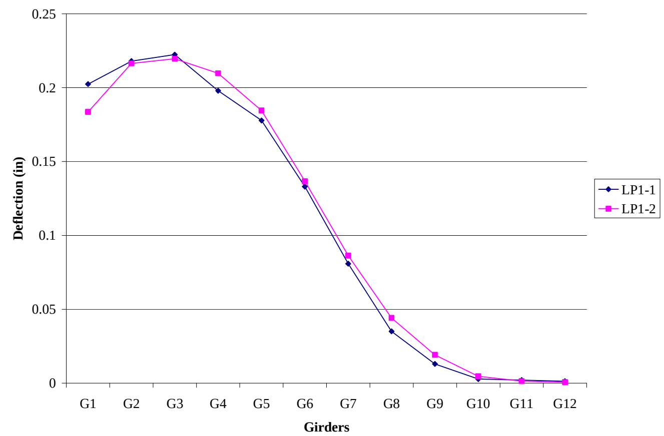


Figure 319. Maximum Deflections for Load Path 1

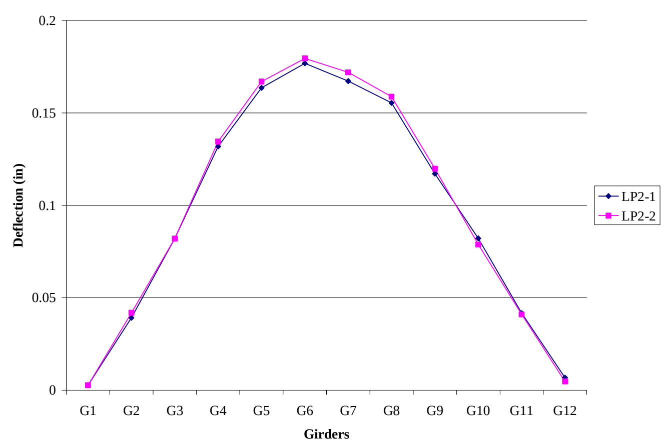


Figure 320. Maximum Deflections for Load Path 2

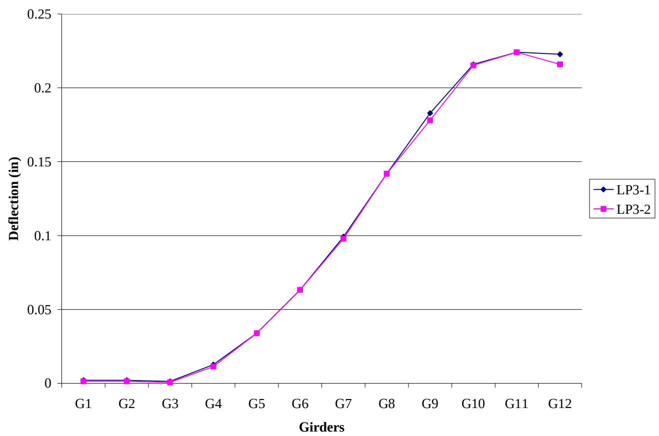


Figure 321. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 52. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 21

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 53. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
669	829	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 of the n-values are greater than the recommended n-values stated in Table 3.

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 Equation 2.

Equation 22

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i = load fraction of the i^{th} girder
Δi = deflection of the i^{th} girder
$\Sigma\Delta_i$ = sum of all girder deflections
n = number of girders

Figure 22 shows the load fractions for each girder for each load path.

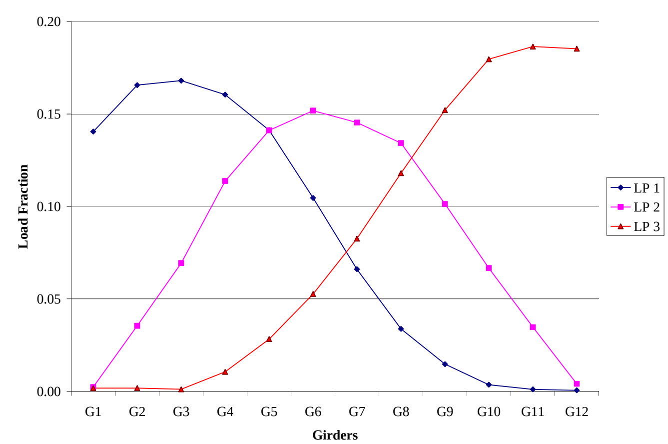


Figure 322. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

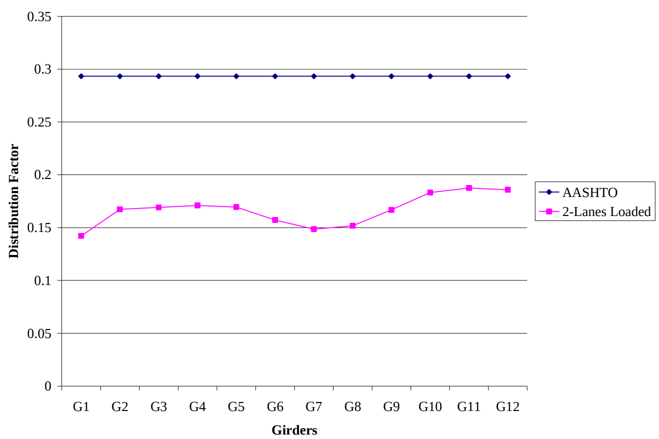


Figure 323. AASHTO Design Live Load Distribution

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. Figures 23 through 25 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 4.

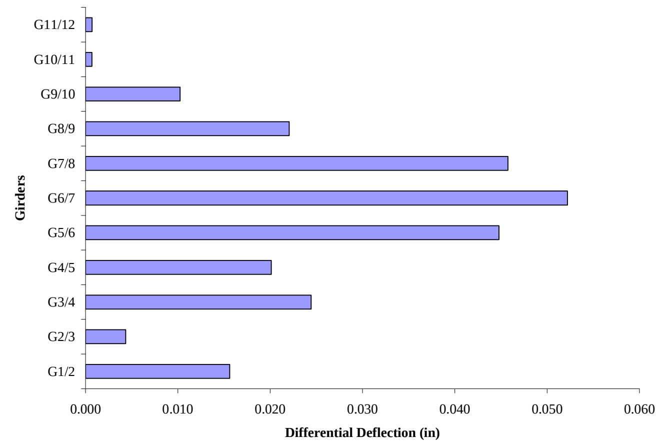


Figure 324. Differential Deflections for Load Path 1

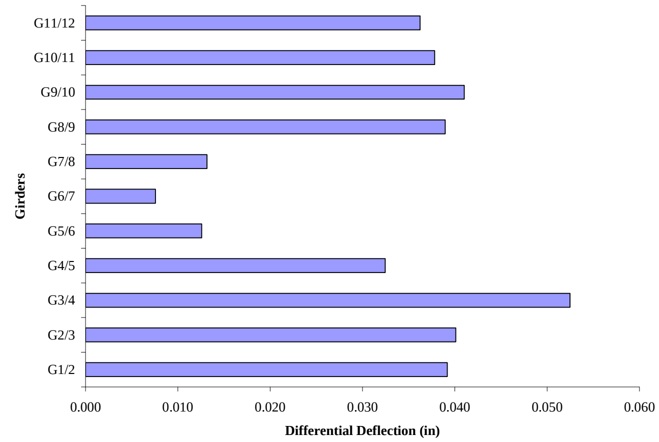


Figure 325. Differential Deflections for Load Path 2

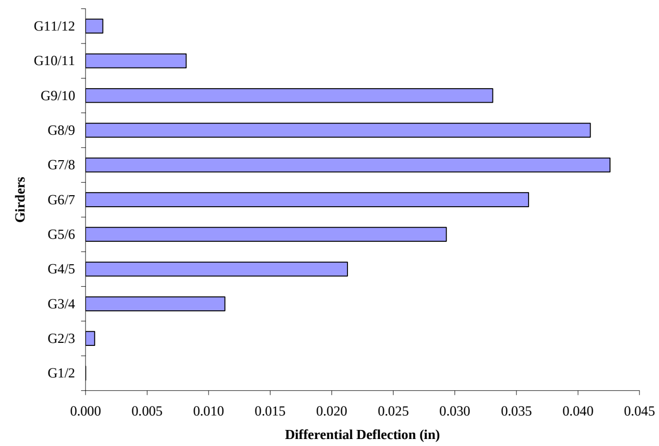


Figure 326. Differential Deflections for Load Path 3

Table 54. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.052	0.052	0.043

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 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 26 through 28 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 26 through 28 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, i.e., the girders act independently of the deck when subjected to bending.

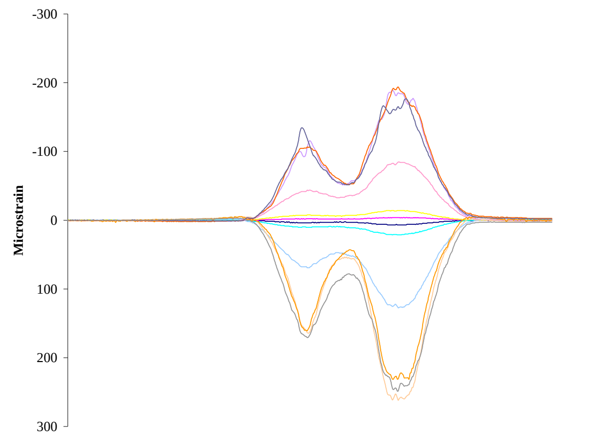


Figure 327. Strain at Midspan for Load Path 1

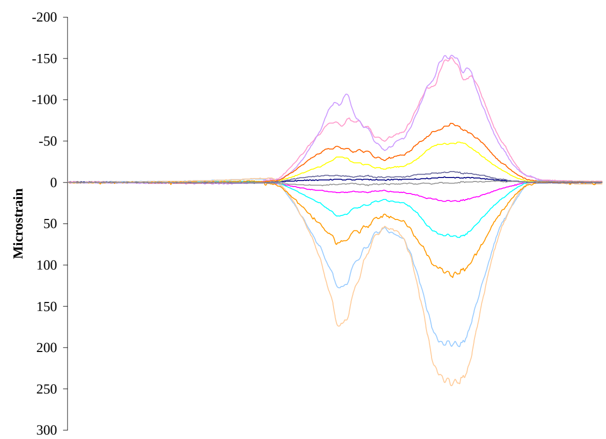


Figure 328. Strain at Midspan for Load Path 2

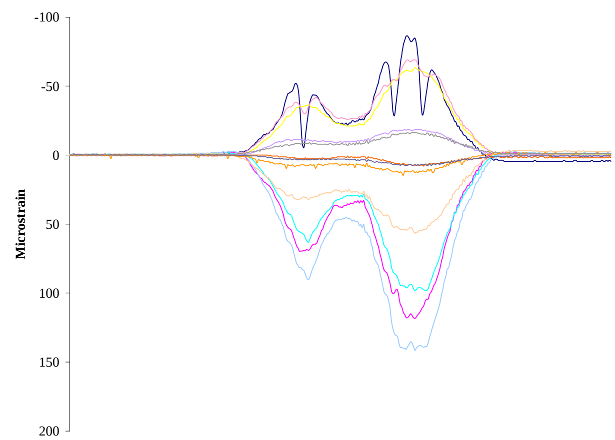


Figure 329. Strain at Midspan for Load Path 3

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, 6, and 12 at the 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 percent. The moisture content measurements are summarized in Table 5.

Table 55. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	25.4
Bottom of Deck Between Girders 5 & 6	24.1
Bottom of Deck Between Girders 11 & 12	24.1

Finite Element Analysis

A finite element model was developed (see Figure 89) 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.

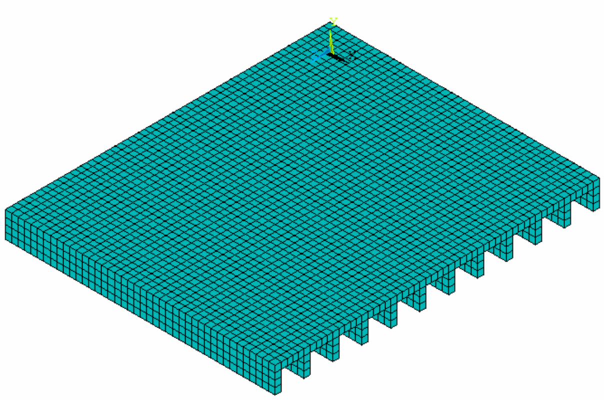


Figure 330. Finite Element Model

Figures 31 through 33 show the calibrated model results when subjected to the same load as that during the static load test. Notice the similarities between each plot.

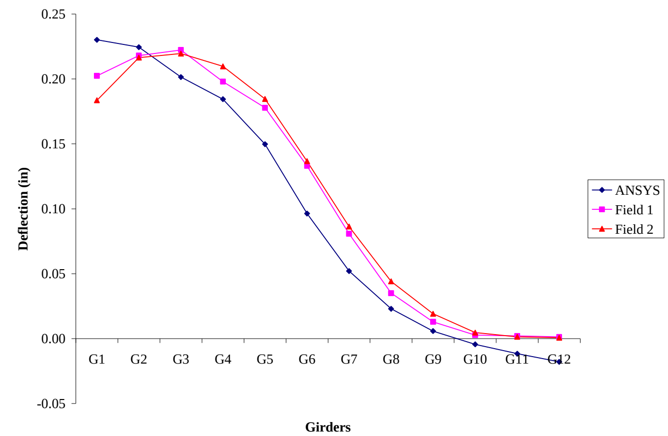


Figure 331. ANSYS Calibration Results Load Path 1

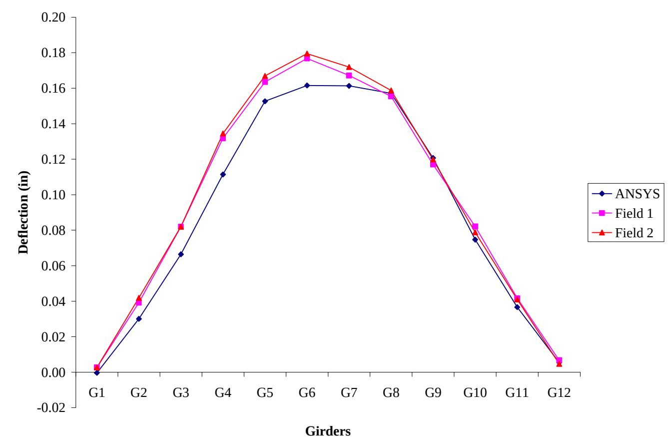


Figure 332. ANSYS Calibration Results Load Path 2

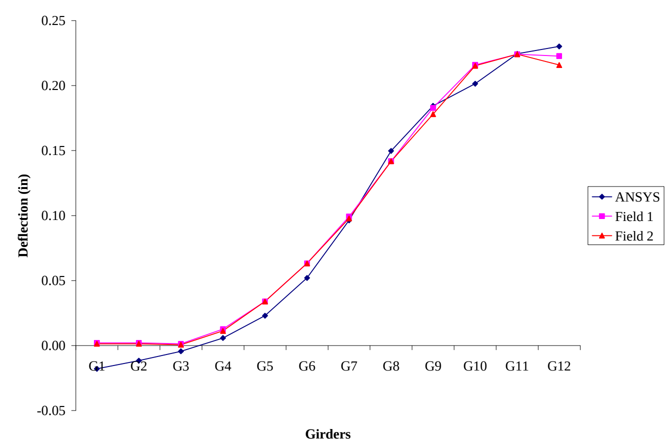


Figure 333. ANSYS Calibration Results Load Path 3

Figure 93 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.

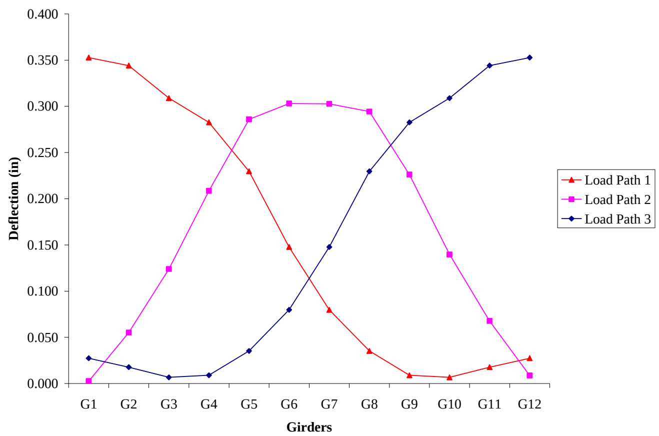


Figure 334. ANSYS Deflection Results for Each Load Path when Subjected to HS20 Tandem Axle Design Vehicle

Figure 35 shows the maximum tensile stresses at midspan due to the AASHTO HS20 tandem axle design vehicle traveled along each load path.

