Glulam Timber Bridges for Local Roads

Zachary Charles Carnahan
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GLULAM TIMBER BRIDGES FOR LOCAL ROADS

BY

ZACHARY CHARLES CARNAHAN

A thesis in partial fulfillment of the requirements for the
Master of Science
Major in Civil Engineering
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2017
GLULAM TIMBER BRIDGES FOR LOCAL ROADS

This thesis is approved as a creditable and independent investigation by a candidate for the Master of Science in Civil Engineering degree and is acceptable for meeting the thesis requirements for this degree. Acceptance of this does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

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DISCLAIMER

The contents of this report, funded in part through grant(s) from the Federal Highway Administration, reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the South Dakota Department of Transportation, the State Transportation Commission, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>AITC</td>
<td>American Institute of Timber Construction</td>
</tr>
<tr>
<td>ANSI</td>
<td>American National Standards Institute</td>
</tr>
<tr>
<td>ASD</td>
<td>Allowable Stress Design</td>
</tr>
<tr>
<td>AWC</td>
<td>American Wood Council</td>
</tr>
<tr>
<td>DOT</td>
<td>Department of Transportation</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>ft</td>
<td>Feet</td>
</tr>
<tr>
<td>in</td>
<td>Inch</td>
</tr>
<tr>
<td>kip</td>
<td>1000 pounds</td>
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<tr>
<td>klf</td>
<td>Kip per linear foot</td>
</tr>
<tr>
<td>ksi</td>
<td>Kip per square inch</td>
</tr>
<tr>
<td>lbs</td>
<td>Pounds</td>
</tr>
<tr>
<td>LRFD</td>
<td>Load and Resistance Factor Design</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transformer</td>
</tr>
<tr>
<td>NDS</td>
<td>National Design Specification for Wood Construction</td>
</tr>
<tr>
<td>psi</td>
<td>pounds per square inch</td>
</tr>
<tr>
<td>SDDOT</td>
<td>South Dakota Department of Transportation</td>
</tr>
<tr>
<td>SDSU</td>
<td>South Dakota State University</td>
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The most common type of bridges on South Dakota (SD) local roads are prestressed precast double-tee bridges. Currently, there is only one double-tee girder manufacturer in South Dakota (SD). In an attempt to provide more bridge type selection options for local governments, a study was performed at South Dakota State University to investigate the feasibility and performance of new types of single-span bridges suitable for local loads with low traffic. In one part of the study, Mingo (2016) developed a fully precast bridge incorporating full-depth deck-panels and prestressed inverted bulb-tee girders. The study presented in this thesis was performed to investigate the feasibility and performance of glulam timber bridges as additional design alternatives for SD local roads.

There are two types of glulam timber bridges: (1) transverse glulam deck on glulam girders and (2) longitudinal glulam deck. The performance of each type was experimentally investigated through full-scale testing. The full-scale glulam girder
bridge test model was 50-ft long and 9.25-ft wide. The full-scale glulam slab bridge was 16.5-ft long and 8-ft wide. Both bridges were first tested under the AASHTO Fatigue II limit state loading followed by strength testing. Both bridge types showed minimal damage during the fatigue testing. The only damage of the girder bridge was cracking of male-to-female deck-to-deck connections, which can be eliminated using flat-end panels. Ultimate testing of the two bridge systems confirmed that the AASHTO method of the design for timber bridges is adequate. Girders of glulam girder bridges should be designed as fully non-composite members. Furthermore, design and construction guidelines for both types of bridges were proposed. A cost analysis showed that the superstructure cost of glulam timber bridges and glulam slab bridges can be 70% and 50% respectively of that for double-tee bridges.

Based on the construction, testing, and cost analysis, it can be concluded that both types of glulam timber bridges are viable alternatives to the double-tee girder bridges.
1. INTRODUCTION

The present thesis is part of a study funded jointly by the South Dakota Department of Transportation (SDDOT) and Mountain Plains Consortium (MPC) performed at South Dakota State University (SDSU) to develop alternatives to double-tee bridge systems that are common on South Dakota local roads. The study specifically evaluates two types of glulam timber bridges as alternatives to the double-tee bridge system.

1.1 Problem Statement

Many bridges across the United States of America are in need of replacement. There are a total of 5,870 bridges in South Dakota. SDDOT owns approximately 30% of these bridges and the remainder is owned by local governments. Of the 5,870 bridges, 1,208 are structurally deficient and 237 are functionally obsolete according to the Federal Highway Administration (FHWA 2012). Although 90% of State-owned bridges are not deficient, a large portion of bridges owned by local governments are either structurally deficient or functionally obsolete due to inadequate maintenance. The most common bridge type on SD local roads is the double-tee girder bridge with more than 700 currently in service. Bridges are designed for a service life of 75 years; however, many
of the current double-tee bridges in South Dakota are showing signs of deterioration or are in need of replacement after only 40 years in service.

The double tee bridges are used because of their ease of construction, reduced construction time, and relatively low cost. With only one supplier of double-tee bridges in the state, new alternative systems are needed to provide local governments more options when designing a new bridge. Alternative systems and suppliers allow local governments the ability to select the best system by comparing performance, availability, and cost of different options. The present study was carried out to investigate the feasibility of two types of glulam timber bridge systems that could be utilized in South Dakota.

1.2 Objectives and Scope of Work

This study and the study performed by Mingo (2016) are part of a project titled “Development of an Alternative to the Double Tee Bridge System” and funded jointly by SDDOT and MPC. The main objectives of the project are to: (1) identify or develop new bridge systems that can resist the AASHTO HL93 load requirements, can span up to 70 feet, and have a design life of at least 75 years, (2) perform ultimate and fatigue testing on the selected alternative bridge systems, and (3) compare cost, constructability, and performance of the selected alternative bridge systems with the existing double-tee girder bridges. Mingo (2016) developed a fully-precast single-span bridge system incorporating precast full-depth panels and inverted bulb-tee girders. The present study was performed to investigate the feasibility and performance of two types of glued laminated (glulam) timber bridges as alternatives to double-tee bridges.
An extensive literature review was performed to identify alternative bridge types that would be suitable for SD. The most suitable bridge types were ranked and presented to the SDDOT. Based on the ranking, the SDDOT selected three new bridge systems for testing. These included a full-depth deck panel precast concrete system, a glulam transverse deck on glulam girders system, and a glulam longitudinal deck system. This study is focused on the two glulam systems. Two full-scale bridge specimens representing the two glulam systems were constructed and instrumented, then tested under fatigue and ultimate loading to determine their performance. Fatigue loads were based on AASHTO LRFD (2013) to simulate traffic loading to which the bridges would be subjected to in the 75-year design life.

The test results were compared to those of double-tee systems. The comparison included performance, constructability, cost, and strength of each system. Design recommendations were developed based on the aforementioned parameters as well as the findings of the experiments.
This chapter includes a summary of findings of a literature review on the feasibility, performance and past application of two different glued laminated (glulam) timber bridge types.

### 2.1 Overview of Glulam Timber Bridges

Glulam timber bridges are constructed of glulam members manufactured from lumber laminations that are bonded together on their wide faces with waterproof structural adhesives. According to Ritter (1990), glulam is the most common material used for the fabrication of timber bridges because glulam members can be manufactured to any size and shape. In general, the span length of glulam bridges ranges from 20 to 80 feet, but construction of longer bridges with span lengths of 140 feet or longer is possible (Ritter, 1990). Important design and construction considerations such as design method, wearing surfaces, railing systems, and abutments are discussed in the following sections.

Wood is a renewable material that is readily available in South Dakota. Ritter (1990) also states that glulam timber bridges are very economical, light-weight, easy to fabricate, and environmentally friendly. Construction of glulam timber bridges is relatively simple and usually can be done without the need of highly skilled labor. Since they can be fabricated off-site and installed in place in a short period of time, glulam
bridges are suitable for accelerated bridge construction (ABC) and can be installed in most weather conditions.

Glulam timber bridges are not very common; therefore, data on the long-term performance of such bridges is scarce. Timber bridges will deteriorate rapidly if they are exposed to moisture for a long duration; therefore, frequent inspection and retreating are needed. Early detection of moisture is critical in extending the life of timber bridges (Ritter 1990).

![Rendering of a Glulam Timber Bridge](image1)

![Glulam Timber Bridge in Buchanan County, Iowa](image2)

Figure 2.1: Glulam Timber Bridges

### 2.2 Types of Glulam Timber Bridges

There are two main types of glulam timber bridges that have been used in the field: (1) longitudinal glulam deck bridges (Fig. 2.2a), and (2) transverse glulam deck bridges (Fig. 2.2b). The former type consists of glulam deck panels, which are typically 4-ft wide, spanning in the longitudinal direction of the bridge. These panels are held together by transverse stiffeners, which cannot be spaced more than 8 ft apart. The longitudinal glulam deck bridges can only span up to 38 feet (Wacker and Smith, 2001). The latter type of glulam bridge consists of transverse glulam deck panels supported by
stringers placed in the longitudinal direction of the bridge. The deck panels are typically 4-ft wide and the stringers typically have a spacing of 4 ft. These bridges typically span up to 80 feet.

![Glulam Timber Bridges](image)

(a) Longitudinal Glulam Deck  (b) Transverse Glulam Deck

**Figure 2.2: Glulam Timber Bridges**

### 2.3 Timber Bridge Structural Components

The material and various structural components of a glulam timber bridge including girders, deck panels, connections, and stiffeners are discussed herein.

#### 2.3.1 Glulam Materials

Timber is a nonhomogeneous and brittle material (Fig. 2.3). The strength of glulam timber is evaluated (rated) either mechanically or visually. Design values for typical glulam timber are specified in Chapter 8 of AASHTO LRFD (2013). However, other glulam materials and species are allowed by the code. These design values are then adjusted by correction factors as specified by AASHTO accounting for several parameters affecting the behavior of wood such as wet service conditions, temperature, member size, member volume, and load duration.
2.3.2 Girders (Stringers)

The size of glulam girders varies based on the bridge span length, the girder spacing, lamination species, and design loads. The nominal width of a glulam girder is typically ranges from 8 to 12 inches. The nominal depth of a glulam girder can vary from 12 to 60 inches. The lamination species is selected based on the availability of the material and the cost. Southern Pine is the most commonly used species in South Dakota.

2.3.3 Deck Panels

The deck panels for the girder bridges are typically 4-ft wide. A weaker species of wood may be used in the panels due to low stress demands. The panels should have a minimum nominal depth of 6 in. as required by AASHTO LRFD (2013). These panels
can be manufactured to cover either the full or half width of the bridge based on the preference of the owner. When full width deck panels are used, the slope of the bridge in the transverse direction shall only be one grade in order to avoid discontinuity.