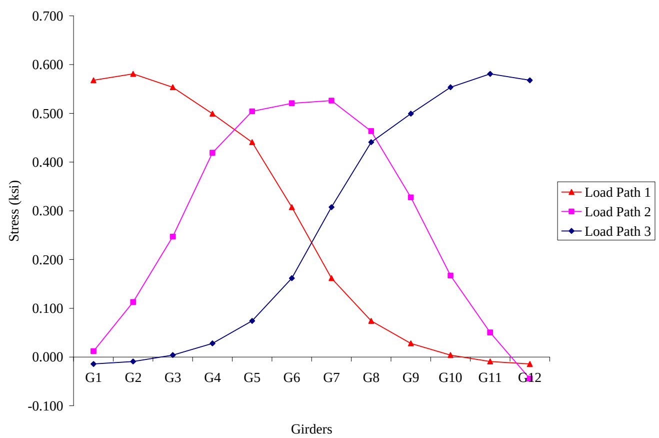


Figure 335. ANSYS Tensile Stress for Each Load Path when Subjected to HS20 Tandem Axle Design Vehicle

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, 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

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 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, i.e., 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 22 where it is seen that the girders opposite the truck path carry minimal load.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures 26 though 28 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 281 and 389 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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 2 were 581 and 526 psi, 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; 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 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 1500 psi 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.

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APPENDIX M

PERFORMANCE REPORT

MONTANA BRIDGE NO. P00009040+01001

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

MONTANA BRIDGE NO. P00009040+01001

Terry Wipf
Brent Phares
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Doug Wood
Michael Ritter
Justin Dahlberg

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 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.

Acknowledgements

We would like to 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.

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.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Montana Bridge.

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, (i.e., 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.

Background

The location of Montana state bridge P00009040+01001, hereinafter referred to as the Montana Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 336. Montana Bridge Location

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

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 Figure 337).

Figure 337. Bridge Substructure

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 2.

Figure 338. 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 in. x 16-3/4 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 2 in. x 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 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.

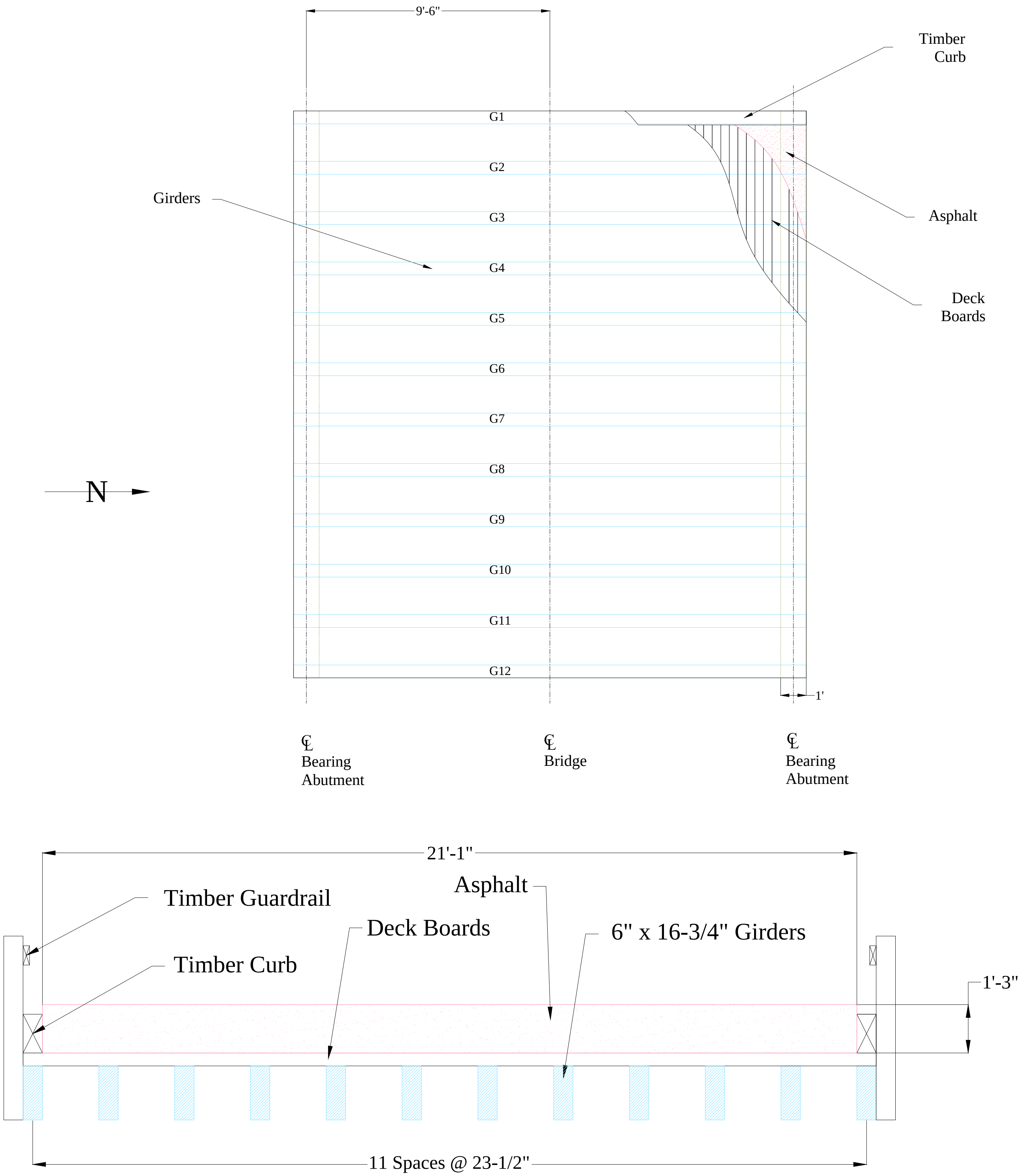

Figure 339. Plan and Profile Layout of Montana Bridge

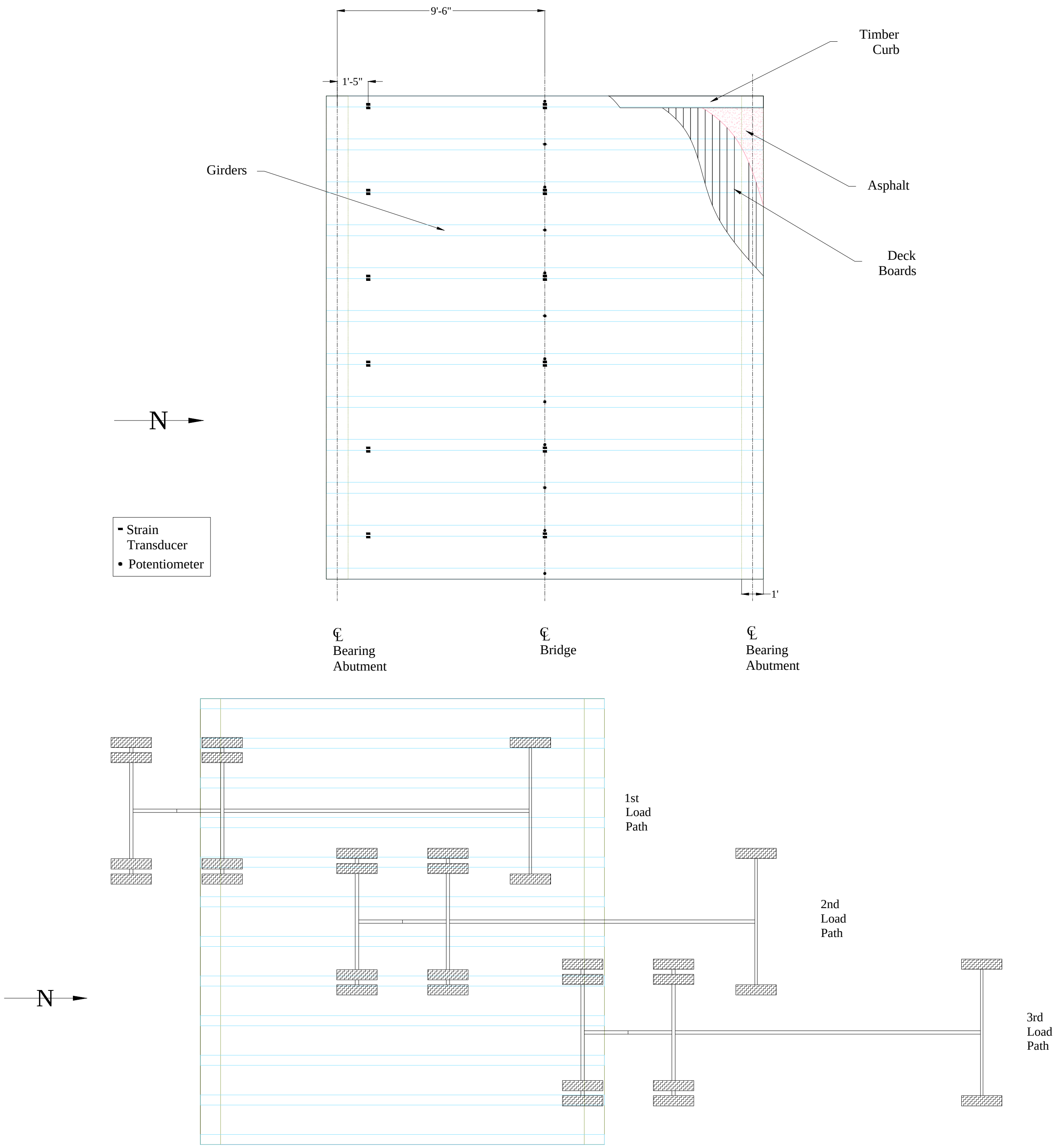

Figure 340. Instrumentation and Load Paths of Montana Bridge

Figure 341. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near the girder ends.

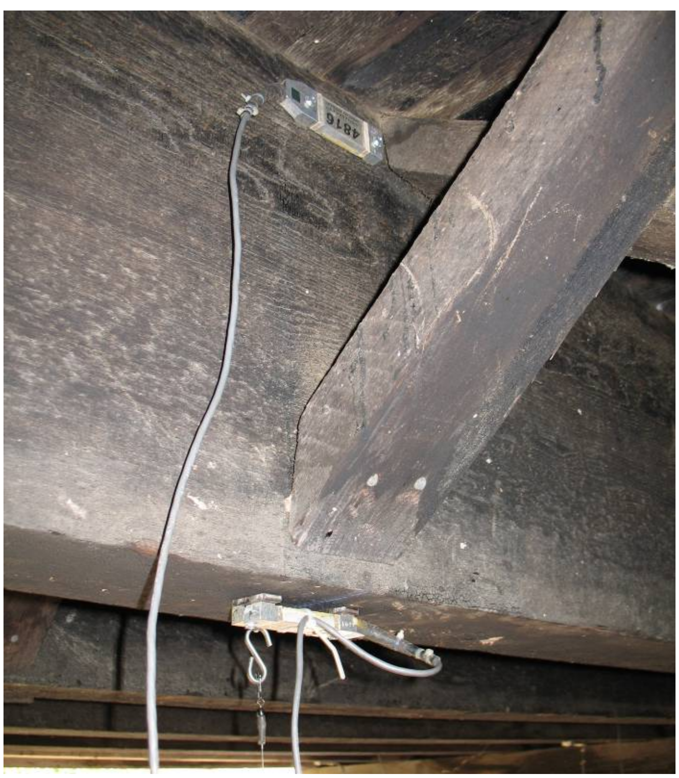

Figure 342. Strain Transducers

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.

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 8. The rear wheel base was 6 ft-0in.; 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 lbs where the total rear weight equaled 28,080 lbs and the front axle weight was 10,940 lbs. Assuming equal weights on each rear axle, the rear axles weighed 14,040 lbs. The axle weights are shown in Figure 8 and the load truck in shown in Figure 344.

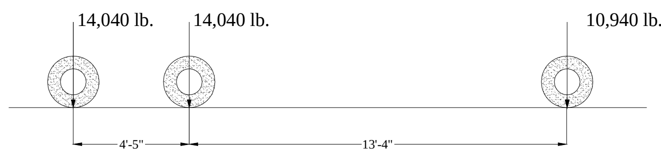

Figure 343. Truck Configuration and Axle Loads

Figure 344. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

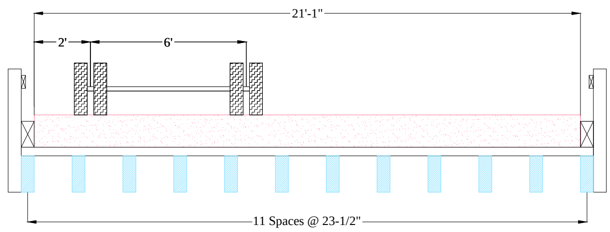

Figure 345. Transverse Truck Position - Load Path 1

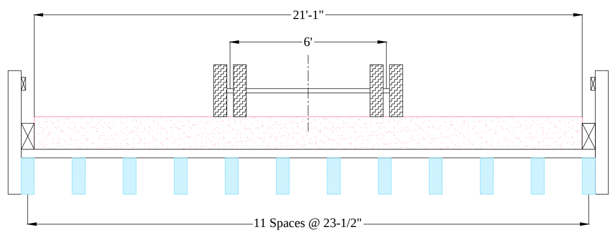

Figure 346. Transverse Truck Position - Load Path 2

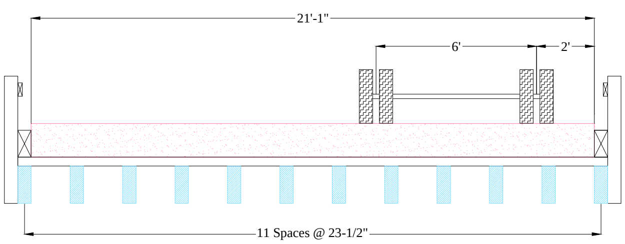

Figure 347. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the ISU research team. In particular, the wearing surface, deck, and superstructure were thoroughly assessed. In addition, the substructure was viewed, though the ISU team was primarily concerned with the superstructure. 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.

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 257 as the wearing surface is shown above the curb.

Figure 348. Wearing Surface Thickness

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 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 Figure 349). 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.

Figure 349. White Residue Formation on Deck

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 Figure 258). 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. The girder bearing on the sill was sufficient and no misalignment was observed.

Figure 350. Checking in Exterior Girders

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 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 16 through 18 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

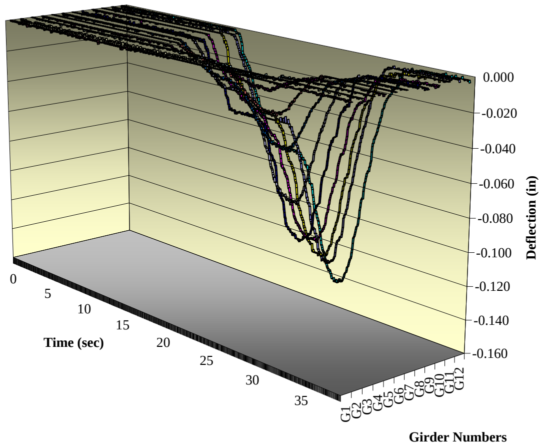

Figure 351. Deflections for Load Path 1

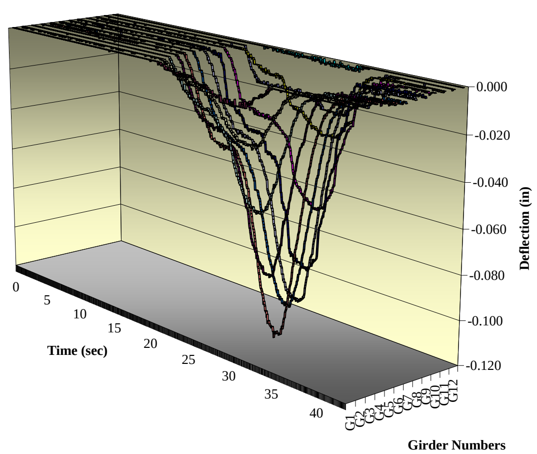

Figure 352. Deflections for Load Path 2

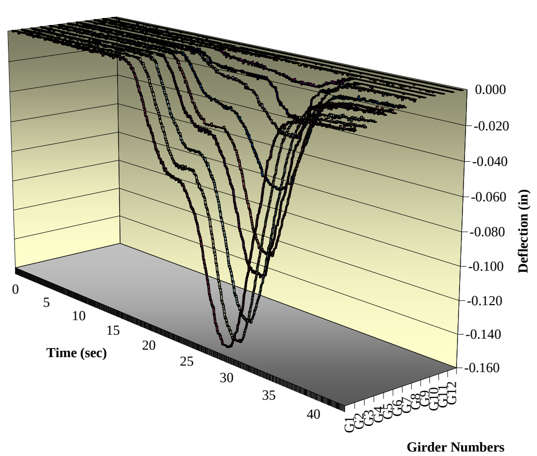

Figure 353. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 18 through 20. 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.