The deck panels for the slab bridges are typically 4-ft wide as well. The strongest wood material available in the market is usually used due to high stress demands on the slab. The nominal depth of a deck panel varies from 6 to 16 inches. Note that nominal depths greater than 12 in. are not common thus they are more expensive.

2.3.4 Stiffeners

For glulam slab bridges, stiffeners are required to unify the deformation of the individual panels and to make the panels act as one system. One stiffener must be placed at the mid-span of the bridge, and the additional stiffeners should be placed no more than 8 ft apart on the remainder of the span length according to AASHTO LRFD (2013). AASHTO LRFD (2013) requires that the rigidity of a stiffener beam \((EI)\) shall not be less than 80,000 kip-in\(^2\) (AASHTO 9.9.4.3.1). Any size and material can be used as long as it satisfies the rigidity requirement.

2.3.5 Diaphragms

AASHTO LRFD (2013) currently specifies that either solid diaphragms (Fig. 2.4a) or steel cross braces (Fig. 2.4b) be installed on the timber bridges to improve the stability of the bridge. The feasibility and performance of glulam cross braces is investigated in the present study (Fig. 2.5).
2.3.6 Deck to Stringer Connections

Currently there are two methods that are primarily used to connect the deck panels to the stringers. One method is to install lag bolts from the top of the deck through the entire panel into the top of the beam as shown in Fig. 2.6. One of the disadvantages of using lag bolts is that these bolts are very large and must be field bored. Since, it is
very impractical to drill all of these holes before the pressure treatment, the bridge is more susceptible to decay due to water penetration. Furthermore, it is not possible to retighten these bolts if they loosen since the wearing surface will cover them all.

Another connection detailing is through the use of deck brackets as shown in Fig. 2.7. Aluminum brackets usually have small teeth that bite into a routed slot that is cut into the girder. The top portion of the bracket is then bolted into the deck. The deck bracket connection offers a tight connection that can be retightened if they loosen. They also do not affect the deck preservative treatment as lag bolts can be placed from the bottom of the deck. Downsides of this type of connection are that a large number of brackets are required and the slots require removal of a large volume of the wood. Furthermore, many holes are placed in the deck if bolts are installed from the top of the bridge.
The third deck-to-stringer connection that has been incorporated in the Cedar Rock Bridge in Buchanan County, Iowa is through the use of epoxy as shown in Fig. 2.8. Epoxy provides a strong bond between the deck and the stringer. This connection reduces the areas where water can seep into the deck as only small structural screws are needed to hold the deck panels to the stringers as the epoxy cures. The performance of this type of deck-to-stringer connection is investigated in the present study.
2.4 Timber Bridge Failure Modes

Glulam timber bridges fail in different modes under excessive loads. A timber member can fail in bending (Fig. 2.9), shear (Fig. 2.10), compression (Fig. 2.11), or the delamination of the layers (bond failure).

The most common failure mode for a glulam timber bridge is bending failure. Failure in shear is not very common but it can occur in short span bridges. Failure due to compression (compression face of beam, Fig. 2.10) is only a local failure and usually does not affect the overall performance of the bridge. If glue fails (bond failure), other major types of failure can occur due to the localized stresses (Franke et. al. 2015).
Figure 2.9: Bending Failure of Glulam Beam (Franke et. al. 2015)

Figure 2.10: Shear Failure of Glulam Beam (Franke et. al. 2015)
Figure 2.1: Compression Failure of Glulam Beam (Franke et. al. 2015)

Figure 2.12: Glulam Delamination (Franke et. al. 2015)
2.5 Long Term Performance of Glulam Timber Bridges

Ritter (1990) states that timber has been used as a bridge material for hundreds of years, but the application of treated timber was very rare until the early 1900s. Numerous untreated timber bridges performed well for the long term but the use of these types of bridges has recently declined since the naturally weather resistant North American wood species are no longer available in the size and quantity needed for bridge construction. Furthermore, it is no longer feasible or economical to cover the bridges for protection against moisture if the wood is untreated.

Brashaw et al. (2013) investigated the long-term performance of many different types of bridges including five glulam timber bridges (Table 2.1), which are located in Faribault County, Minnesota. Figures 2.13 to 2.17 show these five bridge conditions. The National Bridge Inventory (NBI) rating as well as the rating system for these glulam bridges are presented in Tables 2.2 and 2.3, respectively. It can be concluded that glulam timber bridges can last more than 60 years if they are maintained adequately.

| Table 2.1: Glulam Timber Stringer Bridges located in Minnesota (Brashaw et al., 2013) |
|---|---|---|---|---|---|
| Bridge ID | Year Built | Span (ft) | Average Daily Traffic | Width (ft) | Wearing Surface |
| 22508 | 1968 | 33.5 | 95 | 33.3 | Bituminous |
| 22514 | 1968 | 40 | 35 | 26 | Gravel |
| 22518 | 1969 | 38.5 | 70 | 33.1 | Gravel |
| 22519 | 1969 | 33.5 | 539 | 32 | Bituminous |
| 9967 | 1951 | 36.2 | 175 | 27.4 | Bituminous |

| Table 2.2: NBI Condition Rating (Brashaw et al., 2013) |
| NBI Condition Rating | Bridge Number |
|---|---|---|---|---|---|
| Deck | 22508 | 22514 | 22518 | 22519 | 9967 | Group Mean |
| Superstructure | 7 | 6 | 7 | 6 | 7 | 5.6 |
| | 7 | 7 | 7 | 6 | 5.8 |
### Table 2.3: NBI Condition Rating System

<table>
<thead>
<tr>
<th>FHWA - SI&amp;A Sheet Condition Rating Code</th>
<th>FHWA- SI &amp; A Sheet Condition Rating Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Not Applicable</td>
</tr>
<tr>
<td>9</td>
<td>Excellent Condition - New or like new condition.</td>
</tr>
<tr>
<td>8</td>
<td>Very Good Condition - No problems noted.</td>
</tr>
<tr>
<td>7</td>
<td>Good Condition - Some minor problems but no structural defects at critical locations (wood decay is a defect).</td>
</tr>
<tr>
<td>6</td>
<td>Satisfactory Condition - Structural elements show some minor defects and/or deterioration at critical locations. No measurable section loss.</td>
</tr>
<tr>
<td>5</td>
<td>Fair Condition - All primary structural elements are sound but may have minor to moderate defects and/or deterioration with measurable section loss at critical locations. No significant reduction in primary structural member load carrying capacity.</td>
</tr>
<tr>
<td>4</td>
<td>Poor Condition - Primary structural elements show moderate to serious defects, deterioration, corrosion, cracking, crushing, and/or scour. Advanced section loss at critical locations. Diminished load carrying capacity of members is evident.</td>
</tr>
<tr>
<td>3</td>
<td>Serious Condition - Serious and widespread defects have substantially reduced load carrying capacity of primary structural members. Local failures may be evident. Deflection/misalignment of members may be evident. Signs of severe structural stress are visible. Fatigue cracks in steel, shear cracks in concrete, and severe decay, checking, splitting, and crushing of beams or stringers in wood elements may be present.</td>
</tr>
<tr>
<td>2</td>
<td>Critical Condition - Advanced deterioration of primary structural elements. Defects have now resulted in significant local failures. Scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.</td>
</tr>
<tr>
<td>1</td>
<td>Imminent Failure Condition - Major deterioration or section loss present in critical structural components and/or obvious vertical or horizontal movements affecting structure stability. Bridge is/should be closed. However, corrective action may put bridge back in light service.</td>
</tr>
<tr>
<td>0</td>
<td>Failed Condition- Out of service. Beyond corrective action.</td>
</tr>
</tbody>
</table>
Figure 2.13: Glulam Timber Bridge No 22508 Located in Faribault County, Minnesota

Figure 2.14: Glulam Timber Bridge No 22514 Located in Faribault County, Minnesota
Figure 2.15: Glulam Timber Bridge No 22518 Located in Faribault County, Minnesota

Figure 2.16: Glulam Timber Bridge No 22519 Located in Faribault County, Minnesota
According to Ritter (1990), a wearing surface is the top layer placed on the bridge deck to form the road surface. The main purpose of a wearing surface is to improve safety, provide a smoother surface, improve skid resistance, and protect the deck. Typically, a wearing surface of a timber bridge can consist of (1) an asphalt overlay, (2) an asphalt chip seal, (3) sacrificial lumber covering the whole deck, (4) cover steel plates, (5) cover lumber planks, and (6) aggregate overlay. In the case that no wearing surface is used, routine inspections must be performed more often to ensure that the deck remains properly sealed to maintain the acceptable condition.

Asphalt is the most commonly used wearing surface as it provides a smooth and skid-resistant surface. Furthermore, asphalt provides a tight water-proof layer that protects the timber deck from abrasion. The only negative aspect of using asphalt is that reflective cracks can form allowing water to seep into the wood, which can decrease the
The service life of the bridge. Geotextile fabrics are highly recommended with an asphalt wearing surface to prevent this reflective cracking and to improve the bond between the glulam deck and the asphalt wearing surface. The asphalt also has to be maintained to prevent any moisture from reaching the deck. The asphalt approaches must be paved a minimum of 75 feet beyond the ends of the bridge in order to prevent the formation of potholes at the ends of the bridge. Examples of the asphalt wearing surface are shown in Figures 2.13, 2.16, and 2.17. All of the asphalt surfaces in the pictures have performed reasonably well. All of them have slight cracks or potholes and might need to be repaired.

Another commonly used wearing surface is the asphalt chip seal (Ritter, 1990). An asphalt chip seal is formed by placing a layer of aggregate onto liquid asphalt. Like the asphalt wearing surface the chip seal is smooth and skid-resistant. The big advantage of the asphalt chip seal compared to regular asphalt is that the chip seal is thinner and much more flexible than regular asphalt, reducing the amount of reflective cracking. Two 3/4-in. thick layers are recommended in order to seal the deck. Geotextile fabric is also recommended for an asphalt chip seal wearing surface.

The use of an aggregate wearing surface is not common. Two examples are given in Figures 2.14 and 2.15. Both bridges with the aggregate wearing surface are in reasonable condition. They might need some grading in the near future to remove the ruts from daily traffic.