Table 56. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.156	0.118	0.143

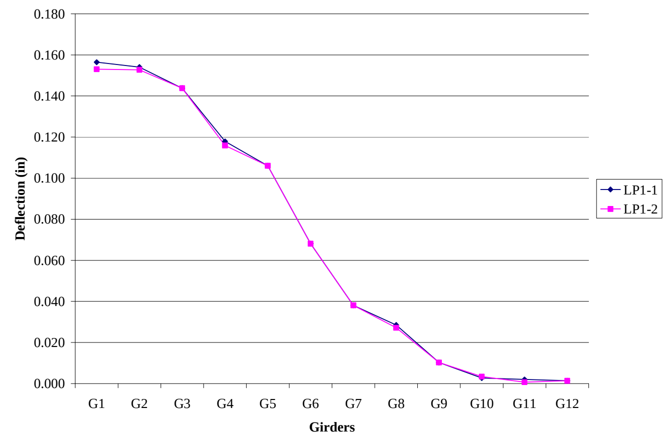

Figure 354. Maximum Deflections for Load Path 1

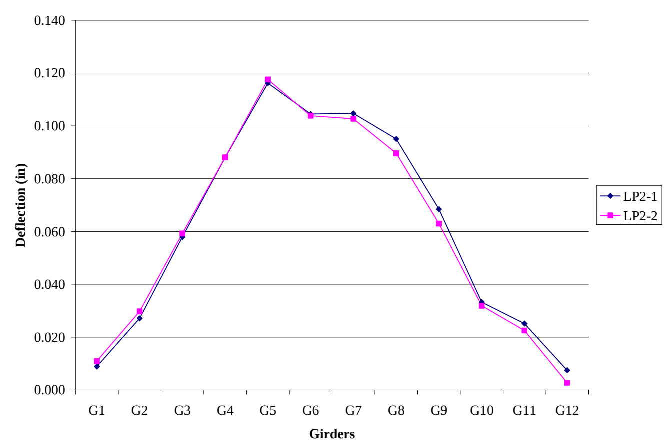

Figure 355. Maximum Deflections for Load Path 2

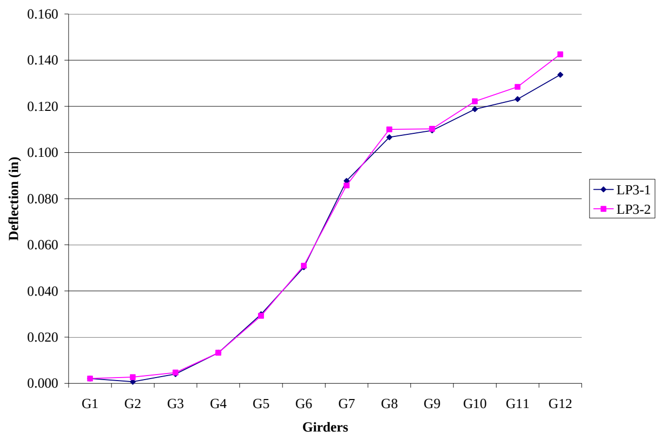

Figure 356. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 57. Live Load Deflection Limit States

Source	**n-Value**
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 23

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 58. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
400	496	475

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 3.

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 Equation 2.

Equation 24

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

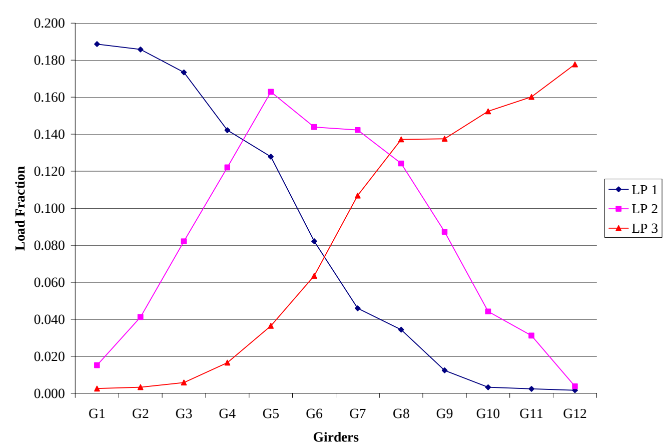

Figure 357. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

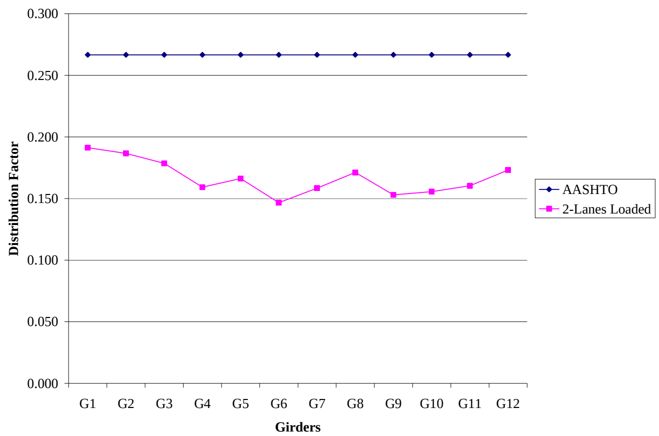

Figure 358. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

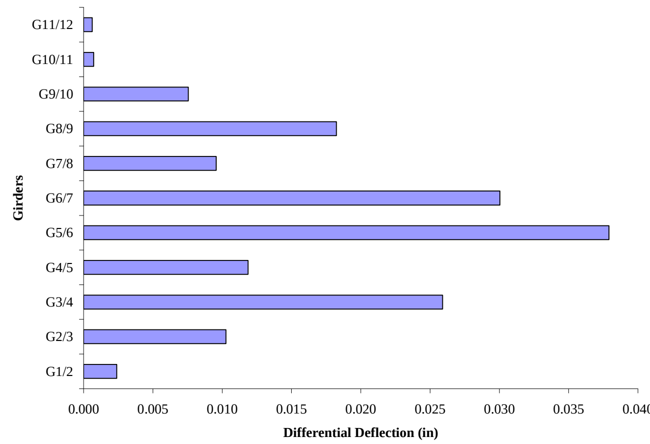

Figure 359. Differential Deflections for Load Path 1

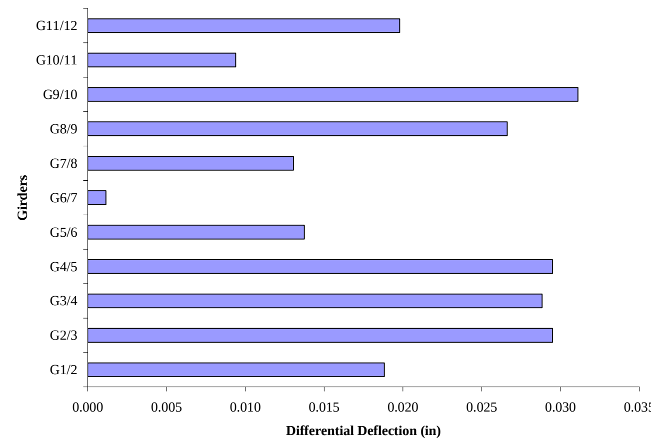

Figure 360. Differential Deflections for Load Path 2

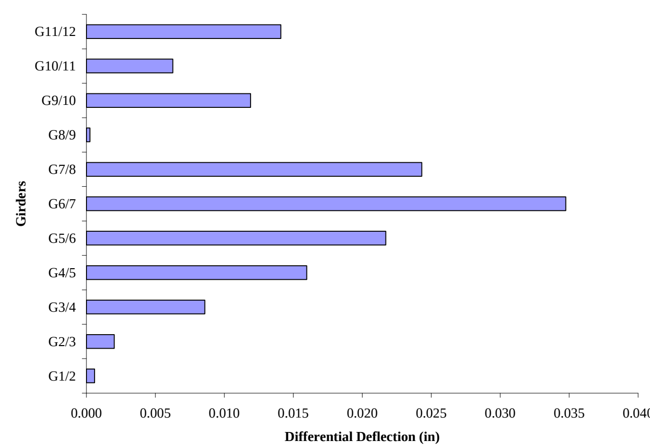

Figure 361. Differential Deflections for Load Path 3

Table 59. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.038	0.031	0.035

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 27 through 29 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 through 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, i.e., the girders act independently of the deck when subjected to bending.

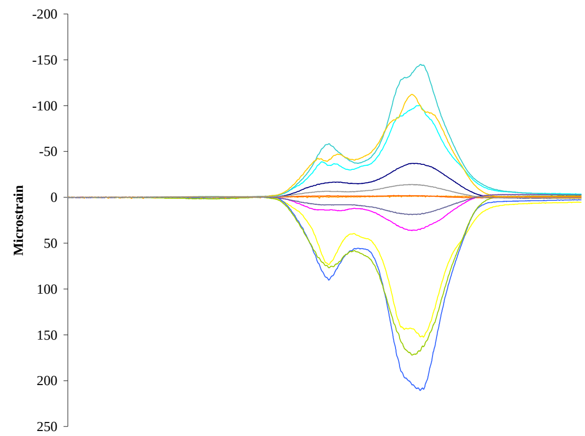

Figure 362. Strain at Midspan for Load Path 1

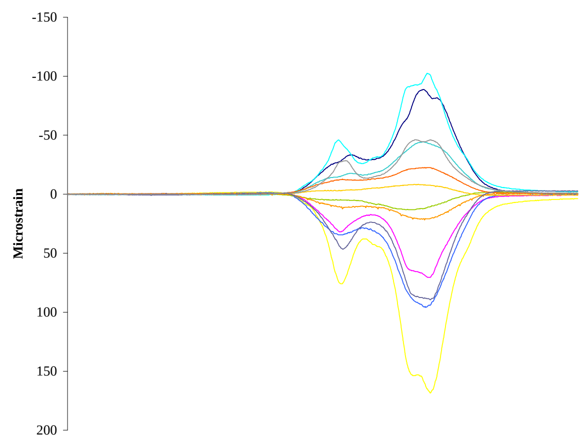

Figure 363. Strain at Midspan for Load Path 2

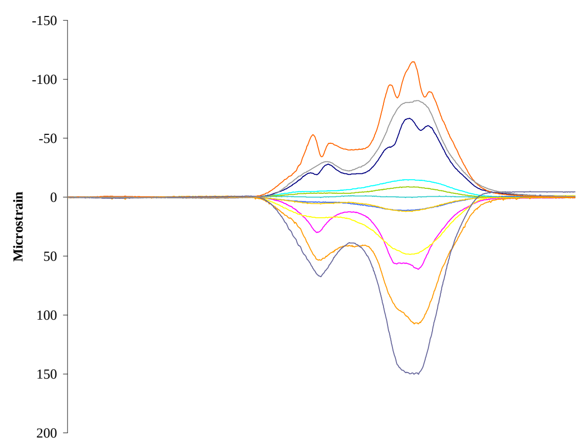

Figure 364. Strain at Midspan for Load Path 3

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, 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 percent. The moisture content measurements are summarized in Table 5.

Table 60. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	15.0
Bottom of Deck Between Girders 6 & 7	11.6
Bottom of Deck Between Girders 11 & 12	12.0

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, 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. 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, i.e., 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 27 though 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 145 and 211 microstrain, respectively. These strains equate to maximum stresses of 167 and 243 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 297 and 433 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 1500 psi 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.

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APPENDIX N

PERFORMANCE REPORT

MONTANA BRIDGE NO. L25003009+09001

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

MONTANA BRIDGE NO. L25003009+09001

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 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.

Acknowledgements

We would like to 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.

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.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Montana Bridge.

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, (i.e., 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.

Background

The location of Montana state bridge L25003009+09001, hereinafter referred to as the Montana Bridge, is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 365. Montana Bridge Location

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

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 oustside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Figure 366).

Figure 366. Bridge Substructure

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 2.

Figure 367. Montana Bridge Parapet Support

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. center-to-center, measuring 6 in. x 17 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 2 in. x 4 in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck is a 11 in. thick layer of asphalt wearing surface. Figure 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.