The remaining wearing surface types were not recommended by Ritter (1990) as they can trap water. These types of wearing surfaces are typically used on very low volume roads. They should only be used when no other options are feasible.
An experimental wearing surface was recently used on the Cedar Rock Bridge in Buchanan County, Iowa (Fig. 2.18). The bridge deck was flooded with epoxy to fill all of the gaps in the wood and then very small rocks were imbedded into the epoxy in order to improve traction. The epoxy is typically applied in three layers with an approximately 3/8-in. thickness for each layer. The life of the epoxy depends on its exposure. This wearing surface was applied in 2015 and its performance cannot be fully evaluated at this time.

![Figure 2.18: Epoxy with embedded grit wearing surface](image)

2.7 Preservative Treatment of Wood

All wood used in the construction of glulam bridges must be treated with preservatives as required by AASHTO LRFD (2013, Article 8.4.3). Water repellents are used to slow the absorption of water and to keep the moisture content low, which helps prevent decay and slows the weathering process. Wood preservatives are used to prevent

Many types of treatments are available. Creosote was formerly the most popular option for treatment; however, it is no longer allowed to be used in bridge construction due to environmental and health concerns. Copper naphthenate is an oil-based preservative suitable for above-ground, ground contact, and freshwater applications but not in salt water. It is typically used in field treatments of cuts and holes. Copper naphthenate should not be used in areas of frequent human contact. Oxine copper is an oil-based preservative used in difficult-to-treat species such as Douglas-fir but is not very effective when the wood is in direct contact with the ground or water. Chromated copper arsenate (CCA) is a water-based preservative that was the most popular preservative from the late 1970’s until 2004. CCA was a good treatment choice for many applications. However, CCA is currently restricted by the United States Environmental Protection Agency (US EPA). Ammoniacal copper zinc arsenate (ACZA) is another water-based preservative. ACZA is good for most applications, but it accelerates fastener corrosion. Even with this drawback ACZA is still the required treatment chosen by many agencies. Alkaline copper quaternary (ACQ) is a water-based preservative that is an alternative to CCA. ACQ was recently developed, but it has been proven to be a viable and effective treatment material. ACQ also corrodes fasteners. Copper azole is another water-based alternative to CCA, which was developed recently. Copper azole also accelerates the corrosion of the fasteners.

Fire retardant treatments are generally not recommended by AASHTO LRFD (2013) as the large size of timber components used in bridge construction have inherent
2.8 Maintenance and Inspection Required for Glulam Timber Bridges

Since dry wood lasts longer than wood that has been exposed to moisture, it is necessary to perform routine maintenance to keep the wearing surface and other exposed areas in good condition. It is also highly recommended that timber bridges be inspected every 2 years and any wood that is exposed be retreated every 6 years (Ritter, 1990). Retreatment can be done by brushing on the preservative with a small brush.

There are four main types of defects that can lead to decay and deterioration of wood: (1) checks, (2) splits, (3) shakes, and (4) knots. A check is a separation of the wood fibers that occurs across the annual growth rings and parallel to the grain direction. A split is an advanced check that extends completely through the piece of wood. A shake is a separation of the wood fibers parallel to the grain between the annual growth rings. A knot is a separation of the wood fibers due to the trunk growing around an embedded limb (FHWA Bridge Inspector’s Reference Manual, 2012).
The primary cause of timber bridge deterioration is decay due to fungi. Fungi will grow in timber if (1) sufficient oxygen is available (at least twenty percent of the volume of wood must be occupied by air for fungi to become active), (2) temperature is moderate to high (maximum growth occurs between 75°F and 85°F), (3) an adequate food supply exists (e.g. untreated wood), and (4) moisture is high (fungi does not grow if the moisture content is below twenty percent but growth occurs rapidly above twenty-five percent). The fungi that cause the most damage are white rot and brown rot as shown in Fig. 2.20 (FHWA Bridge Inspector’s Reference Manual, 2012).
Insects can also cause major damage in wood structures. The most common types include termites, powder-post beetles, carpenter ants, and caddisflies (Fig. 2.21).

Termites can be very destructive without any signs at the surface. The only visible sign of infestation is the white mud shelter tubs that extend from the ground to the wood. These infestations are rare as the constant vibration of the bridge due to traffic acts as a deterrent. Powder-post beetles hollow out the inside of wood structures leaving small holes all over the outer surface. Carpenter ants destroy wood that is soft and decaying. An accumulation of sawdust on the ground indicates their presence. Caddisflies do not eat the wood, but dig small holes in areas where decay is present for shelter (FHWA Bridge Inspector’s Reference Manual, 2012).
Other sources of deterioration that could cause damage are chemical spills (acids or alkalis), loose connections, fire, vehicle impacts, vehicle wearing of the deck, abrasion due to tides, mechanical wear of fastener holes, overstress, and weathering (FHWA Bridge Inspector’s Reference Manual, 2012).

According to Ritter (1990), several methods can be utilized for timber bridge inspection. Visual inspection is the most convenient method in which an inspector looks over the bridge for signs of deterioration, decay, mold, fungi, insect activity, or any other abnormal changes in the wood. Probing is another inspection method usually performed with the visual inspection. A moderately pointed tool is incorporated to locate any soft spots in the wood. The third and the most common inspection method for wood is to use sounding in which the bridge inspector strikes the wood with a hammer or another object.
The inspector can determine if there is decay by listening to the sound feedback. If decay is suspected, the inspector then must drill or core the area for further inspection. If decay is found, a plan of action must be made to fix the distressed region.

Preventative maintenance is crucial for long-term serviceability of timber bridges. For example, resealing the exposed wood can prevent decay and deterioration by keeping the moisture out. Remedial maintenance should be performed when decay is present. However, remedial maintenance is only applicable where the distress is not severe enough to affect the overall performance of the bridge. In this case, a small section of a timber bridge can be replaced. Major maintenance is usually performed when deterioration results in strength degradation. In this case, a few members of the bridge have to be replaced to increase the bridge load-carrying capacity. When the deterioration is severe, the bridge has to be replaced.

2.9 Railing Systems

A bridge railing system must be positioned to safely contain an impacting vehicle without allowing it to pass over, under, or through the rail elements. Furthermore, a proper railing system must be free of features that may catch on the vehicle or cause it to overturn or decelerate too rapidly.

Any crash-tested railing configuration or any railing designed according to AASHTO LRFD (2013, Article 13.7) can be used for timber bridges. The rail material can be timber, metal, or concrete. One example of a timber railing is shown in Fig. 2.22.
2.10 Timber Bridge Abutments

Many studies stated that existing abutment detailing can be used for glulam timber bridges. Timber bridge abutments are typically constructed using either timber or concrete as shown in Fig. 2.23. The connections should be designed to resist appropriate design loads.
2.11 Timber Bridge Fabrication

One of the advantages of glulam timber bridges is that they can be completely prefabricated offsite and then shipped to the project site for installation (Fig. 2.24). This feature is in-line with accelerated bridge construction (ABC), which has been recently emphasized in the US. For wide timber bridges, the bridge can be prefabricated in segments of one or two lanes to be shipped and assembled onsite.

For onsite construction of glulam girder timber bridges, assembly is typically started with the placing of the center girder followed by the placing of the other girders working outwards. Subsequently, the deck panels are placed then curbs and railings are installed. Once the bridge superstructure is completed, the substructure backwalls can be placed and the approach can be backfilled. The last step is the wearing surface. The entire construction process for a 60-ft bridge can be completed in 60 hours or less (Ritter 1990). The construction time for prefabricated timber bridges is expected to be significantly less than that for timber bridges built onsite.
Figure 2.24: Erie Canal Bridge Being Placed in Port Byron, NY in 2014 (laminatedconcepts.com)
3. TRANSVERSE GLULAM DECK ON GLULAM STRINGER BRIDGE TEST SPECIMEN

Two types of glulam timber bridges were introduced in the previous chapter: (1) bridges built with transverse glulam decks supported on glulam stringers (referred to as “girder bridges” hereafter”), and (2) longitudinal glulam deck bridges (referred to as “slab bridges”). The long-term structural performance and the limit state design requirements were evaluated for both bridge types through full-scale experiments. This chapter includes the design, construction, instrumentation, test setup, and loading protocols for a full-scale girder bridge test specimen.

3.1 Design of Girder Bridge Test Specimen

The prototype girder bridge was assumed to be 50-ft long and 34.5-ft wide (Fig. 3.1). A full-scale bridge model was selected for testing but with a width approximately equal to the width of one lane of traffic. The bridge test specimen (Fig. 3.2) consisted of (1) three 50-ft long girders with a depth of 30.25 in. and a width of 8.5 in., (2) thirteen deck panels each 48-in. long (in the longitudinal direction of the bridge), 110.75-in. wide
(in the transverse direction of the bridge), and 5.5-in. thick, and (3) ten rectangular glulam cross braces each with a dimension of 5 by 10 in. to improve the stability of the bridge.

AASHTO LRFD (2013) was used in the design of the bridge components. This bridge was designed for the HL-93 loading which consists of a design truck or tandem
accompanied by the design lane load. The design truck (Fig. 3.3) consists of a pair of 32-kip axle loads and an 8-kip front axle load. The spacing of the 32-kip axle loads is varied between 14 and 30 ft to produce the highest demand. The design tandem consists of two 25-kip axle loads spaced 4-ft apart as shown in Figure 3.4. The design lane load consists of a distributed load of 0.64 kips per linear foot over a 10-ft width of the bridge.

### 3.1.1 Design of Deck Panels

The deck panels were analyzed and designed according to AASHTO LRFD (2013). A structural analysis was performed under the Strength I limit state with the assumptions that the deck panels are continuous beams and that the girders are pin supports. According to the structural analysis, the deck could be less than 6 in.
However, the depth of the deck was controlled by the minimum nominal thickness of 6 in. required by AASHTO. The width of the deck panels was determined to be 4 ft to ease the fabrication and installation. Note that the deck panels can be sufficiently large to span the whole width of the bridge.

### 3.1.2 Design of Girders

The girders were also designed according to AASHTO LRFD (2013). Live load distribution factors were used to calculate the moment demand for an interior girder since the girders used in the test bridge simulate interior girders of the prototype bridge model. The distribution factor can be found using Eq. 3.1. The girders were designed for the Strength I Limit State load combination (Eq. 3.2). The moment capacity was calculated using Eq. 3.4 through 3.8.

\[
LLDF = S / 10 \tag{Eq. 3.1}
\]

\[
M_{\text{int}} = 1.25DC + 1.5DW + 1.75LL \tag{Eq. 3.2}
\]

\[
M_n = LLDF * M_{\text{int}} \tag{Eq. 3.3}
\]

\[
M_n = F_b SC_L \tag{Eq. 3.4}
\]

\[
C_L = \frac{1 + A}{1.9} - \sqrt{\frac{(1 + A)^2}{3.61} - \frac{A}{0.95}} \tag{Eq. 3.5}
\]

\[
A = \frac{F_{bE}}{F_b} \tag{Eq. 3.6}
\]

\[
F_{bE} = \frac{K_{bb} E}{R_B^2} \tag{Eq. 3.7}
\]

\[
R_B = \sqrt{\frac{L_d}{b^2}} \leq 50 \tag{Eq. 3.8}
\]
where $LLDF$ is the Live load distribution factor, $S$ is the spacing of the girders, $DC$ is the dead load of structural components and nonstructural attachments, $DW$ is the dead load of wearing surfaces and utilities, $LL$ is the vehicular live load, $M_{int}$ is the maximum moment of an interior girder, $M_u$ is the moment demand, $M_n$ is the moment capacity, $F_b$ is the flexural design strength, $S$ is the section modulus, $C_L$ is the beam stability factor, $K_{be}$ is 1.1 for glulam, $E$ is the adjusted modulus of elasticity.

The girders were assumed to be partially composite with the deck and were designed based on the mechanical properties for 26F-1.9E Southern Yellow Pine (Table 3.1). The girder final size was 30.25-in. deep and 8.5-in. wide. To provide sufficient bearing area for girders at the ends and to have 50-ft clear span, the girder length was increased from 50 to 52 ft.

### Table 3.1: Mechanical Properties of Glulam Timber

<table>
<thead>
<tr>
<th>Properties</th>
<th>Notation</th>
<th>Unit</th>
<th>26F-1.9E</th>
<th>24F-2.0E</th>
<th>M-29</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Zone Stressed in Tension</td>
<td>$F_{bxo}$</td>
<td>ksi</td>
<td>2.6</td>
<td>2.4</td>
<td>1.55</td>
</tr>
<tr>
<td>Compression Zone Stressed in Tension</td>
<td>$F_{bco}$</td>
<td>ksi</td>
<td>1.95</td>
<td>1.45</td>
<td>1.55</td>
</tr>
<tr>
<td>Shear Parallel to Grain</td>
<td>$F_{vgo}$</td>
<td>ksi</td>
<td>0.265</td>
<td>0.265</td>
<td>0.175</td>
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<tr>
<td>Modulus of Elasticity</td>
<td>$E_{xo}$</td>
<td>ksi</td>
<td>$1.9 \times 10^3$</td>
<td>$2.0 \times 10^3$</td>
<td>$1.7 \times 10^3$</td>
</tr>
</tbody>
</table>

To further aid bridge designers, a spreadsheet was developed for the design of glulam girders checking the capacity to demand ratio for all design parameters as well as the girder deflection.

### 3.1.3 Design of Cross Braces

The cross braces were designed to resist lateral loads according to AASHTO LRFD Section 8.11 (AASHTO LRFD, 2013). Even though solid diaphragms and steel
cross braces are recommended by AASHTO, the use of glulam cross braces was proposed and investigated in this project due to the ease of construction. The final glulam rectangular cross braces were 6.875-in. wide and 8-in. deep.

3.1.4 Design of Deck-to-Stringer Connection

As discussed in the previous chapters, there are three main types of deck-to-stringer connections: (1) lag bolt connection, (2) aluminum bracket connection, and (3) epoxy connection. The use of the epoxy to connect the deck to the stringers was proposed and the connection performance was evaluated in the present study. The deck panels were attached to the girders by a layer of epoxy (Fig. 3.5). This connection type is better compared to other types due to minimal drilling on the top of the deck. Note that some screws were utilized to compress the deck to the girder to activate the epoxy.
3.2 Fabrication, Assembly and Transportation of Test Specimen

The entire test bridge was fabricated by a manufacturer then shipped as one piece to the Lohr Structures Laboratory. The following sections discuss the fabrication, assembly and transportation of the test specimen.

3.2.1 Fabrication of Deck Panels

The glulam deck panels were built from M-29 Southern Yellow Pine. Thirty-five 1.375-in. thick laminations were glued together to form the deck panels (Fig. 3.6a). Each panel was clamped to apply pressure and to activate the epoxy between the laminations. This type of epoxy does not activate until a minimum pressure of 150 psi is applied. The panels were stored in the construction facility with ambient room temperature to allow the epoxy to dry and harden. After epoxy hardening, the panel edges were grooved and routed to form a male-female connection as shown in Fig 3.6b.
3.2.2 Fabrication of Girders

The girders were specified to be built using 26F-1.9E Southern Yellow Pine. However, the ultimate testing showed that a wrong type of wood (24F-2.0E) was used in the fabrication process by mistake. This issue will be discussed later under the testing results. Twenty-two 1.375-in. thick laminations were glued together to form the girders (Fig. 3.7). Each girder was clamped to apply pressure and to activate the glue after placing the epoxy between the laminations. The girders were placed in the construction area with an ambient room temperature to allow the epoxy to dry and harden.

![Figure 3.7: Finished Test Girders](image)

3.2.3 Fabrication of Cross Braces

The cross braces were specified to be built with 26F-1.9E Southern Yellow Pine as well. However, they were also built with 24F-2.0E by mistake. The cross braces were cut and prepared with high precision (Fig. 3.8) to easily fit between the stringers.
3.2.4 Assembly of Bridge Test Specimen

The test specimen was completely assembled at the manufacturing site then shipped to the Lohr Structures Laboratory at SDSU for testing. First the girders were placed beside each other (Fig. 3.9a) then the cross braces were installed in between. Epoxy was placed between the cross braces and the girders before lag bolts were installed. After completion of the diaphragms, the first deck panel was placed at the south end of the specimen. The panel was held in the air by a fork lift while the epoxy was placed on the top of the girders (Fig. 3.9b). Long screws were then installed to hold the panel in place and to allow the epoxy to cure (Fig. 3.9c). The next panel was installed with the same method but was placed with care to make sure that the panel to panel connection was adequate. A thin line of epoxy was placed on the male connection before the second panel was in place (Fig. 3.9d to 3.9f). This process continued until the deck system was completed.
3.2.5 Transportation of Test Specimen

The test specimen was transported from the manufacturer site in Tea, SD to the Lohr Structures Lab at SDSU in one piece. The bridge was loaded onto a truck using two fork lifts. Upon arrival at the lab, the truck backed as far into the lab as possible. Two
straps were placed approximately 20 ft apart. Wooden blocks were installed at these points to keep the straps in place and to avoid stressing the deck during lifting. The straps were hooked to a large chain that was connected to a 15 ton crane. The test specimen was then lifted and the truck drove away. Finally, the abutments (reaction blocks) were placed and the test specimen was dropped in place.

![Figure 3.10: Transportation of Girder Bridge Test Specimen](image)

(a) Arrival of the Test Specimen  
(b) Placement of the Straps  
(c) Lifting of the Test Specimen  
(d) Truck Driving Away

3.3 Test Setup

The girder bridge test specimen was tested under two different loading scenarios: (1) fatigue loading and (2) ultimate (strength) loading. The test setup for the two test procedures were slightly different and are discussed herein.
3.3.1 Fatigue Test Setup

The bridge test specimen girders were supported on three reaction blocks at each end (Fig. 3.11). The reaction blocks at the North end were 28.5-in. tall while the reaction blocks at the South end were 4.5 in. shorter (24-in. tall) to allow for placement of load cells. Rectangular neoprene bearing pads each with a dimension of 6 by 12 in. were placed under each girder. The length of the pad was based on the AASHTO LRFD requirements. Two 22-kip actuators were utilized to apply the load at the midspan at 6.6 in. from the inside edge of the exterior girders. The location of the point loads was selected to produce equal reactions in the three girders. The load frame used to support the actuators had a height of 20 ft and a clear spacing of 10 ft between the columns.
Figure 3.11: Fatigue Test Setup for Girder Bridge
3.3.2 Ultimate (Strength) Test Setup

For the ultimate test, a 328-kip actuator was utilized to monotonically apply the load at the midspan of the bridge. The load was distributed directly to the three girders using a spreader beam as shown in Fig. 3.12

Figure 3.12: Ultimate Test Setup

3.4 Instrumentation

The bridge test specimen was instrumented with strain gauges, linear voltage differential transformers (LVDTs), load cells, and string potentiometers (string pots) to measure response of the bridge at different load levels. Note that actuators also provide load and displacement data at the location of the applied load. This section presents the bridge instrumentation detailing.
### 3.4.1 Strain Gauges

Figure 3.13 shows the strain gauge plan for the test bridge. Strain gauges were installed only at the midspan where the bending moment was maximum. Three strain gauges were installed on the interior girder, two gauges were placed on the exterior girders. Additionally, 16 strain gauges were installed on the top of the deck to investigate the effective width of the deck for composite design. Wood strain gages (PFL-30-11-5L) each with a length of 30 mm (1.18 in.) were utilized in this project.

![Figure 3.13: Strain Gauge Locations](image)

To install strain gauges (Fig. 3.14), the wood surface was grinded, sanded then cleaned with acetone. Since wood surface is naturally porous, an adhesive (PS type) with 0.5-mm (0.02-in.) thickness was placed to form the base for the gauge. Finally, when the base material was hardened, the strain gauge was installed.
3.4.2 Linear Variable Differential Transformers (LVDTs)

Fourteen LVDTs were used to record displacements and rotations at various locations of the bridge (Fig. 3.15 and 3.16). Since the girders were placed on bearing
pads, which compress under applied load, vertical LVDTs were installed at the end of each girder to measure the deformation of the pads and to calculate the net midspan deflection (Fig. 3.16b). Three additional vertical LVDTs were installed at the midspan to measure the girder deflections (Fig. 3.15). Six horizontal LVDTs were used to measure either the relative displacements of the joints or the rotations.

Figure 3.15: LVDT Installation Plan for Fatigue Testing
For the ultimate testing, the LVDT installation plan was slightly modified as shown in Fig. 3.17. Since large displacements were expected under the ultimate testing, the LVDTs at the midspan of the bridge were removed (due to small measuring range) and placed in other locations. For example, VD-4 in Fig. 3.15 was removed and placed as HD-7 in Fig. 3.17. HD-7 was placed at the bottom of the deck to measure the opening of the joint. Note that VD-5 and VD-6 were removed at 1.25-in. displacement to avoid damage of the device.
3.4.3 String Potentiometers (String POTs)

Since string pots usually have a larger measuring range than LVDTs, they were used at the midspan of the bridge during the ultimate test to measure deflections. The string pots were installed at the centerline of girders at their bottom face (Fig. 3.18).
3.4.4 Load Cells

Load cells were placed under each of the three girders at the South end to measure the support reactions (Fig. 3.19). It was assumed that the reactions at both ends of the girders were equal because the load was applied at the midspan.
3.4.5 Data Acquisition System

All of the instrumentation were connected to a 7000-128-SM data acquisition system with 128 channels. A scan rate of 10 readings per second was used for the monotonic loading and a scan rate of 100 readings per second was used for cyclic loading.