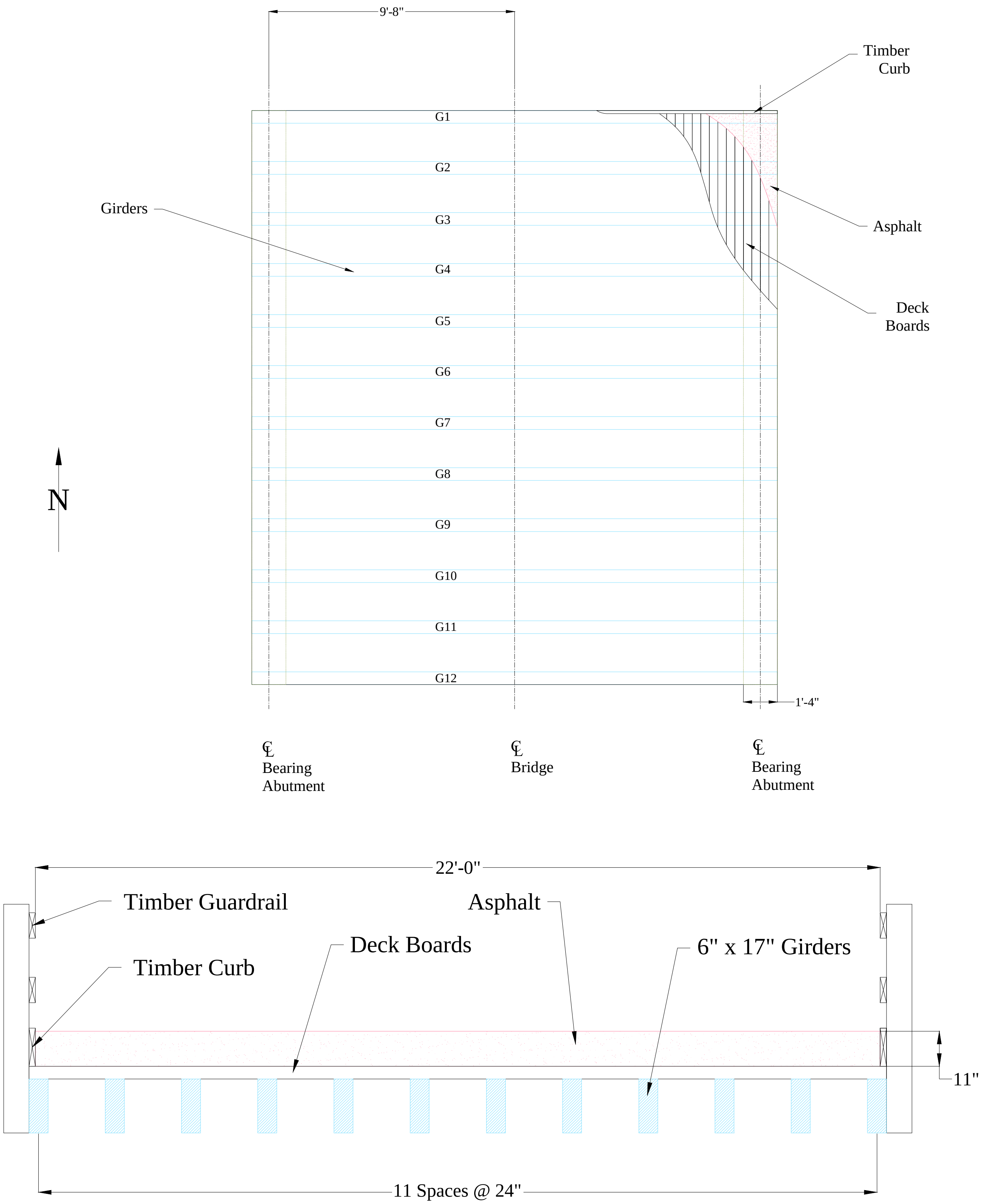

Figure 368. Plan and Profile Layout of Montana Bridge

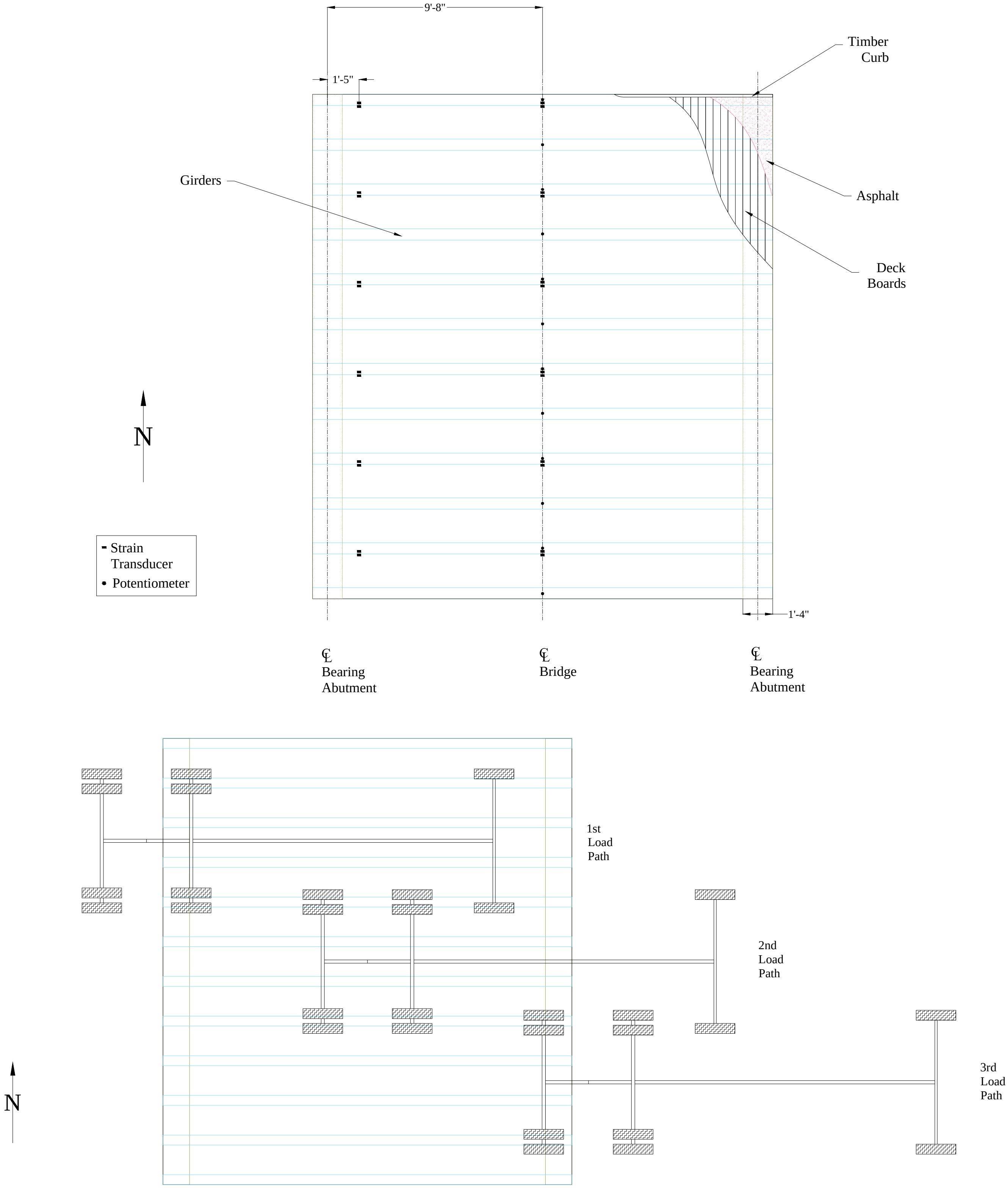

Figure 369. Instrumentation and Load Paths of Montana Bridge

Figure 370. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near the girder ends.

Figure 371. Strain Transducers

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.

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 8. 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 lbs where the total rear weight equaled 28,080 lbs and the front axle weight was 10,940 lbs. Assuming equal weights on each rear axle, the rear axles weighed 14,040 lbs. The axle weights are shown in Figure 8 and the load truck used for testing is shown in Figure 373.

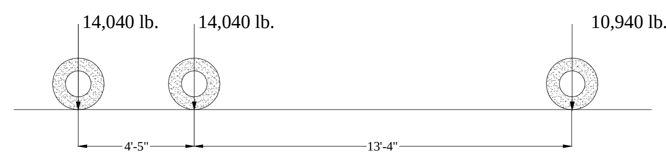

Figure 372. Truck Configuration and Axle Loads

Figure 373. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

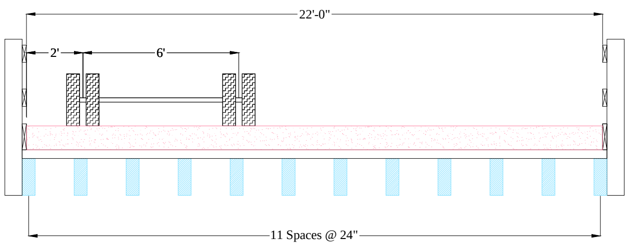

Figure 374. Transverse Truck Position - Load Path 1

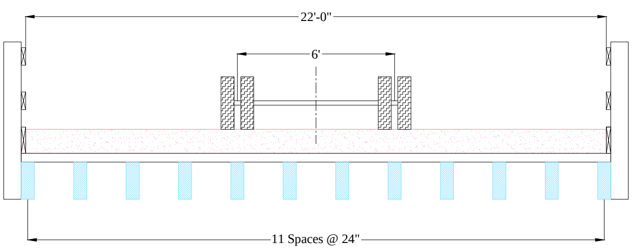

Figure 375. Transverse Truck Position - Load Path 2

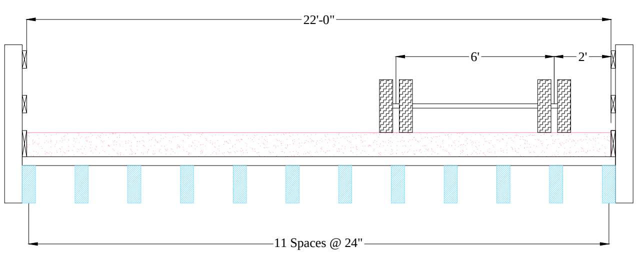

Figure 376. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the ISU research team. In particular, the wearing surface, deck, and superstructure were thoroughly assessed. In addition, the substructure was viewed, though the ISU team was primarily concerned with the superstructure.

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.

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 Figure 377). The ends of the deck boards appeared in good condition, there was no visible detachment of the deck boards from the girders except for 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.

Figure 377. White Residue Formation on Deck

Superstructure

Moderate checking was present along the midline in approximately one-half of 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 the superstructure presumably from water seepage. Exterior girders were in worse condition than the interior girders presumably a result of more exposure to weathering conditions (see Figure 258). 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.

Figure 378. Checking in Exterior Girders

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 ultraviolet degradation. The substructure appeared in good condition though 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 15 through 17 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. Looking to the above mentioned figures this two wave relationship is quite evident and clearly the deflections represent the difference in load from the front axle to the back axles.

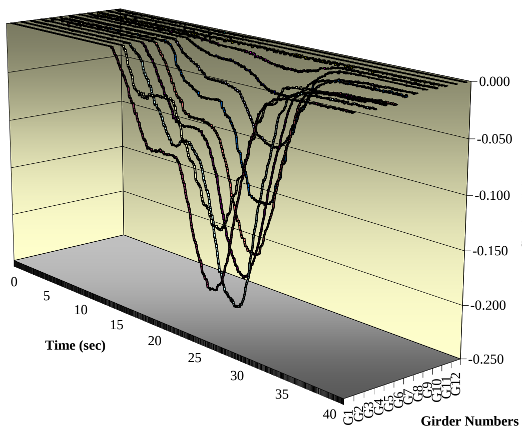

Figure 379. Deflections for Load Path 1

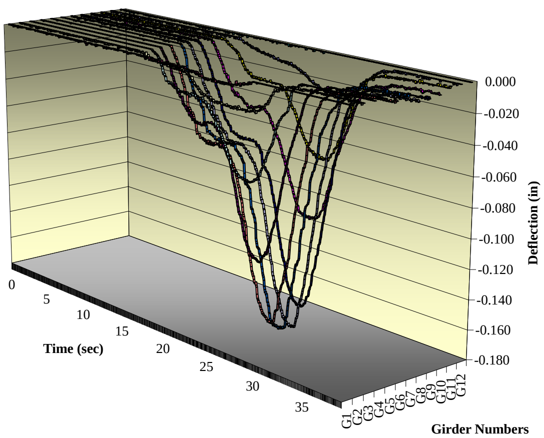

Figure 380. Deflections for Load Path 2

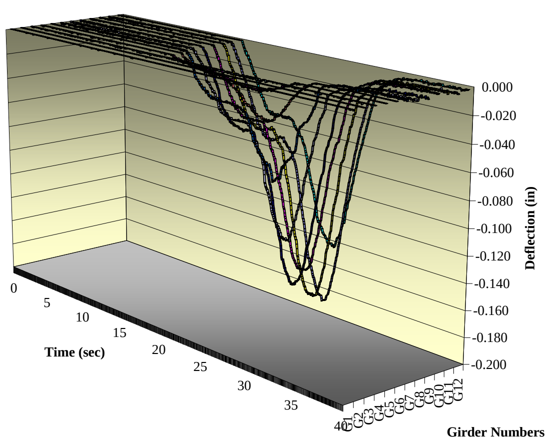

Figure 381. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 18 through 20. 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.

Table 61. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.222	0.177	0.183

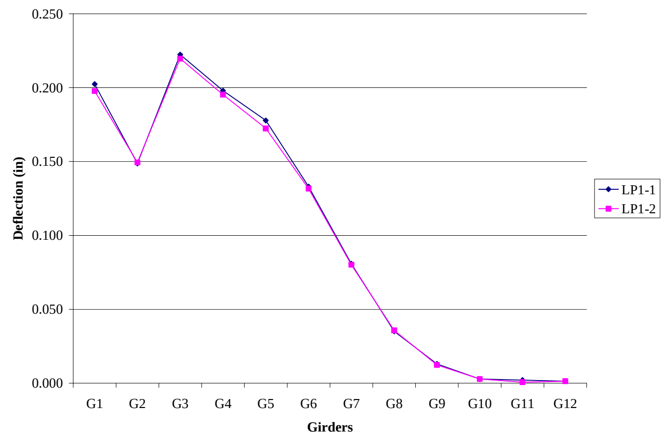

Figure 382. Maximum Deflections for Load Path 1

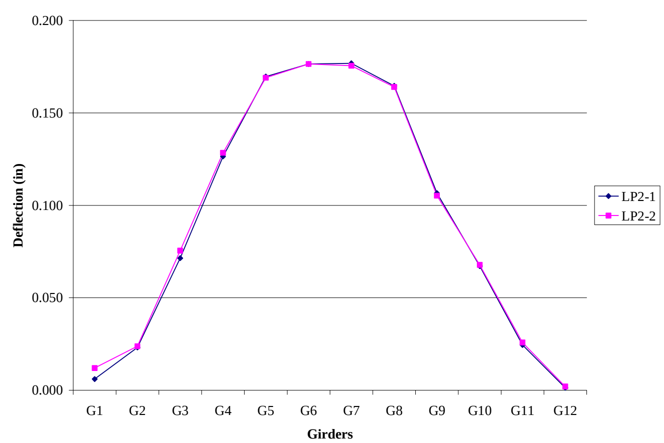

Figure 383. Maximum Deflections for Load Path 2

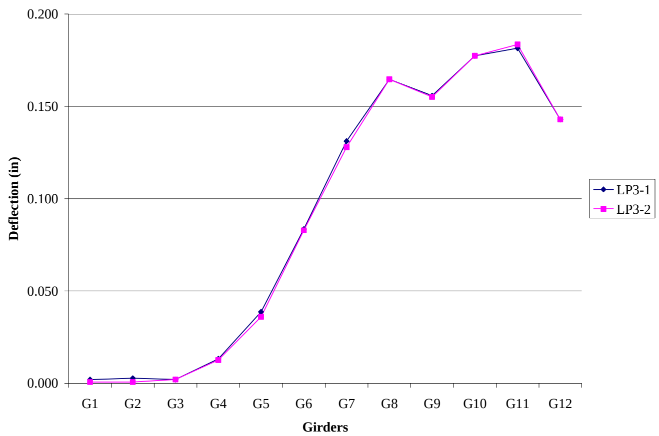

Figure 384. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 62. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 25

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 63. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
543	683	658

The minimum n-value of the three load paths was 543. This value was greater than all of the minimum recommended values for timber girders in Table 3. Therefore, the maximum midspan deflections are more favorable for no wearing surface cracking to develop.

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 Equation 2.

Equation 26

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

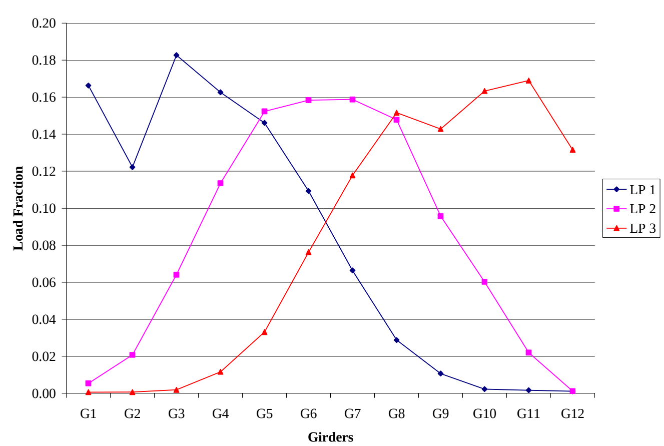

Figure 385. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

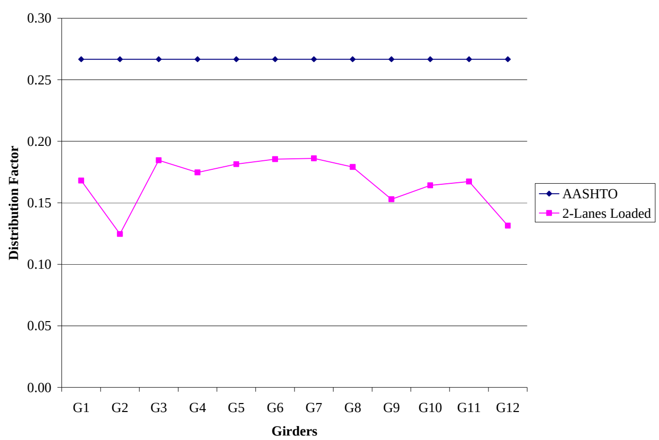

Figure 386. AASHTO Design Live Load Distribution

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. Figures 23 through 25 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 4.

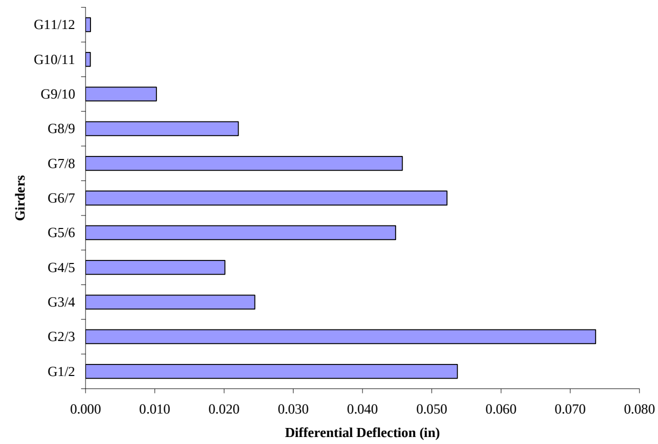

Figure 387. Differential Deflections for Load Path 1

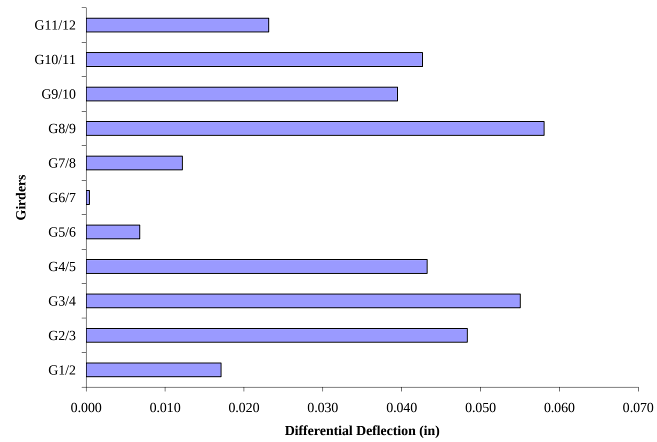

Figure 388. Differential Deflections for Load Path 2

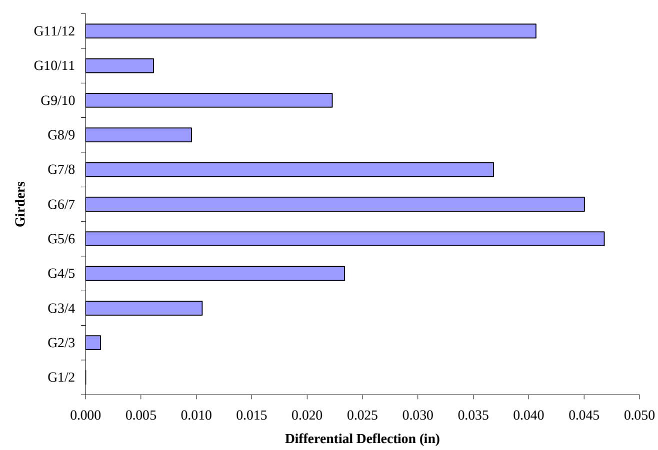

Figure 389. Differential Deflections for Load Path 3

Table 64. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.074	0.058	0.047

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 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 207 and 244 microstrain, respectively. The strain plot at midspan is shown in Figures 26 through 28 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 26 through 28 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, i.e., the girders act independently of the deck when subjected to bending.