Figure 3.19: Load Cell at Girder Ends
3.5 Test Procedure

The bridge specimen was tested under two loading scenarios: (1) fatigue and (2) ultimate. Fatigue testing was performed to investigate the performance of the bridge under 75 years of service life and the ultimate testing was carried out to determine the capacities of the bridge. The test procedures are described in detail herein.

3.5.1 Fatigue Testing

Phase I of the bridge testing consisted of fatigue testing. Two 16-kip point loads were cyclically applied at the midspan of the bridge (Fig. 3.11).

The fatigue loading protocol was determined using AASHTO LRFD Fatigue II Limit State specifications. The fatigue truck specified by AASHTO LRFD (2013) has an 8-kip front axle and two 32-kip mid- and rear axle loads. The spacing between the 8-kip
front axle and the middle 32-kip axle is 14 ft and the spacing between the two 32-kip
axles is 30 ft. The maximum moment resulting from the fatigue truck was 444 kip-ft at
the midspan. The AASHTO LRFD (2013) live load distribution factor (LLDF) was used
to determine the moment for interior girders. The LLDF was calculated to be 0.4. Note
that AASHTO does not specify a dynamic load allowance factor for timber bridges.
Furthermore, a load factor of 0.75 was used for the Fatigue II Limit State. For the fatigue
testing, the amplitude of the point load at the midspan was determined using Eq. 3.9 and
3.10 to produce a moment equivalent to that required by the AASHTO fatigue limit state
demand.

\[ M_{int} = (\gamma _i)(LLDF)(M_{max}) \]  
\[ P_{actuator} = (#Girders)(\frac{M_{int} * 4}{L}) \]

where:

\( \gamma _i \) = The Load factor

LLDF= The Live load distribution factor

\( M_{max} \) = The maximum moment demand resulting from a design vehicle at the
critical location

The number of the fatigue loading cycles was determined to be 410,625 cycles
based on an average daily truck traffic (ADTT) of 15% for the 75 years of the design life.
The total was increased to 500,000 cycles for the fatigue testing to account for increased truck traffic with time. Force-based controlled cyclic loads were applied at a frequency of 0.7 Hz. The lower bound of the applied load during the fatigue testing was 300 lbs to prevent the actuator from uplifting.

![Fatigue Loading Protocol](image)

**Figure 3.21: Fatigue Loading Protocol**

Stiffness tests were performed at an interval of 50,000-load cycles including an initial stiffness test. The stiffness load amplitude was 30 kips. The load was applied under a displacement-based control condition and a displacement rate of 0.007 in./sec.

### 3.5.2 Ultimate Testing

After completion of the fatigue testing, an ultimate test was carried out to determine the capacity of the bridge and to investigate the failure mode. A point load was applied at the midspan of the bridge. The specimen was loaded under a monotonic
displacement-controlled protocol to failure with a displacement rate of 0.007 in./sec. The data was recorded after completion of each displacement step. The displacement step was 0.05 in. up to a displacement of 1.30 in. then the displacement step was increased to 0.1 in. to the end of the testing.
Two types of glulam timber bridges were introduced in chapter 3: (1) bridges built with transverse glulam decks supported on glulam stringers (referred to as the “girder bridges”), and (2) longitudinal glulam deck bridges (referred to as “slab bridges” hereafter). The long-term structural performance as well as the limit state design requirements were evaluated for both bridge types through full-scale experiments. This chapter includes the design, construction, instrumentation, test setup, and loading protocols for a full-scale slab bridge test specimen.

4.1 Design of Bridge Test Specimen

Girder bridges can span up to 30 ft and can cover several lanes of traffic. The prototype girder bridge selected in the present study was assumed to be 16.5-ft long and 34.5-ft wide (Fig. 4.1). The length was selected based on the manufacturer limitations in producing deeper slabs, and the width is typical for two lanes of traffic sufficient for local roads. A full-scale bridge model was selected for testing but with a width approximately equal to the width of one lane of traffic. The bridge test specimen (Fig. 4.2) consisted of (1) two 20-ft long longitudinal deck panels with a depth of 10.75 in. and a width of
48.125 in., and (2) three stiffeners each 7.5-ft long (in the transverse direction of the bridge), 5-in. wide (in the longitudinal direction of the bridge), and 5.5-in. thick. The panels were connected to the stiffeners using two 0.75-in. diameter lag bolts per panel as shown in Figure 4.3.
AASHTO LRFD (2013) was used in the design of the bridge components. The bridge was designed for the HL-93 loading which consists of a design truck or tandem accompanied by the design lane load. The design truck (Fig. 4.4) consists of a pair of 32-kip axle loads and an 8-kip front axle load. The spacing of the 32-kip axle loads is varied between 14 and 30 ft to produce the highest demand. The design tandem consists of two 25-kip axle loads spaced 4-ft apart as shown in Figure 4.5. The design lane load consists of a distributed load of 0.64 kips per linear foot over a 10-ft width of the bridge.
4.1.1 Design of Deck Panels

The deck panels were analyzed and designed according to AASHTO LRFD (2013). Wheel load fractions (Eq. 4.1) were used to calculate the moment demand for each deck panel. The deck panels were designed for the Strength I Limit State (Eq. 4.2). The moment capacities were calculated from Eq. 4.3 to 4.8. The deck panels were designed using mechanical properties for 24F-2.0E Southern Yellow Pine. The final design led to 10.75-in. deep, 48.125-in. wide, and 16.5-ft long panels.

\[
WLF = \frac{b}{4.25 + L/28} \quad \text{(Eq. 4.1)}
\]

\[
M_{\text{panel}} = 1.25DC + 1.5DW + 1.75LL \quad \text{(Eq. 4.2)}
\]

\[
M_u = LLDF \ast M_{\text{int}} \quad \text{(Eq. 4.3)}
\]

\[
M_n = F_bSC_L \quad \text{(Eq. 4.4)}
\]

\[
C_L = \frac{1 + A}{1.9} - \sqrt{\frac{(1 + A)^2}{3.61} - \frac{A}{0.95}} \quad \text{(Eq. 4.5)}
\]
\[ A = \frac{F_{bE}}{F_b} \]  
(Eq. 4.6)

\[ F_{bE} = \frac{K_{bE}E}{R_B^2} \]  
(Eq. 4.7)

\[ R_B = \sqrt{\frac{L_d L}{b^2}} \leq 50 \]  
(Eq. 4.8)

where:

- \( WLF \) is the wheel load fraction,
- \( b \) is the width of the deck panel,
- \( DC \) is the dead load of structural components and nonstructural attachments,
- \( DW \) is the dead load of wearing surfaces and utilities,
- \( LL \) is the vehicular live load,
- \( M_{\text{panel}} \) is the maximum moment of an interior deck panel,
- \( M_u \) is the moment demand,
- \( M_n \) is the moment capacity,
- \( F_b \) is the flexural design value,
- \( S \) is the section modulus,
- \( C_L \) is the beam stability factor,
- \( K_{bE} \) is 1.1 for glulam,
- \( E \) is the adjusted modulus of elasticity.

To further aid the designers, a spreadsheet was developed to find the capacity to demand ratio of the slab bridge in flexure, shear, compression, and tension. The spreadsheet also checks the deflection of the bridge for different scenarios.

### 4.1.2 Design of Transverse Stiffeners

The transverse stiffeners were also designed according to AASHTO LRFD (2013). One stiffener must be placed at the midspan and the spacing of the stiffeners cannot exceed 8 feet. According to AASHTO, the strength of a stiffener based on the adjusted modulus of elasticity \( (E') \) times the moment of inertia \( (I) \) must be greater than
80,000 k-in². The stiffeners were designed using 24F-2.0E Southern Yellow Pine. The final design led to 5.5-in. deep, 5-in. wide, and 7.5-ft long stiffeners.

4.1.3 Design of Deck-to-Stiffener Connections

The deck panels were attached to the stiffeners using 0.75-in. diameter lag bolts, each 12-in. long. Two bolts were used per panel. Initially epoxy was considered for use along the length of the stiffener in addition to the bolts, but the epoxy was deemed unnecessary for this connection. The spacing of the bolts was shown in Fig. 4.3.

4.2 Fabrication, Assembly and Transportation of Test Specimen

The entire test bridge was fabricated in Tea, South Dakota then shipped as one piece to the Lohr Structures Laboratory at South Dakota State University (SDSU). The following sections discuss the fabrication, assembly, and transportation of the test specimen.

4.2.1 Fabrication of Deck Panels

The glulam deck panels were built from 24F-2.0E Southern Yellow Pine. Thirty-five 1.375-in. thick laminations were glued together to form one deck panel. The laminations were clamped together to apply pressure and to activate the epoxy between the laminations. This type of epoxy does not activate until a minimum pressure of 150 psi is applied. The panels were stored in the construction facility with ambient room temperature to allow the epoxy to dry and harden.
4.2.2 Fabrication of Stiffeners

The stiffeners were also made from 24F-2.0E Southern Yellow Pine. Four 1.375 inch thick laminations were glued together to form each stiffener. After the epoxy was placed between the laminations, the panel was clamped to apply pressure. The stiffeners were then stored in ambient room temperature until the epoxy dried and hardened.

4.2.3 Transportation of Test Specimen

The test specimen was transported from the manufacturer site in Tea, SD to the Lohr Structures Laboratory at SDSU in Brookings, SD on a trailer pulled by a pickup truck. In Tea, the deck panels were loaded onto the trailer by a fork lift and the stiffeners were placed in the truck bed. Upon arrival at the Lab, the trailer backed as far into the lab as possible. Two straps were placed around the panels to lift them. The straps were hooked to a 15-ton crane. The panels were then lifted and placed on the reaction blocks.

4.2.4 Assembly of Test Specimen

The test specimen was assembled in the Lohr Structures Laboratory. First the deck panels were placed beside each other on the reaction blocks then shimmed up to have continuous support. Subsequently, the stiffeners were installed from the underside of the deck. The center stiffener was installed first then the other two were bolted to the deck. A pilot hole was initially drilled in the stiffener then the lag bolts were screwed 6.5 in. into the deck from the bottom.
4.3 Test Setup

The slab bridge test specimen was tested under two different loading scenarios: (1) fatigue loading, and (2) ultimate (strength) loading. The test setup for the two test procedures were slightly different and are discussed herein.

4.3.1 Fatigue Test Setup

The bridge test specimen was continuously supported on two reaction blocks at each end (Fig. 4.6). A continuous neoprene bearing pad was used at each end between the panel and the abutment to allow the specimen to rotate freely. Two 22-kip actuators were utilized to apply the load at the center of the panel at the midspan. The load frame used to support the actuators has a height of 20 ft and a clear spacing of 10 ft between the columns.
4.3.2 Ultimate Strength Test Setup

For the ultimate test, a 328-kip actuator was utilized to monotonically apply the load at the midspan of the bridge. The load was equally distributed to the two panels using a spreader beam as shown in Fig. 4.7.
4.4 Instrumentation

The bridge test specimen was instrumented with strain gauges, linear voltage differential transformers (LVDTs), load cells, and string potentiometers (string pots) to measure response of the bridge at different load levels. Note that the actuators also provide load and displacement data at the location of the applied load. This section presents the bridge instrumentation detailing.

4.4.1 Strain Gauges

Each panel was instrumented with three strain gauges on the side surface to measure the strains at different depths of the panel (Fig. 4.8). Furthermore, two additional strain gauges were installed on the top and bottom of the deck 6 in. away from the bridge longitudinal centerline. All deck panel strain gauges were offset 6 in. from the bridge transverse centerline to avoid interfering with the stiffener. The center stiffener will also be instrumented with 5 strain gauges measuring the strain in the transverse
direction (Fig. 4.8c). Wood strain gauges (PFL-30-11-5L) each with a length of 30 mm (1.18 in.) were utilized in this project.

(a) Bridge Plan View

(b) Section 1-1 (Deck Panel Strain Gauges in Longitudinal Direction of Bridge)

(c) Section 2-2 (Stiffener Strain Gauges in Transverse Direction of Bridge)

Figure 4.8: Strain Gauge Locations
To install strain gauges (Fig. 4.10), the wood surface was grinded, sanded then cleaned with acetone. Since wood surface is naturally porous, 0.5-mm (0.02-in.) thick adhesive (PS type) was placed to form the base for the gauge. Finally, the strain gauge was installed when the base material was hardened.

![Images of strain gauge installation]

(a) Preparing First Layer of Epoxy  
(b) Letting Epoxy Harden  
(c) Placement of Strain Gauge  
(d) Protecting Gauges

Figure 4.9: Strain Gauge Installation

4.4.2 Linear Variable Differential Transformers

Fourteen LVDTs were used to record displacements and rotations at various locations of the bridge (Fig. 4.10). Since the girders were placed on bearing pads, which compress under applied load, vertical LVDTs were installed at the end of each girder to measure the deformation of the pads and to calculate the net midspan deflection (Fig.
4.11a). Two additional vertical LVDTs were placed at the midspan under the stiffener to measure the deflection as shown in Fig. 4.11b. Two more vertical LVDTs were placed 4.5 in. away from the midspan on the deck panels (Fig. 4.11c). Six horizontal LVDTs were used to measure the slippage, relative displacements, and rotations. HD-1 was installed to measure the slippage between the deck panels in the longitudinal direction (Fig. 4.11d). HD-2 was used to measure the relative transverse displacement of the deck panels (Fig. 4.11e). HD-3 was installed to measure the slippage between the deck panels and the stiffener (Fig. 4.11f). Two rotational LVDTs were installed above and below the longitudinal joint to measure the joint rotation (Fig. 4.11g and 4.11h).

Figure 4.10: LVDT Installation Plan for Fatigue Testing
(a) Vertical LVDT to Measure Compression of Neoprene Pad
(b) Vertical LVDT Directly Under Stiffener at Midspan
(c) Vertical LVDT Offset From Midspan to Measure Deck Deflection
(d) Horizontal LVDT Measuring Slippage Between Deck Panels
(e) Horizontal LVDT Measuring Gap Between Deck Panels
(f) Horizontal LVDT Measuring Slippage Between Stiffener and Deck Panels
(g) Horizontal LVDTs Measuring Rotation
(h) Horizontal LVDTs Measuring Rotation

Figure 4.11: LVDT Installation Plan
4.4.3 String Potentiometers (String POTs)

Since string pots usually have a larger measuring range than LVDTs, they were used at the midspan of the bridge during the ultimate test to measure deflections. The string pots were installed at the centerline of the girders at their bottom face (Fig. 4.12).

![Figure 4.12: String Pots](image)

4.4.4 Data Acquisition System

All of the instrumentation were connected to a 7000-128-SM data acquisition system with 128 channels. A scan rate of 10 readings per second was used for the monotonic loading and a scan rate of 100 readings per second was used for cyclic loading.
4.5 Test Procedure

The bridge specimen was tested under two loading scenarios: (1) fatigue and (2) ultimate. Fatigue testing was performed to investigate the performance of the bridge under 50 years of service life and the ultimate testing was carried out to determine the capacities of the bridge. The test procedures are described in detail herein.

4.5.1 Fatigue Testing

Phase I of slab bridge testing was fatigue loading. Two 11-kip point loads were cyclically applied at the midspan of the bridge (Fig. 4.6).

The fatigue loading protocol was determined using AASHTO LRFD Fatigue II Limit State specifications. The fatigue truck specified by AASHTO LRFD (2013) has an 8-kip front axle and two 32-kip mid- and rear axle loads. The spacing between the 8-kip
front axle and the middle 32-kip axle is 14 ft and the spacing between the two 32-kip axles is 30 ft. The maximum moment resulting from the fatigue truck was 49.5 kip-ft at the midspan. The wheel load fraction was used to determine the moment for interior deck panels. The LLDF was calculated to be 0.924. Note that AASHTO does not specify a dynamic load allowance factor for timber bridges. Furthermore, a load factor of 0.75 was used for the Fatigue II Limit State. For the fatigue testing, the amplitude of the point load at the midspan was determined using Eq. 4.9 and 4.10 to produce a moment equivalent to that required by the AASHTO fatigue limit state demand.

\[
M_{\text{int}} = (\gamma_i)(WLF)(M_{\text{max}}) \quad \text{(Eq. 4.9)}
\]

\[
P_{\text{actuator}} = (#\text{Panels})\left(\frac{M_{\text{int}} \ast 4}{L}\right) \quad \text{(Eq. 4.10)}
\]

where:

\[
\gamma_i = \text{The load factor},
\]

\[
WLF = \text{The wheel load fraction},
\]

\[
M_{\text{max}} = \text{The maximum moment resulting from the design vehicle at the critical location}.
\]

Since the slab bridge specimen is shorter than 40 ft, every truck passing over the bridge applies two load cycles since there are two 32-kip axles per truck in which each 32-kip axle has significant contribution to the maximum moment. The fatigue test was
performed with 550,000 cycles of loading, which is equivalent to 50.23 years of the service life based on the expected average daily truck traffic of 15. The load was applied at a frequency of 1.3 Hz and a magnitude of 22 kips (Fig. 4.14).

![Figure 4.14: Phase I Loading Function](image)

Stiffness tests were performed at an interval of 50,000-load cycles including an initial stiffness test. The stiffness load amplitude was 30 kips. The load was applied under a displacement-based control condition and a displacement rate of 0.007 in./sec.

### 4.5.2 Ultimate Testing

After completion of the fatigue testing, an ultimate test was carried out to determine the capacity of the bridge and to investigate the failure mode. A point load was applied at the midspan of the bridge. The specimen was loaded under a monotonic displacement-controlled protocol to failure with displacement rate of 0.007 in./sec. The data was recorded after completion of each displacement step, which was 0.02 in.
5. Glulam Girder Bridge Experimental Results

This chapter includes experimental results of a full-scale glulam girder bridge that was discussed in chapter 3. Material properties and the performance of the bridge under fatigue and ultimate loading are discussed herein.

5.1 Specimen Material Properties

Different materials were used in the different bridge components discussed in Chapter 3. Mechanical properties for (1) wood used in the deck panels, (2) wood used in the girders, (3) epoxy used to connect the timber components and (4) properties of the elastomeric neoprene bearing pads are presented in this section.

5.1.1 Properties of Glulam Timber

Table 5.1 (same as Table 3.1) presents mechanical properties of the glulam that was used in the as-built test specimen (24F-2.0E), the glulam that was specified to be used in the bridge (26F-1.9E) as well as M-29 glulam, which was used to construct the deck panels.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Notation</th>
<th>Unit</th>
<th>26F-1.9E</th>
<th>24F-2.0E</th>
<th>M-29</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Zone Stressed in Tension</td>
<td>( F_{bxo}^+ )</td>
<td>ksi</td>
<td>2.6</td>
<td>2.4</td>
<td>1.55</td>
</tr>
<tr>
<td>Compression Zone Stressed in Tension</td>
<td>( F_{bxo}^- )</td>
<td>ksi</td>
<td>1.95</td>
<td>1.45</td>
<td>1.55</td>
</tr>
<tr>
<td>Shear Parallel to Grain</td>
<td>( F_{vxo} )</td>
<td>ksi</td>
<td>0.265</td>
<td>0.265</td>
<td>0.175</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>( E_{xo} )</td>
<td>ksi</td>
<td>(1.9\times10^3)</td>
<td>(2.0\times10^3)</td>
<td>(1.7\times10^3)</td>
</tr>
</tbody>
</table>
The girders were specified to be built using 26F-1.9E Southern Yellow Pine. However, the ultimate testing showed that a wrong type of wood (24F-2.0E) was used in the fabrication process by mistake. This issue is discussed in depth in Section 5.2.2.

5.1.2 Properties of Epoxy

Mechanical properties of the Super-Toughened Adhesive Epoxy, which was used to connect the deck panels to the girders, are presented in Appendix D.

5.1.3 Properties of Elastomeric Neoprene Bearing Pads

Neoprene bearing pads were used at the end of girders to separate the timber from the abutment concrete and to allow the girder to freely rotate. A 6 in. by 6 in. by 3/8-in. elastomeric neoprene pad was tested under compressive loads to determine the force-deformation relationship of the bearing pads used at the supports (Fig. 5.1). The stiffness of the linear portion of the force-displacement relationship was 1,128 kip/in. Note that the maximum load on each bearing or the design support reaction (Strength I Limit State) was 28.5 kips.
5.2 Test Results for Glulam Girder Bridge

The girder bridge specimen was first tested under 500,000 cycles of Fatigue II loading using two 22-kip actuators at the midspan. Then, it was loaded monotonically to failure using a 328-kip actuator applying point loads at the midspan. Results of both tests are presented herein.