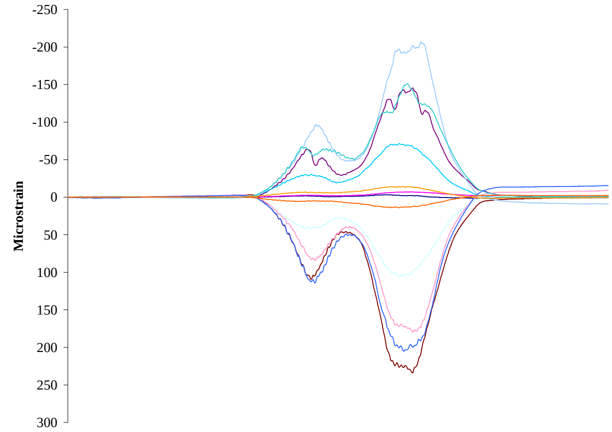

Figure 390. Strain at Midspan for Load Path 1

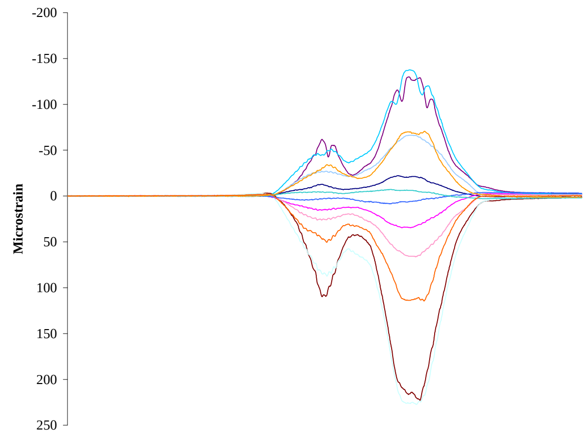

Figure 391. Strain at Midspan for Load Path 2

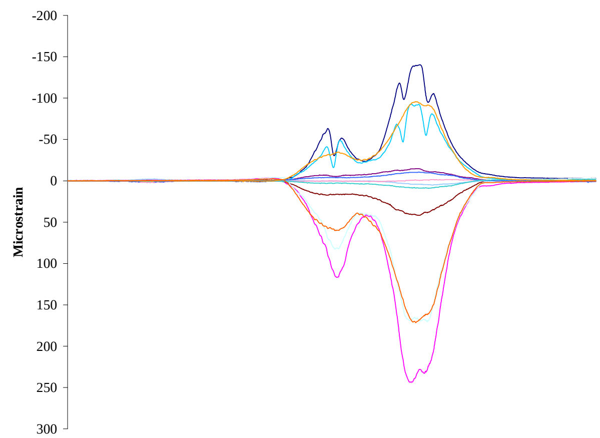

Figure 392. Strain at Midspan for Load Path 3

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, 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 percent. The moisture content measurements are summarized in Table 5.

Table 65. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	9.8
Bottom of Deck Between Girders 6 & 7	8.5
Bottom of Deck Between Girders 11 & 12	11.0

Finite Element Analysis

A finite element model was developed (see Figure 89) 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 AASHTO HS20 tandem axle design vehicle. Deflection and tensile strain results at midspan were obtained from the model.

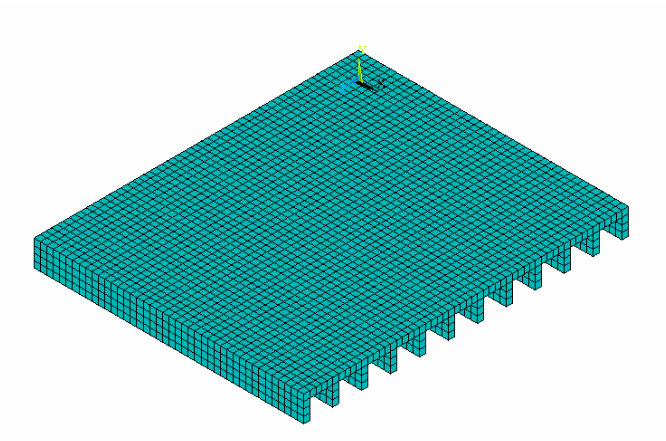

Figure 393. Finite Element Model

Figures 30 through 32 show the calibrated model results when subjected to the same load as that during the static load test. Notice the similarities between each plot.

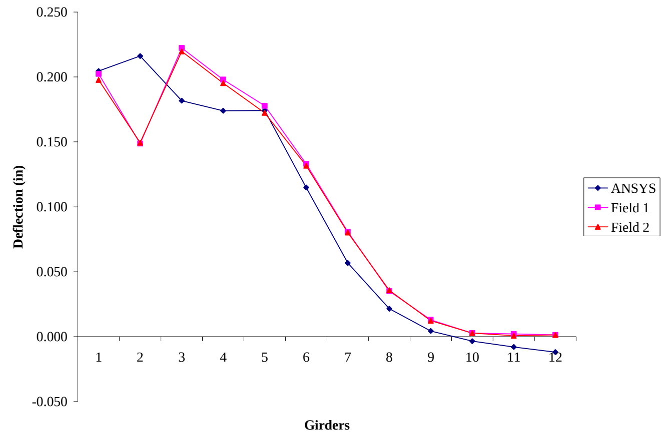

Figure 394. ANSYS Calibration Results Load Path 1

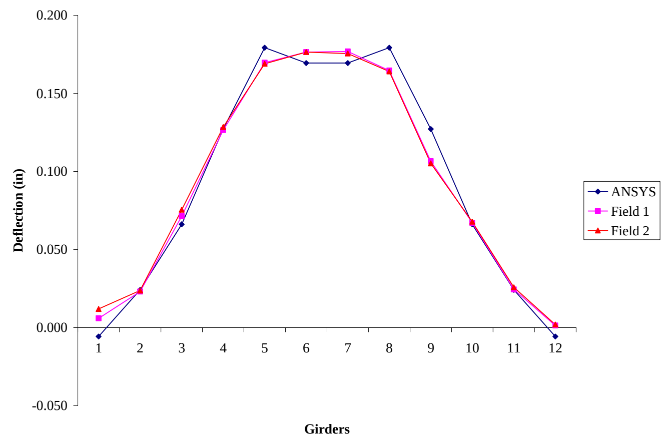

Figure 395. ANSYS Calibration Results Load Path 2

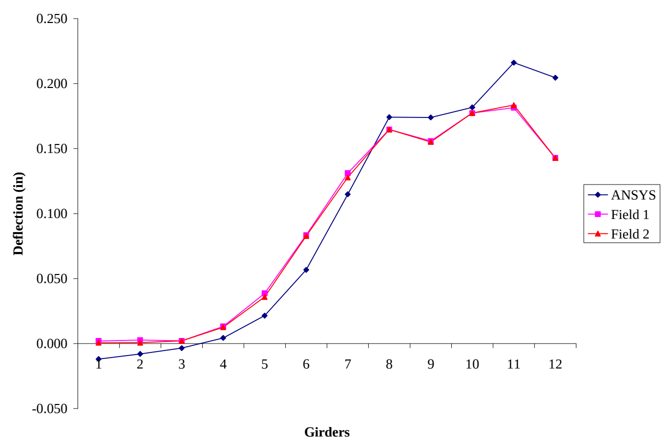

Figure 396. ANSYS Calibration Results Load Path 3

Figure 93 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.

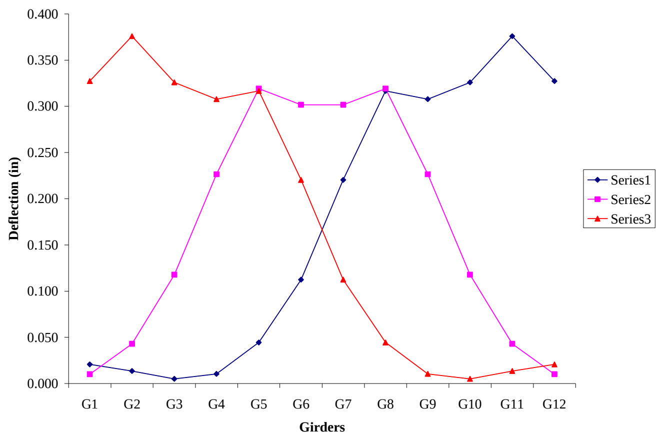

Figure 397. ANSYS Deflection Results for Each Load Path when Subjected to HS20 Tandem Axle Design Vehicle

Figure 398 shows the maximum tensile stresses at midspan due to the AASHTO HS20 tandem axle design vehicle traveled along each load path.

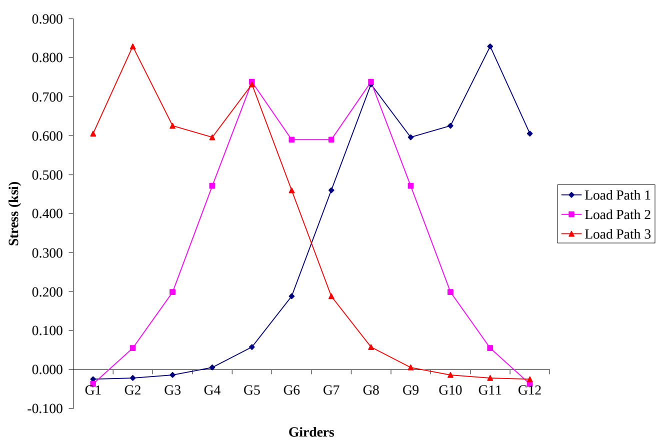

Figure 398. ANSYS Tensile Stress for Each Load Path when Subjected to HS20 Tandem Axle Design Vehicle

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, 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. 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, i.e., 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 26 though 28 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 424 and 500 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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 psi, 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; 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 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 1500 psi 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.

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APPENDIX O

PERFORMANCE REPORT

NATIONAL FOREST SERVICE BURNT FORK BRIDGE

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NATIONAL FOREST SERVICE BURNT FORK BRIDGE

Terry Wipf
Brent Phares
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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 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.

Acknowledgements

We would like to 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.

John Kettel, United States Forest Service employee who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Burnt Fork Bridge.

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, (i.e., single span, timber girder bridges), 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.

Background

The location of the Burnt Fork Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 399. Burnt Fork Bridge Location

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 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 Figure 400).

Figure 400. Bridge Substructure

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 2.

Figure 401. Burnt Fork Bridge Parapet Support

Girders measure 28 ft-3 in. from end to end and have a clear span of 26 ft-3 in. A total of 7 girders, spaced 25-1/2 in. center-to-center, measuring 8 in. x 23-1/2 in. in cross-section are present and are seated and toe-nailed to the 12-in. x 12-in. timber sills with spikes. The deck consists of individual 2 in. x 6 in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck are ten 3 in. x 12 in. timber runners totaling 10 ft wide along the center of the bridge. Figure 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 7 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.

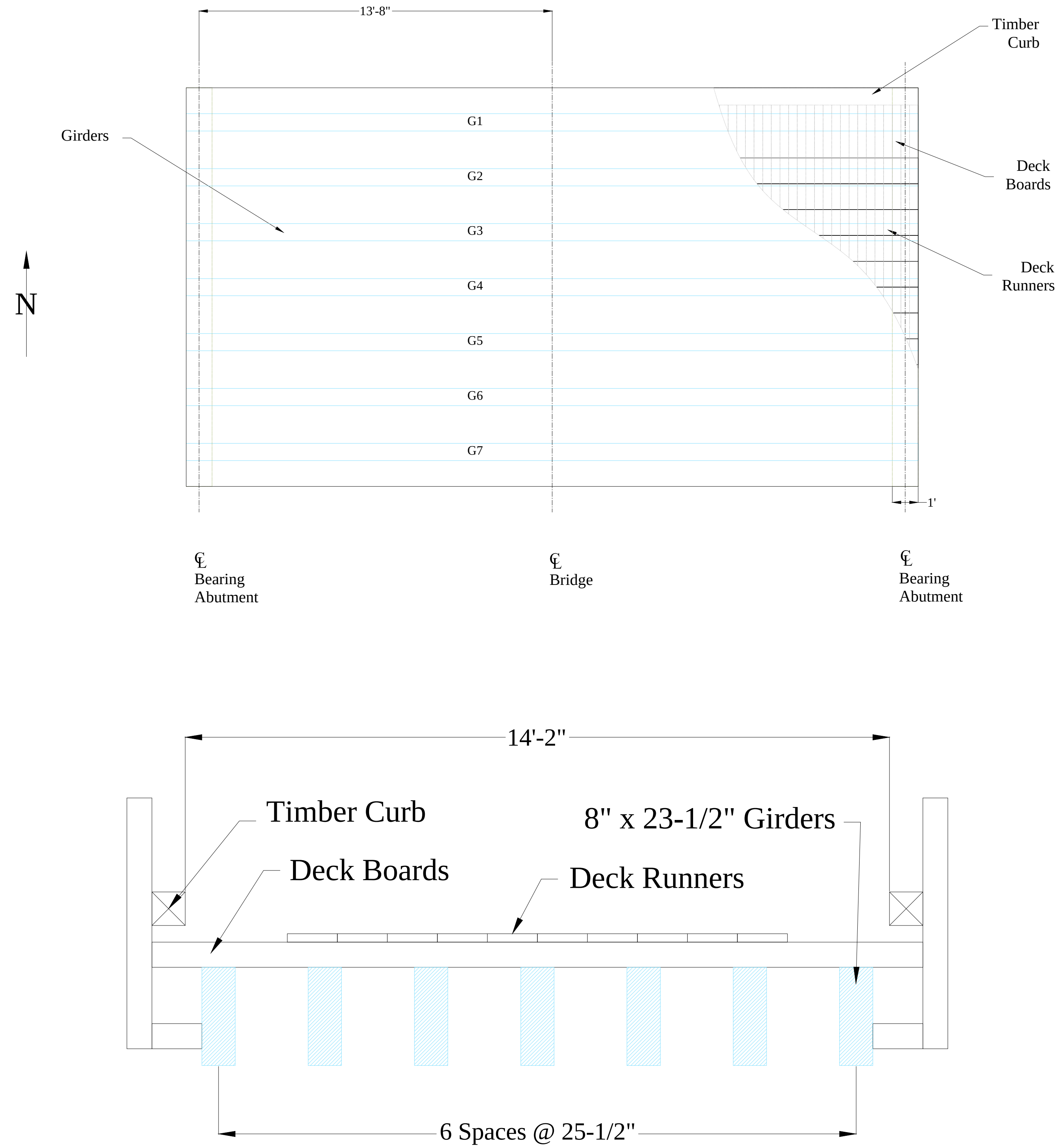

Figure 402. Plan and Profile Layout of Burnt Fork Bridge

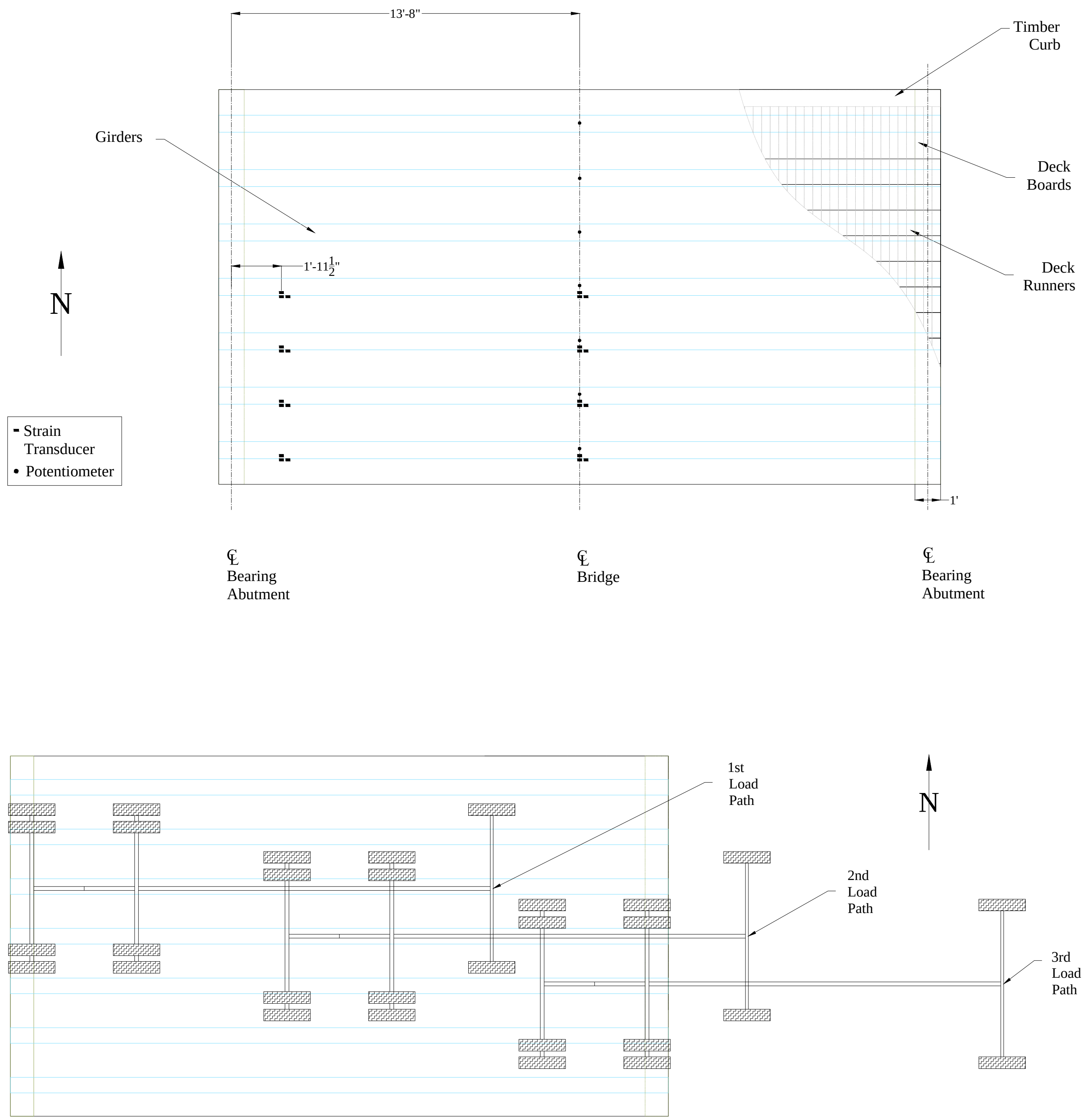

Figure 403. Instrumentation and Load Paths of Burnt Fork Bridge

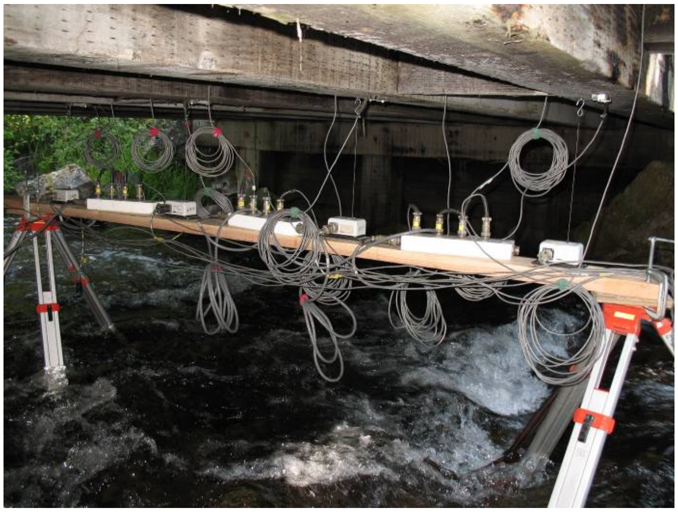

Figure 404. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near the girder ends.

Figure 405. Strain Transducers

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.

Static Loading

Static loading of the bridge was completed using a tandem axle dump truck provided by the U.S. Forest Service. Dimensions of the truck are shown in Figure 8. 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 lbs where the total rear weight equaled 38,626 lbs and the front axle weight was 16,554 lbs. Assuming equal weights on each rear axle, the rear axles weighed 19,313 lbs. The axle weights are shown in Figure 8 and the load truck used for the testing is shown in Figure 407.

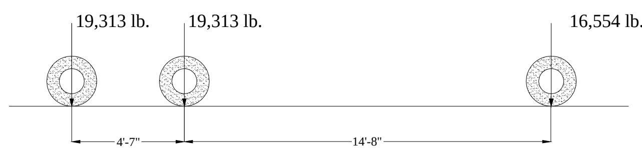

Figure 406. Truck Configuration and Axle Loads

Figure 407. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

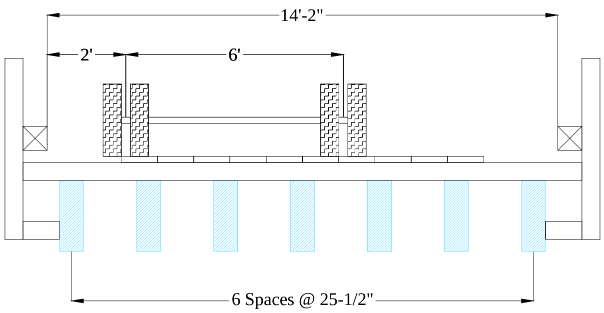

Figure 408. Transverse Truck Position - Load Path 1

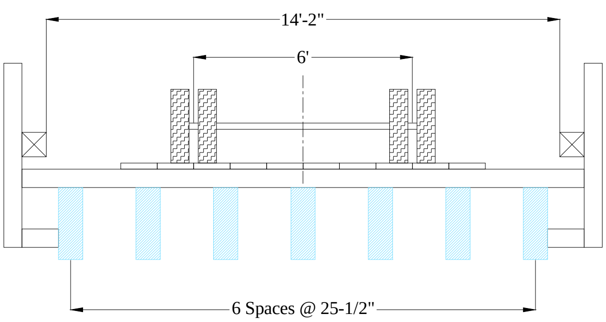

Figure 409. Transverse Truck Position - Load Path 2

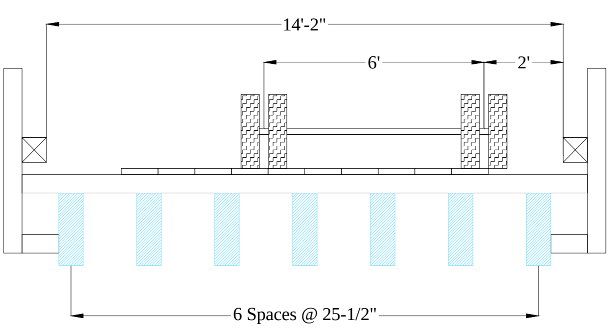

Figure 410. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the ISU research team. In particular, the wearing surface, deck, and superstructure were thoroughly assessed. In addition, the substructure was viewed, though the ISU team was primarily concerned with the superstructure.

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.

The report for the bridge inspection conducted on October 20, 2000 was obtained from the U.S. 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 257.

Figure 411. Wearing Surface

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 Figure 349). Water appeared to be leaking through the decking and water staining was observed by the ISU research team.

Figure 412. Twisting of Deck Boards

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 like checking was observed. Conversely, checking was observed on the two exterior girders at the girder midline (see Figure 258). 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 413. Checking in Exterior Girders

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 16 through 18 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.

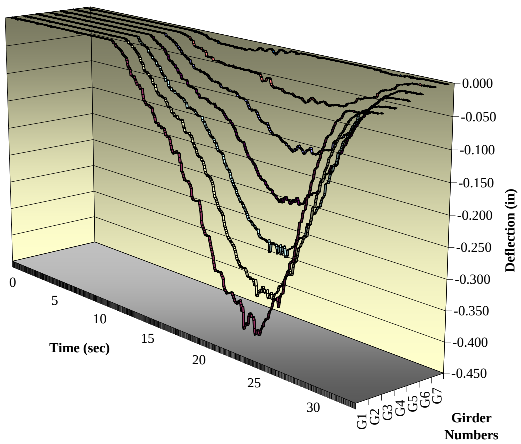

Figure 414. Deflections for Load Path 1

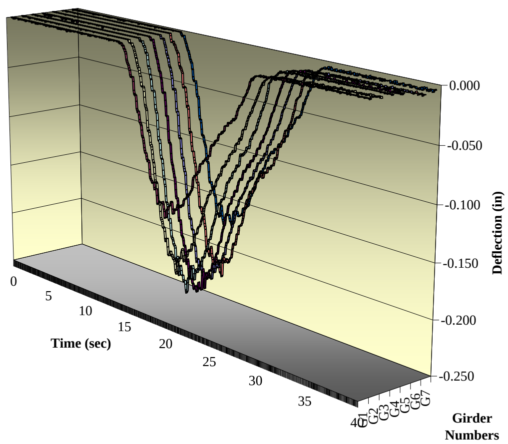

Figure 415. Deflections for Load Path 2

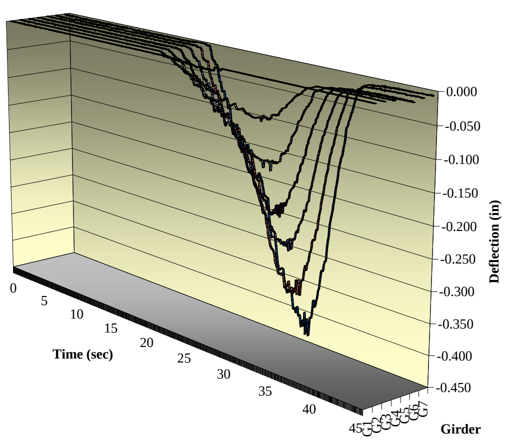

Figure 416. Deflections for Load Path 3

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 18 through 20. 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.

Table 66. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.413	0.232	0.435

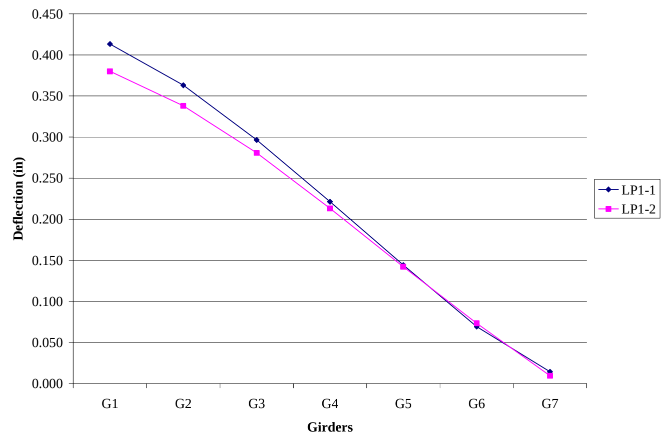

Figure 417. Maximum Deflections for Load Path 1

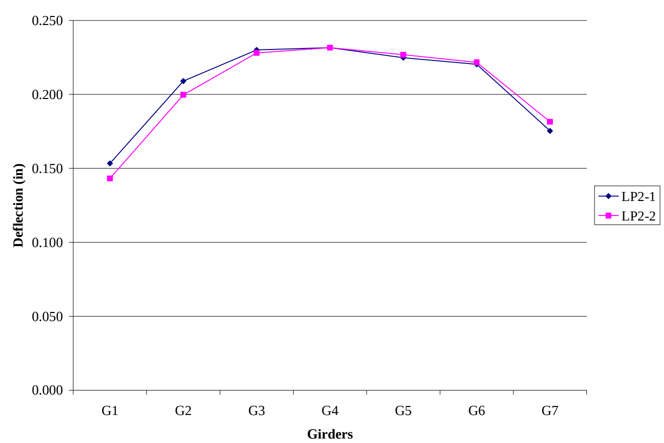

Figure 418. Maximum Deflections for Load Path 2

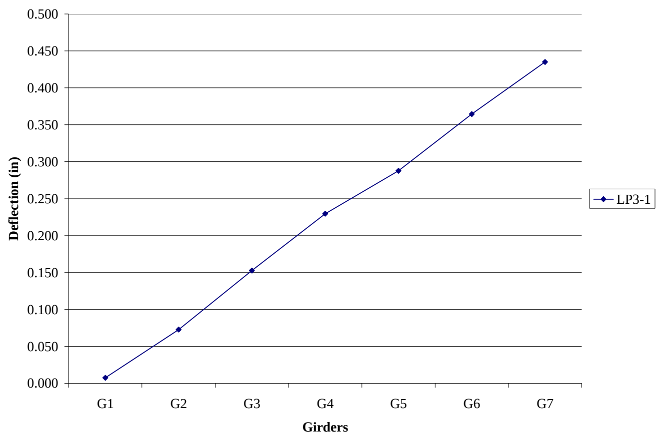

Figure 419. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). These recommendations are primarily derived from the effects of deflection on the wearing surface of the bridge, though other degradation is possible, 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 deterioration exists.

Table 67. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 27

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 68. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
589	1051	559

The minimum n-value of the three load paths was 559. This value was greater than all of the minimum recommended values for timber girders stated in Table 3.

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 Equation 2.

Equation 28

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

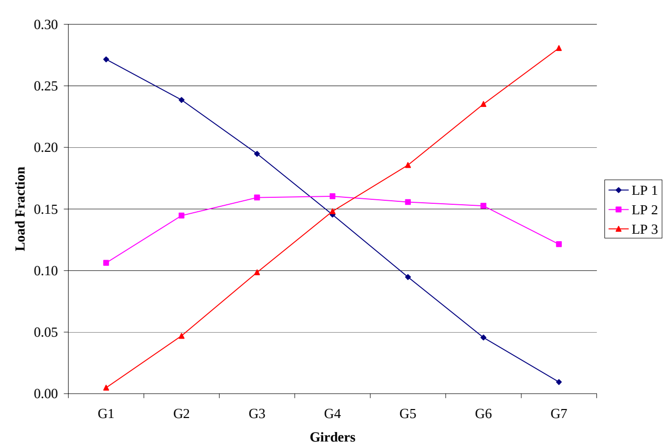

Figure 420. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

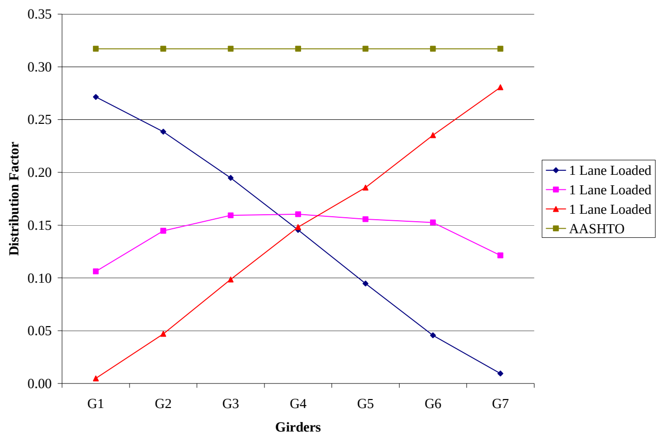

Figure 421. AASHTO Design Live Load Distribution

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. 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 bridge. One should note that differential deflection primarily effects cracking in the wearing surface. Even so, other types of degradation can occur with large differential deflections. Figures 24 through 26 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 4.

Table 69. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.077	0.056	0.080

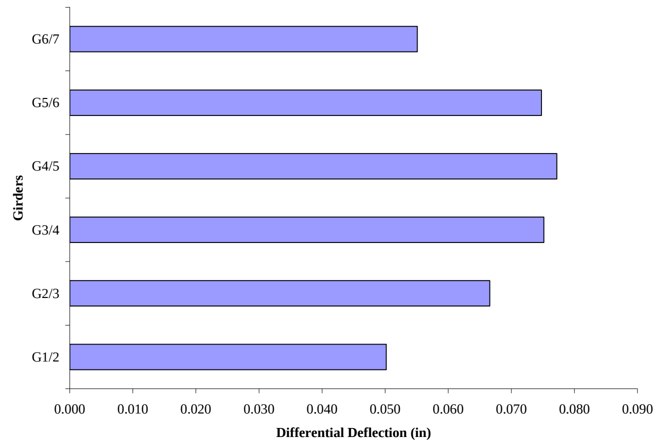

Figure 422. Differential Deflections for Load Path 1

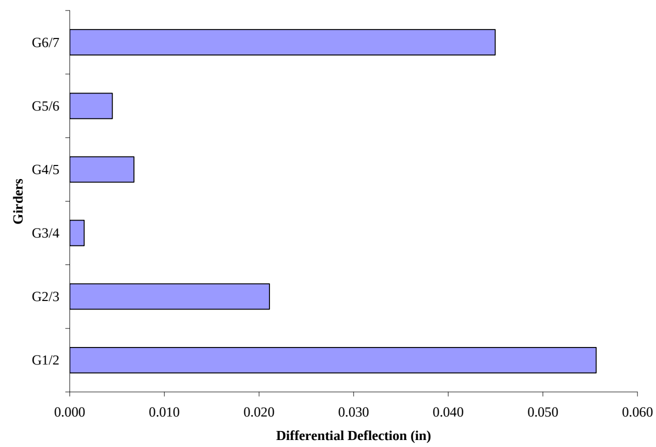

Figure 423. Differential Deflections for Load Path 2

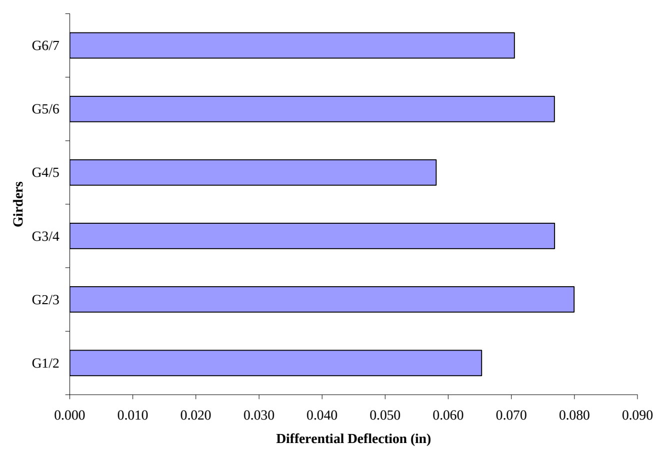

Figure 424. Differential Deflections for Load Path 3

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 27. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 145 and 188 microstrain, respectively. The strain plot at midspan is shown in Figures 27 through 28 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 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 through 28 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, i.e., the girders act independently of the deck when subjected to bending.