5.2.1 Fatigue Testing

5.2.1.1 Observed Damage

No damage to any components of the bridge was observed up to 250,000 load cycles, which was approximately 46 years of service life. However, deck-to-deck connections cracked at this load cycle then the crack extended and widened at higher load cycles. Figure 5.2 compares the damage of some of the joints before and after loading.
Figure 5.2: Cracking of Deck-to-Deck Panel Connections
There was no other apparent damage in any other components during the fatigue testing.

5.2.1.2 Stiffness Degradation and Integrity of Specimen

Figure 5.3 shows the measured force-displacement relationship during the stiffness tests, which were performed every 50,000 load cycles. It can be seen that the bridge essentially remained linear-elastic during the fatigue testing with no stiffness degradation. Note that the stiffness is the ratio of the actuator load to the average net midspan deflection of the girders. Since the bridge remained linear-elastic during the stiffness test, a linear regression analysis of the measured force-displacement data can be used to calculate the bridge stiffness.

Figure 5.4 shows the measured effective stiffness ($EI$) versus the number of load cycles. $EI$ can be calculated using Eq. 5.1. It can be seen that the overall bridge stiffness remained constant throughout the fatigue testing confirming that the proposed glulam girder bridge detailing is structurally viable for 75 years of service life.
\[ EI = \frac{PL^3}{48\Delta} \]  

where:

\( E \) = the timber modulus of elasticity (ksi),

\( I \) = the moment of inertia for the non-composite girder cross-section (in.\(^4\)),

\( P \) = the peak applied load in stiffness test (kips),

\( L \) = the test bridge effective span length (in.),

\( \Delta \) = the peak net midspan deflection in stiffness test (in.)

Figure 5.4: Stiffness Degradation during Fatigue II Testing

5.2.1.3 Strain Profiles

The strain profiles of the girders are shown in Fig. 5.5 through 5.7. The two exterior girders were instrumented with two strain gauges on the girder as well as one gauge on top of the deck, while the interior girder was instrumented with three strain
gauges on the girder and one gauge on top of the deck. It can be seen that the strain profiles remained approximately the same through all 500,000 load cycles of fatigue testing. “PI-X” in the graph refers to the stiffness test at X-thousands of load cycles. Although there was some partial composite action, the graphs clearly show that the deck-to-girder connection did not act compositely since the strains of the deck were not compatible with the girder strains. Note that partial composite action was considered during the design of the bridge test girders. This assumption was not conservative as the composite action was minimal during the test. Therefore, the glulam girders should be designed as fully non-composite members.

Figure 5.5: West Girder Strain Profile
The strain profile of the deck is shown in Fig. 5.8. Sixteen strain gauges were installed on the deck surface to determine the effective width of the deck in a composite connection. However, it was found that the full composite behavior cannot be achieved using the proposed deck detailing.
5.2.1.4 Joint Rotations and Slippage

The measured joint rotations versus the number of load cycles for one of the transverse joints is shown in Fig. 5.9. The joint rotations were very small and remained essentially constant throughout the fatigue testing.
The relative horizontal displacements between the girder and the deck (deck-to-girder slippage) was measured at different locations using six LVDTs during each stiffness test (Fig. 5.10). It can be seen that the relative displacements were negligible throughout the fatigue testing indicating that the epoxy was able to hold the deck in-place and to prevent relative movement. Therefore, the proposed deck-to-girder connection using epoxy is adequate and may be used in the construction of new glulam girder bridges.

![Figure 5.10: Deck-to-Girder Slippage during Fatigue Testing](image)

5.2.1.5 Girder Load Distribution

The load cell data can be used to comment on the girder load distribution. The test setup was designed to produce the same loads in the three girders. Figure 5.11 shows the percentage of the load in each girder with respect to the total load during the fatigue testing. It can be seen that the girder loads were 3 to 12% different than the target load (33% for each girder) and the overall distribution remained the same throughout the fatigue testing. The differences can be attributed to the load cell accuracy. The load cell
range was 100 kips while the applied load was 30 kips total resulting in 5 kips per load cell.

![Figure 5.11: Timber Bridge Girder Load Distribution during Fatigue Testing](image)

### 5.2.2 Ultimate Testing

The actuator load was equally spread to the three girders at the midspan of the bridge. The bridge was loaded monotonically using a displacement-controlled loading to failure.

#### 5.2.2.1 Observed Damage

The first crack in the form of delamination was observed in the west girder of the bridge at 101 kips (Fig. 5.12) followed by delamination of the center girder at 113 kips (Fig. 5.13). When the specimen was pushed further, the bridge deck significantly tilted (Fig. 5.14). The specimen failed by simultaneous failure of the west and the interior girders at a peak load of 123 kips (Fig. 5.15). There was no apparent damage in the east girder throughout the testing.
Figure 5.12: First Crack (Delamination) in West Girder

Figure 5.13: Cracking in Center Girder
Figure 5.14: Tilting of Bridge Deck
5.2.2.2 Force-Displacement Relationship

Figure 5.16 shows the measured force-displacement relationship for the glulam girder bridge. The equivalent load level for each of the limit states is also shown in the figure with dashed lines. It can be seen that the bridge remained linear up to the first
cracking, which occurred in the west girder. Load carrying capacity was significantly reduced when the interior girder cracked. The bridge failed at 123 kips.

![Figure 5.16: Force vs. Displacement Relationship During the Ultimate Strength Test](image)

The figure clearly shows the bridge did not meet AASHTO the strength limit state requirements because (1) the as-built girder constituent material was weaker than the specified material due to construction error, and (2) the bridge girders were designed assuming composite action. Review of the material datasheet provided by the manufacturer revealed that the girders were built with 24F-2.0E while the design was based on 26F-1.9E. Furthermore, the strain profiles discussed in the previous section showed that the composite action cannot be achieved in this type of deck system.

Based on these findings, the bridge was redesigned with the as-built material properties and fully non-composite behavior, and the capacity was shown in Fig. 5.16 with a dashed red line. It can be seen that the AASHTO design requirements can be achieved using the proper design assumptions. Therefore, the AASHTO method of design of timber bridges is applicable for the proposed glulam girder bridges.
5.2.2.3 Strain Profiles

Figure 5.17 shows the girder strains during the strength testing. Tensile and compressive strains were identified in the graph. It can be seen that the strain distribution is linear for glulam girders up to the failure. The flexural strain capacity of the girder on the tension side was 1900 micro-strain, which was 58% higher than the design strain capacity ($F_b/E=1200$ micro-strain) for 24F-2.0E.

![Figure 5.17: Measured Girder Strains under Strength Testing](image)

Figures 5.18 to 5.20 show the strain profiles of the girders of the bridge under strength testing at various load levels. The figures show that the glulam girder-deck sections are not composite since the strains are not linear over the depth of the section. However, the assumption of “plane section remains plane” is valid for the glulam girders itself (Fig. 5.19) in a non-composite section.
Figure 5.18: West Girder Strain Profile

Figure 5.19: Center Girder Strain Profile
Figure 5.21 shows the deck strain profile under the ultimate testing for different load levels. The strain was maximum under the applied load. Since the maximum strain was lower than the design strain capacities of the deck, the deck thickness was sufficient in the proposed bridge system.
5.2.2.4 Joint Rotations and Slippage

LVDTs were placed at the top and the bottom of the specimen on the transverse joint closest to the midspan to measure the joint rotation. Figure 5.22 shows the rotation during the ultimate testing. It can be seen that the joint rotated monolithically and rotation increased linearly under the applied load. However, the maximum rotation was negligible.
Figure 5.23 shows the deck to girder slippage for six different locations (HD-1 to HD-6). It can be seen that all of the relative displacements were negligible. For the ultimate test, HD-7 was added to measure the opening of the deck transverse joint at the bottom of the deck. As expected with the higher force the gap opened slightly more due to rotation.
5.3 Summary of Experimental Findings

The main findings of the experimental study on the girder timber bridge are as follows:

- The deck-to-deck male-female joint detailing cracked in the fatigue testing and was found to be inadequate. A connection with flat edges is recommended.

- Even though some composite behavior was achieved, the girders for this type of timber bridge should be designed as non-composite members.

- The structural performance of the deck-to-girder connections by epoxy was sufficient for 75 years of bridge service life. However, long-term performance of the epoxy was not investigated.
- AASHTO methods for the design of girder timber bridges was found to be sufficient with no further modifications.
6. Glulam Slab Test Bridge Experimental Results

This chapter includes experimental results of a full-scale glulam slab bridge that was discussed in chapter 4. Material properties and the performance of the bridge under fatigue and ultimate loading are discussed herein.

6.1 Material Properties

Different materials were used in the different bridge components discussed in Chapter 4. Mechanical properties for (1) wood used in the deck panels and stiffeners and (2) properties of the elastomeric neoprene bearing pads are presented in this section.

6.1.1 Properties of Glulam Timber

Grade 24F-2.0E glulam was used for the construction of the test specimen. Table 6.1 presents the mechanical properties of the material. Correction factors were applied to these values for the design.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Notation</th>
<th>Unit</th>
<th>24F-2.0E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Zone Stressed in Tension</td>
<td>$F_{tx0}$</td>
<td>ksi</td>
<td>2.4</td>
</tr>
<tr>
<td>Compression Zone Stressed in Tension</td>
<td>$F_{cx0}$</td>
<td>ksi</td>
<td>1.45</td>
</tr>
<tr>
<td>Shear Parallel to Grain</td>
<td>$F_{vx0}$</td>
<td>ksi</td>
<td>0.265</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>$E_{xo}$</td>
<td>ksi</td>
<td>2000</td>
</tr>
</tbody>
</table>
### 6.1.2 Properties of Elastomeric Neoprene Bearing Pads

Continuous neoprene bearing pads were used at the end of panels to separate the timber from the abutment concrete and to allow the panels to freely rotate. A 6 by 6 in. by 3/8-in. elastomeric neoprene pad was tested under compressive loads to determine the force-deformation relationship of the bearing pads used at the supports (Fig. 6.1). The stiffness of the linear portion of the force-displacement relationship was 1,128 kip/in. Note that the maximum load on each bearing or the total design support reaction (under the AASHTO Strength I Limit State) was 43 kips per support.

![Figure 6.1: Measured Force-Displacement for Neoprene Bearing Pad](image)

### 6.2 Test Results of Glulam Slab Bridge Specimen

The slab bridge specimen was first tested under 550,000 cycles of the AASHTO Fatigue II loading using two 22-kip actuators at the midspan. Then, it was loaded
monotonically to failure using a 328-kip actuator applying point loads at the midspan. Results of each testing are presented herein.