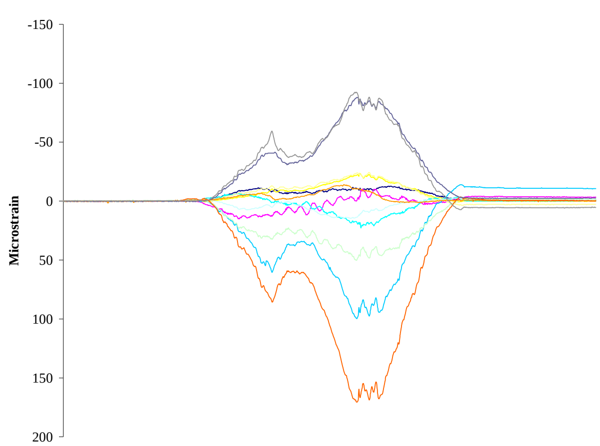

Figure 425. Strain at Midspan for Load Path 1

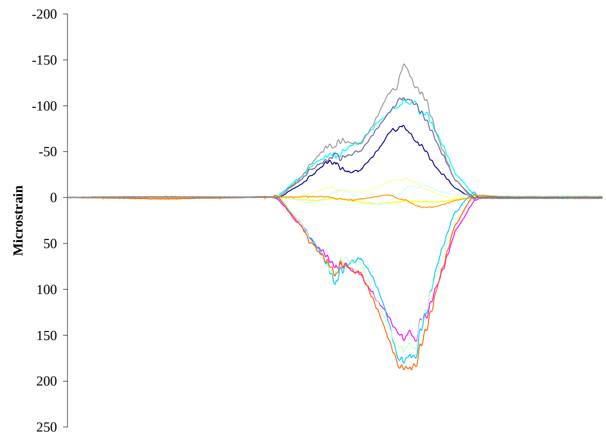

Figure 426. Strain at Midspan for Load Path 2

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, 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 percent and are summarized in Table 5.

Table 70. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 2	16.1
Bottom of Deck Between Girders 4 & 5	17.8
Bottom of Deck Between Girders 6 & 7	14.4

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, 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 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 27 though 28 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 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 216 and 280 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 1500 psi 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.

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APPENDIX P

PERFORMANCE REPORT

NATIONAL FOREST SERVICE TRAPPER CREEK BRIDGE

United States Department of Agriculture

Forest Service

Forest Products Laboratory

Iowa State University

PERFORMANCE REPORT

NATIONAL FOREST SERVICE TRAPPER CREEK BRIDGE

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

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 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.

Acknowledgements

We would like to 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.

John Kettell, United States Forest Service employee who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Contents

Introduction

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

While traffic loads increased, the use of high strength materials like 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 steel and concrete industries maintained low prices. Cleary 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.

A number of benefits exist when using timber as a primary bridge construction material. 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 like 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. Typically, inspections are conducted through visual inspection methods which often do not thoroughly detect deterioration in timber members. The development of inspection and maintenance practices is still in the early stages; therefore, more efficient practices are desired. With future advancements in timber bridge construction these 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 United States 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 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.

This report is presented as the summary and results of one of fifteen total bridge tests intended to gather and analyze information on timber bridge performance under load. The following explains the testing procedure and reports the test results for the Trapper Creek Bridge.

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, (i.e., 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.

Background

The location of Trapper Creek Bridge is shown in Figure 1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

Figure 427. Trapper Creek Bridge Location

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 surface set on a 10 degree 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 Figure 428).

Figure 428. Bridge Substructure

The parapet consists only of a timber 8 in. by 12 in. timber curb bolted to the deck boards through an 8 in. by 12 in. board. The curb is shown in Figure 2.

Figure 429. Trapper Creek Bridge Parapet Support

Girders measure 35 ft-10 in. from end to end and have a clear span of 33 ft-10 in. A total of 4 girders, spaced 46 in. center-to-center, measuring 9 in. x 27 in. in cross-section are present and are seated and bolted to the 12-in. x 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 4 illustrates the layout of the bridge.

Evaluation Methodology

The 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 ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. The signals from these instruments were collected using an Optim Megadac 3415AC data acquisition system running TCS windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) using WinSTS software.

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 5. 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 4 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 in. x 6 in. planks and tripods. Deflection instrumentation is shown in Figure 250.

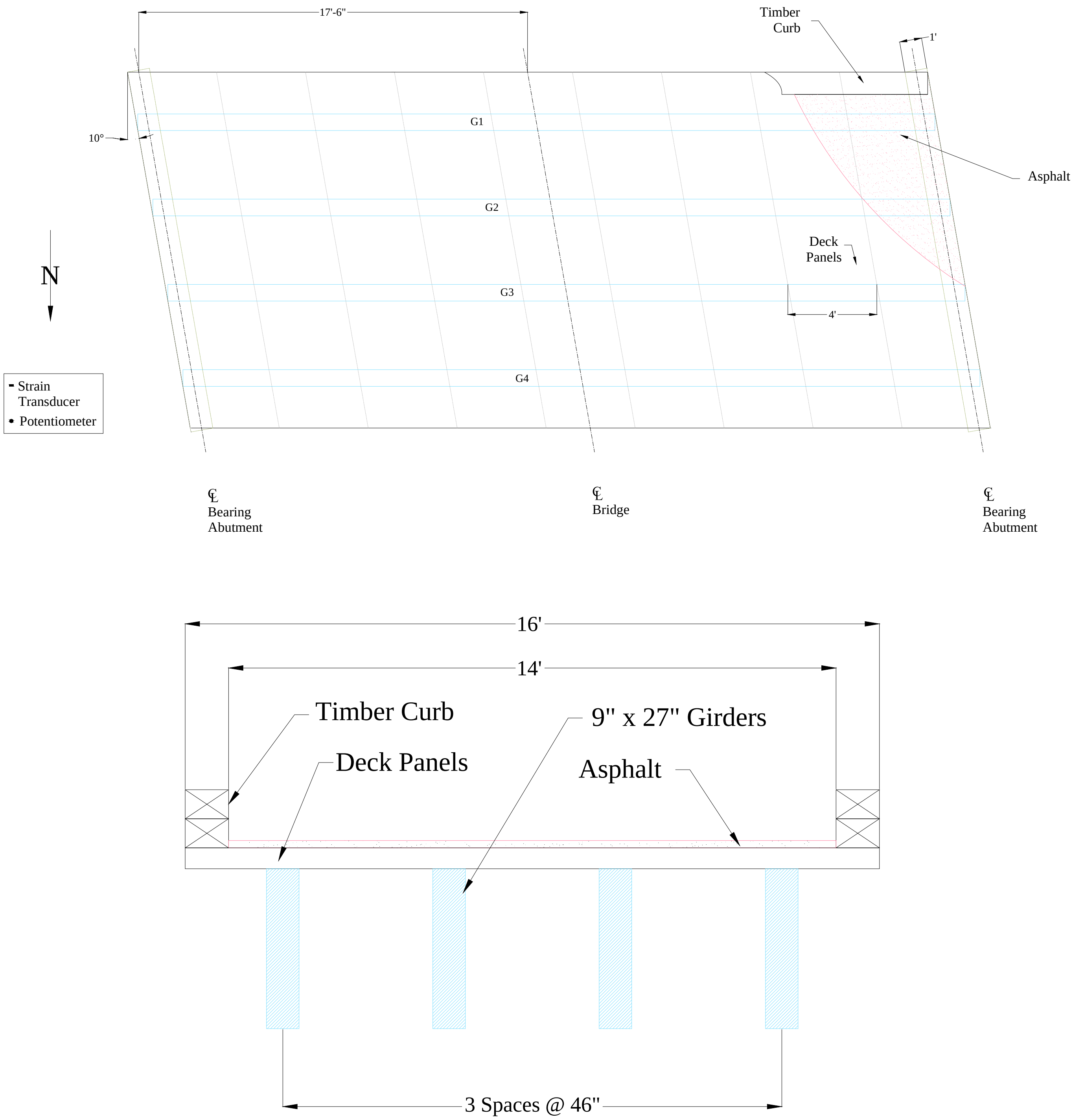
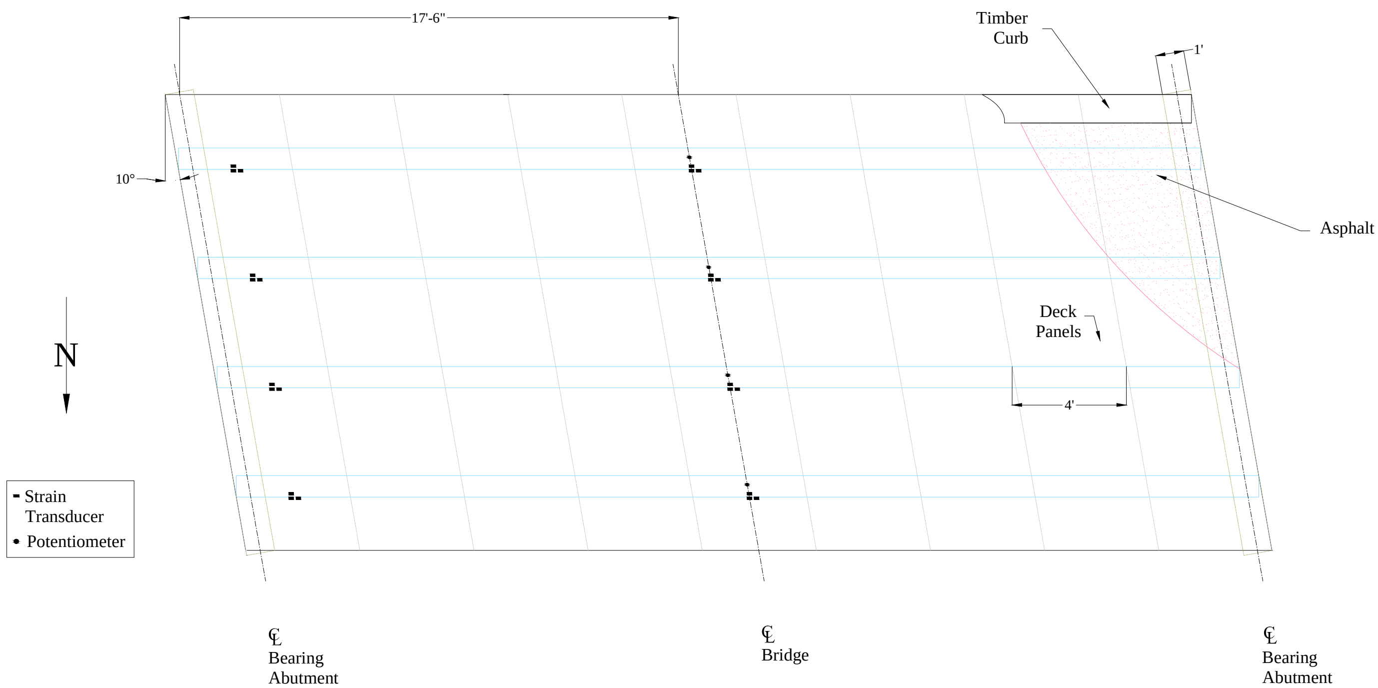


Figure 430. Plan and Profile Layout of Trapper Creek Bridge



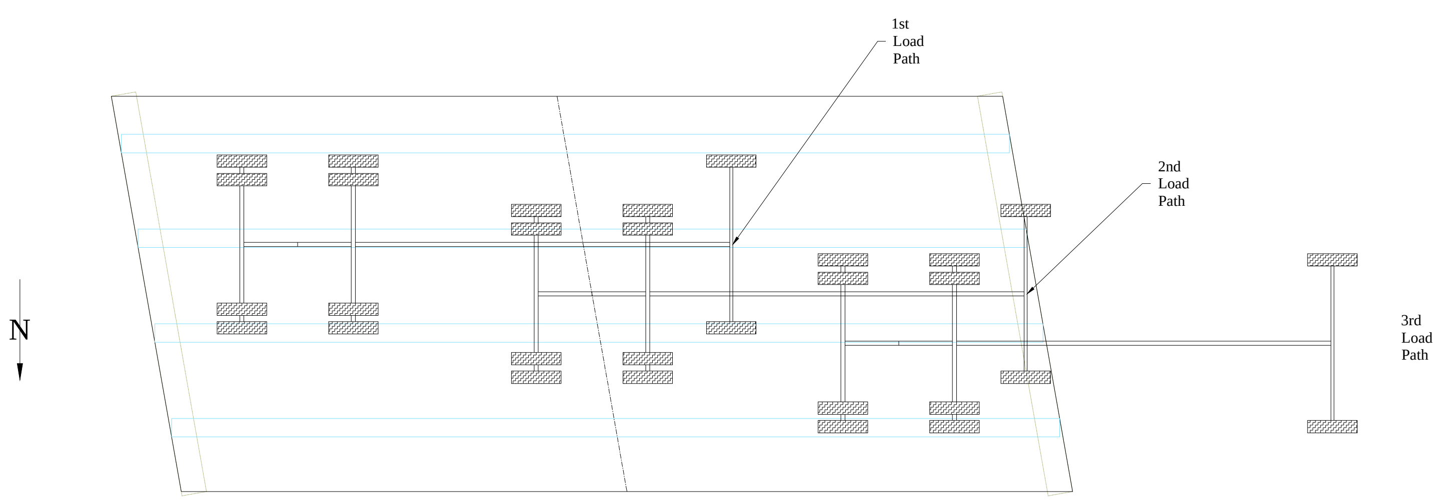


Figure 431. Instrumentation and Load Paths of Trapper Creek Bridge

Figure 432. Deflection Instrumentation

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 Figure 5). 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 7 shows a typical setup of strain transducers near midspan.

Figure 433. Strain Transducers

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.

Static Loading

Static loading of the bridge was completed using a tandem axle dump truck provided by the U.S. Forest Service. Dimensions of the truck are shown in Figure 8. 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 lbs where the total rear weight equaled 38,626 lbs and the front axle weight was 16,554 lbs. Assuming equal weights on each rear axle, the rear axles weighed 19,313 lbs. The axle weights are shown in Figure 8 and the load truck used for testing is shown in Figure 435.

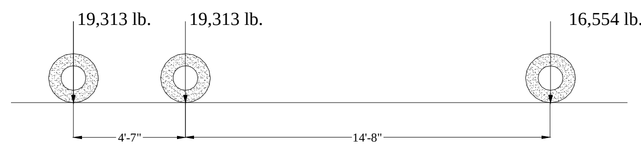


Figure 434. Truck Configuration and Axle Loads

Figure 435. Tandem Axle Load Truck

Three load paths were considered when testing the bridge (see Figures 10 through 12). 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.

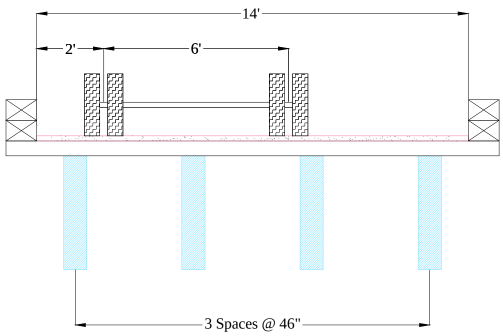


Figure 436. Transverse Truck Position - Load Path 1

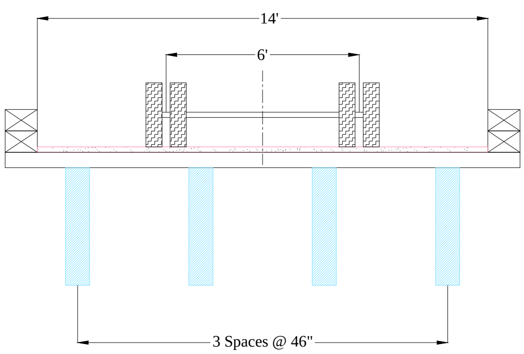


Figure 437. Transverse Truck Position - Load Path 2

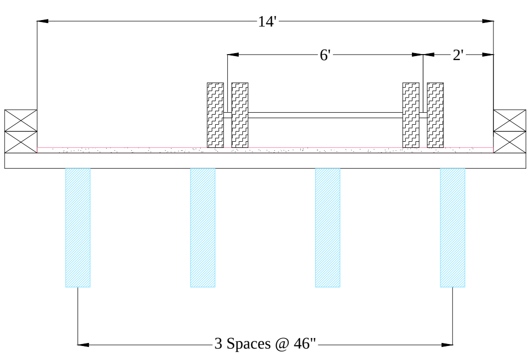


Figure 438. Transverse Truck Position - Load Path 3

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the ISU research team. In particular, the wearing surface, deck, and superstructure were thoroughly assessed. In addition, the substructure was viewed, though the ISU team was primarily concerned with the superstructure.

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.

The report for the bridge inspection conducted on November 8, 2005 was obtained from the U.S. 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. Aside from the cracking, a large amount of gravel debris was observed on the wearing surface and was noted in the 2005 Forest Service report. The cracking and accumulation of debris is shown in Figure 257.

Figure 439. Wearing Surface Cracking and Debris

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 Figure 349). 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.

Figure 440. Washing of Creosote at Panels Ends

Superstructure

Water appeared to be leaking through the decking as the creosote treatment on the girders was streaking (see Figure 258). 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.

Figure 441. Girder Streaking and Holes

Overall Structure

The overall structure was in good 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. 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 16 through 18 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.

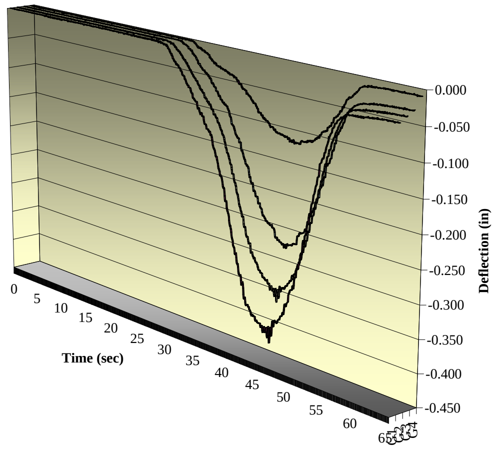


Figure 442. Deflections for Load Path 1

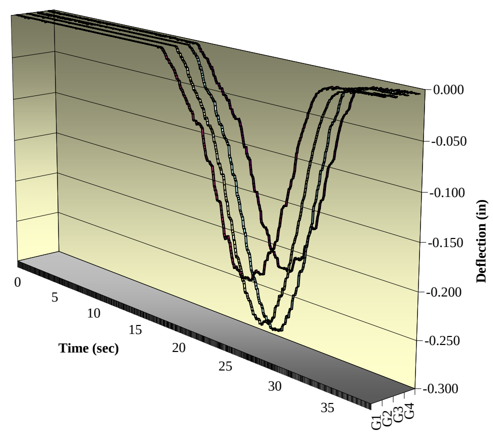


Figure 443. Deflections for Load Path 2

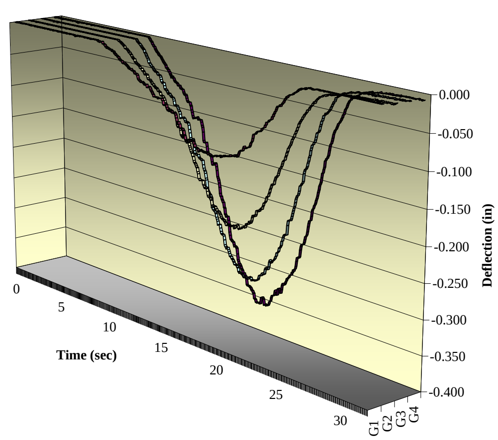


Figure 444. Deflections for Load Path 3

The maximum deflections achieved for each load path are presented in Table 1. Each passing of the three load paths is illustrated in Figures 19 through 21. 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.

Table 71. Maximum Girder Deflections

Maximum Midspan Deflection For Each Passing (in.)		
Load Path 1	Load Path 2	Load Path 3
0.413	0.284	0.354

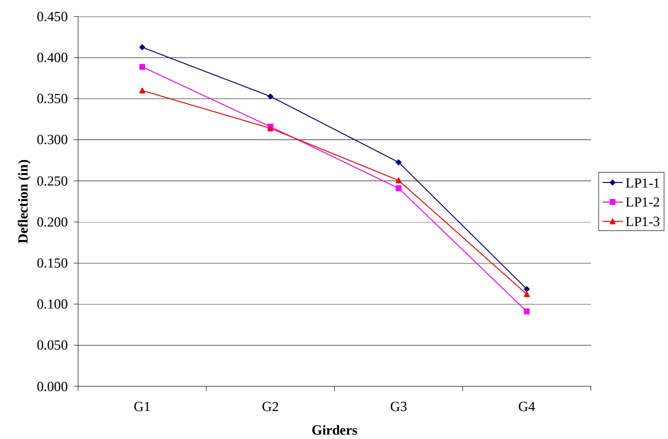


Figure 445. Maximum Deflections for Load Path 1

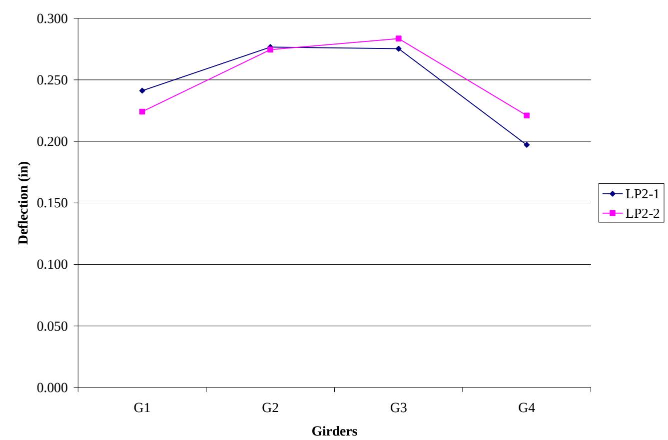


Figure 446. Maximum Deflections for Load Path 2

Maximum Deflections

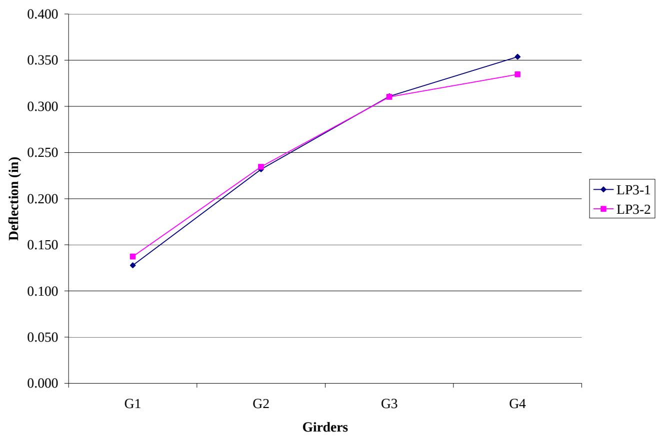


Figure 447. Maximum Deflections for Load Path 3

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table 2). 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.

Table 72. Live Load Deflection Limit States

Source	n-Value
Timber Bridges [8]	L/360
Highway Bridges [2]	L/425
AASHTO [1]	L/500

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 HS20 tandem axle loading, which is most like the trucks used in this study. The equation for the n-value is

Equation 29

$$n = \frac{Length}{Deflection \times \dfrac{HS20Load}{ActualLoad}}$$

where, deflection and length are in inches. Table 3 lists the n-value for the girder of most deflection for each load path.

Table 73. Most Critical n-Values

n-Value for the Girder of Most Deflection for Each Load Path		
Load Path 1	Load Path 2	Load Path 3
760	1106	887

The minimum n-value of the three load paths was 760. This value is greater than all of the minimum recommended values for timber girders stated in Table 3.

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 Equation 2.

Equation 30

$$LF_i = \frac{\Delta_i}{\sum_{i=1}^{n} \Delta_i}$$

where,

LF_i	= load fraction of the i^{th} girder
Δi	= deflection of the i^{th} girder
$\Sigma\Delta_i$	= sum of all girder deflections
n	= number of girders

Figure 22 shows the load fractions for each girder for each load path.

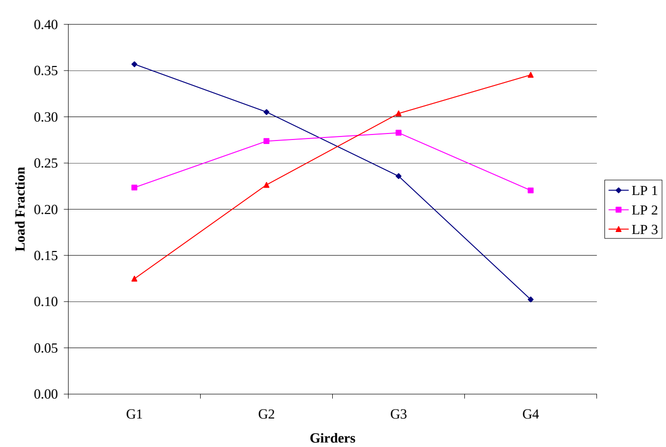


Figure 448. Load Fractions for Each Load Path

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, and S is equal to 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 23 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.

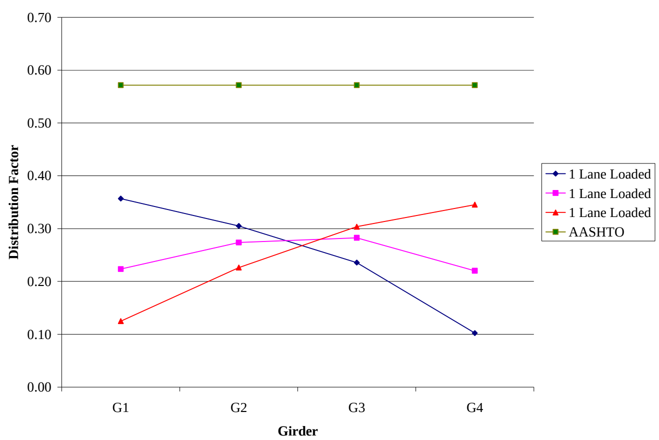


Figure 449. AASHTO Design Live Load Distribution

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. Figures 24 through 26 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 4.

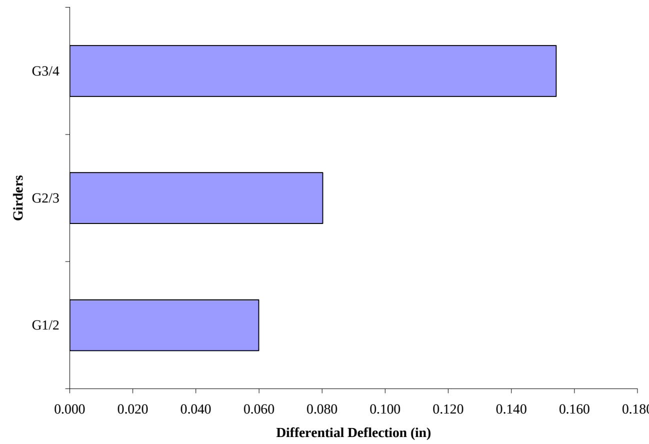


Figure 450. Differential Deflections for Load Path 1

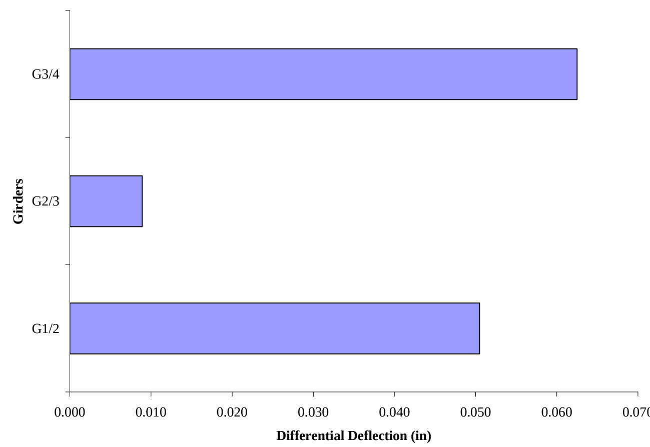


Figure 451. Differential Deflections for Load Path 2

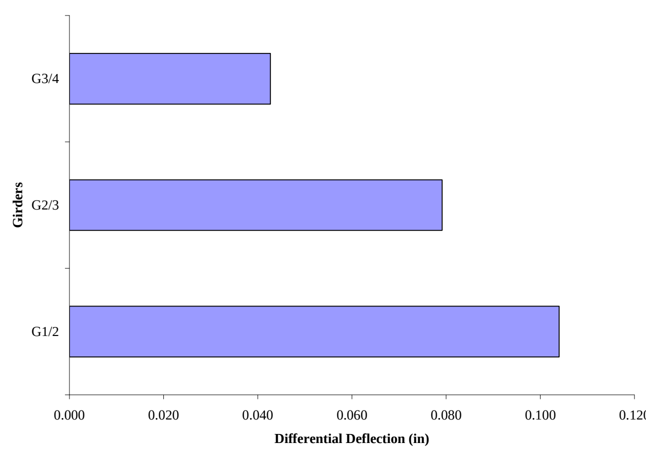


Figure 452. Differential Deflections for Load Path 3

Table 74. Maximum Differential Deflection

Maximum Differential Deflections at Midspan Between Adjacent Girders (in.)		
Load Path 1	Load Path 2	Load Path 3
0.154	0.063	0.104

The maximum differential deflection of 0.154 in. occurs in load path 1 and equals 0.040 in. per ft of girders spacing. 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.

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 27 through 29 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 through 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, i.e., the girders act independently of the deck when subjected to bending.

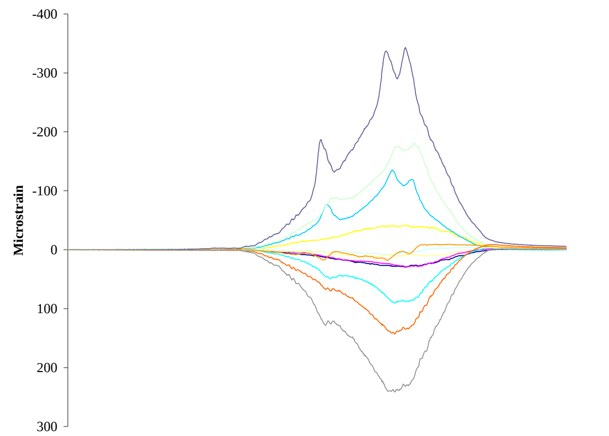


Figure 453. Strain at Midspan for Load Path 1

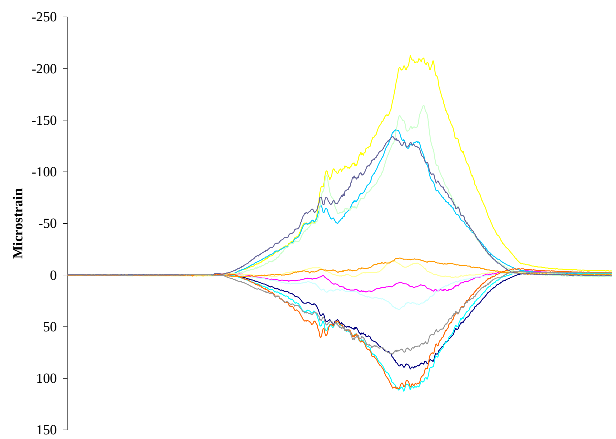


Figure 454. Strain at Midspan for Load Path 2

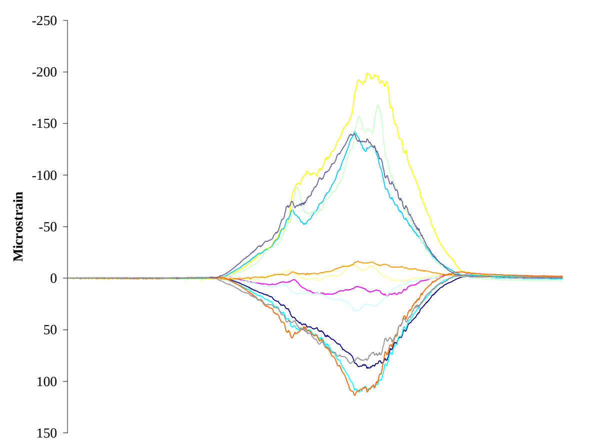


Figure 455. Strain at Midspan for Load Path 3

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 percent. The moisture content measurements are summarized in Table 5.

Table 75. Moisture Content Summary

Moisture Content Measurement Locations and Values	
Location	%
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 & 3	21.2
Bottom of Deck Between Girders 3 & 4	21.0
Outside of Girder 1	18.1

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 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.

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 27 though 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 342 and 241 microstrain, respectively. These strains equate to maximum stresses of 393 and 277 psi, respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 509 and 359 psi are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 psi 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; 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 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 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 1500 psi 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 of the recommended maximum values.

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