6.2.1 Fatigue Testing

6.2.1.1 Observed Damage

The only apparent damage during the fatigue test was the widening and extending of existing manufacturing cracks at higher load cycles. For example, the crack between the two laminations at the south end of the bridge was increased from 0.06 in. to 0.0625 in. before and after 550,000 load cycles. In field applications, the bridge deck will be flooded with epoxy thus the observed damage will be eliminated. No other damage was observed in the fatigue testing of the slab bridge.
6.2.1.2 Stiffness Degradation

Figure 6.3 shows the measured force-displacement relationship for the glulam slab bridge during the stiffness tests, which were performed after every 50,000 load cycles. It can be seen that the bridge essentially remained linear-elastic during the fatigue testing with no stiffness degradation. Note that the stiffness is the ratio of the actuator load to the average net midspan deflection of the deck panels. Since the bridge remained linear-elastic during the stiffness test, a linear regression analysis of the measured force-displacement data can be used to calculate the bridge stiffness.
Figure 6.4 shows the measured effective stiffness \(EI\) versus the number of load cycles. \(EI\) can be calculated using Eq. 6.1. It can be seen that the overall bridge stiffness remained constant throughout the fatigue testing confirming that the proposed glulam slab bridge detailing is structurally viable for 50 years of service life.

\[
EI = \frac{PL^3}{48\Delta}
\]  

\text{Eq. 6-1}

where:

\(E\) = the timber modulus of elasticity (ksi),

\(I\) = the moment of inertia for the slab cross-section (in.\(^4\)),

\(P\) = the peak applied load in stiffness test (kips),

\(L\) = the test bridge effective span length (in.),

\(\Delta\) = the peak net midspan deflection in stiffness test (in.)
6.2.1.3 Strain Profiles

The strain profiles of the deck panels are shown in Fig. 6.5 and 6.6. It can be seen that the strain profiles did not change throughout the fatigue testing indicating minimal damage and degradation of the bridge. Furthermore, the strain distribution was almost linear showing that the “plane section remains plane” assumption is valid for the design of slab timber bridges. Note that the strain profile might not be completely linear due to a slight misalignment of the gauges.
6.2.1.4 Joint Rotations and Relative Displacements

The measured joint rotations in the transverse direction of the bridge versus the number of load cycles for one of the transverse joints is shown in Fig. 6.7. The joint
rotations were very small and remained relatively constant through all 550,000 load cycles of the fatigue II testing.

![Figure 6.7: Transverse Joint Rotation vs. Number of Load Cycles for Slab Bridge during Fatigue II Testing](image)

The slippage between the two slabs (HD-1) in the longitudinal direction of the bridge, the opening of the joint at the bottom of the specimen (HD-2) in the transverse direction of the bridge, and the slippage between the deck and the stiffener (HD-3) in the transverse direction of the bridge during each stiffness test are shown in Fig. 6.8. It can be seen that all of these values are negligible indicating adequate performance.
6.2.2 Ultimate (Strength) Testing

The actuator load was equally spread to the two panels at the midspan of the bridge at the centerline of each panel. The deck panels were loaded monotonically under a displacement controlled loading protocol until 270 kips, where the test was stopped due to setup limitations.

6.2.2.1 Observed Damage

There was no major damage throughout the entire strength testing (Fig. 6.9). The only apparent damage was the widening and extending of the existing wood cracks and a minor separation of the stiffeners from the deck panels as shown in Fig. 6.9. This problem could be easily fixed by retightening the bolts, if needed.
6.2.2.2 Force-Displacement Relationship

The measured force-displacement relationship of the slab timber bridge is shown in Fig. 6.10. The equivalent loads for each of the AASHTO limit states are also shown in the figure with dashed lines. The test was stopped at a peak load of 270 kips due to the setup limitation. Based on AASHTO, the displacement limit for the service limit state is 0.466 in. for this bridge. The measures service level displacement was 0.29 in, which is lower than the AASHTO requirement indicating that the design was adequate.
Overall, since there was no significant damage and the bridge surpassed all the AASHTO limit states, it can be concluded that this bridge is a viable short-span option for local roads.

Figure 6.10: Force vs. Displacement Relationship for Slab Bridge during Strength Testing

6.2.2.3 Strain Profiles

Figure 6.11 shows the strains in the deck in the longitudinal direction of the bridge during the strength testing. Negative numbers correspond to compression and positive numbers correspond to tension. It can be seen that the strain distribution is linear for the panels. The lower bound flexural strain capacity of the panels on the tension side was 4000 micro-strain, which was 3.33 times higher than the design strain capacity ($F_b / E = 1200$ micro-strain) for 24F-2.0E glulam timber.
Figure 6.12 shows the strains in the middle stiffener in the transverse direction of the bridge during the strength testing. The strains were not completely linear since there was some slight slippage between the deck panels and the stiffeners changing the load transfer between the members. Overall, it can be concluded that the stiffeners were engaged at different load levels thus they should be utilized in the design and construction of this type of bridges to unify the deck system.
6.2.2.4 Joint Rotations and Relative Displacements

LVDTs were installed at the top and bottom of the specimen on the longitudinal joint in the bridge transverse direction (Fig. 6.13) to measure the joint rotations. Figure 6.13 shows the joint transverse rotation during the ultimate test. It can be seen that the joint rotated monolithically and rotation increased approximately linearly under the applied load. However, the maximum rotation was negligible.
Figure 6.13: Transverse Joint Rotation for Slab Bridge under Strength Testing

The slippage between the two panels (HD-1) in the longitudinal direction of the bridge, the opening of the joint at the bottom of the specimen (HD-2) in the transverse direction of the bridge, and the slippage between the deck and the stiffener (HD-3) in the transverse direction of the bridge during the ultimate testing are all shown in Fig. 6.14. It can be seen that the deck panel relative movement was negligible in the longitudinal direction of the bridge. All relative displacements were negligible at the AASHTO service limit state (less than 0.01 in.) as well as the AASHTO strength limit state (approximately 0.015 in.). However, the longitudinal joint opened in the transverse direction of the bridge and the stiffener slipped with respect to the panels at higher loads.
6.3 Summary of Experimental Findings

The main findings of the experimental study on the slab timber bridge are as follows:

- The structural performance of the connections was sufficient for 50 years of bridge service life. No damage was observed and the stiffness remained essentially constant for the entire fatigue testing. The bridge met all the AASHTO limit states requirements.

- AASHTO methods for the design of slab timber bridges was found to be sufficient with no further modifications.

- Stiffeners were engaged at different load levels and were able to unify the deck as a system. They should be used as specified in AASHTO.
7. Evaluation of Glulam Girder Bridges

The present chapter includes an evaluation of glulam girder bridges for field applications. The evaluation includes: (1) structural performance, (2) constructability, and (3) cost of the superstructure.

7.1 Performance under Fatigue II and Strength I Limit States

Based on the average daily truck traffic (ADTT) of 15 for local roads in South Dakota, approximately 411,000 trucks will cross a bridge in 75 years. The full-scale 50-ft long test bridge was subjected to 500,000 load cycles at the midspan to simulate the traffic loading for 75 years. The load at the midspan corresponded to the maximum moment experienced by the interior girders of the prototype bridge based on the Fatigue II limit state loading specified in AASHTO (2013).

The test bridge did not display any signs of stiffness degradation throughout the 500,000 cycles (Fig. 7.1). The change in the stiffness throughout the fatigue test was less than 3% with respect to the initial bridge stiffness. The only damage observed during the fatigue testing was at the deck-to-deck connections in which the male-to-female connection cracked. This type of connection shall be avoided in field applications. One possible solution is to place the two deck panel faces directly against each other and fill the joint with epoxy (Fig. 7.2). Since there was degradation of the deck and no
degradation of the overall bridge system, it can be concluded that the system is not acting fully compositely.

![Graph of Stiffness Degradation of Girder Bridge under Fatigue II Loading](image)

**Figure 7.1: Stiffness Degradation of Girder Bridge under Fatigue II Loading**

![Diagram of Proposed Deck-to-Deck Panel Connection for Girder Timber Bridges](image)

**Figure 7.2: Proposed Deck-to-Deck Panel Connection for Girder Timber Bridges**

For this bridge, the equivalent AASHTO (2013) service I limit state load was 77.8 kips and the strength I limit state load was 170.9 kips (Fig. 7.3). The timber bridge girders did not crack at the service I limit state. The girders failed before reaching the strength I limit state due to a construction error (a wrong grade of wood was used by
mistake) as well as a design overestimation in which a partial composite action between the girders and the deck panels was assumed. The design capacity of a non-composite girder timber bridge with the same geometry and materials as those utilized in the bridge test model would be 83.21 kips, which is less than the actual capacity indicating that any new glulam girder bridge should be designed fully non-composite. The first girder flexural crack for the test bridge occurred at a load of 100 kips, which indicates that the bridge would have had adequate capacity if the design and construction were based on correct assumptions and material, respectively (Fig. 7.3). The specimen failed at 123 kips. No significant damage was observed in the deck panels throughout the ultimate testing.

Figure 7.3: Measured Girder Force-Deformation Relationship at Mid-Span under Strength Test

Overall, the AASHTO method of design for glulam girder timber bridges was found to be adequate assuming non-composite behavior.
7.2 Constructability

The constructability of the main components of a glulam girder bridge is evaluated herein. The construction of this bridge system is generally fast and does not require skilled labor.

7.2.1 Glulam Girders

The glulam girders will be prefabricated in a controlled environment at the manufacturer’s plant. The fabrication of three glulam girders was discussed in chapter 3.

A 34.5-ft wide bridge will consist of nine glulam girders. Onsite activities regarding the girders would be minimal as they just have to be put in place.

Note that the construction of a glulam girder bridge can be further accelerated if the bride is prefabricated. The timber bridge construction can be in line with accelerated bridge construction (ABC) paradigm if one or two lanes of the bridge is prefabricated and shipped to the bridge site. This is especially favorable since wood is a light-weight material.

7.2.2 Glulam Deck Panels

The glulam deck panels will also be prefabricated in a controlled environment at the manufacturer’s plant. The fabrication of deck panels was discussed in chapter 3.

Field installation of the deck panels is easy and fast. The construction workers will just place the epoxy between the girders and the deck to complete the deck-to-girder connections. As was discussed in the previous section, a fully prefabricated bridge in line with ABC does not need this step.
7.2.3 Glulam Cross Braces

The glulam cross-bracing will be prefabricated in a controlled environment at the manufacturer’s plant (examples were presented in Ch. 3).

The field installation of glulam diaphragms is more involved than the other members since diaphragms have to be placed perfectly between the girders. After alignment, the workers will drill lag bolts through the diaphragms into the girders. If the system is prefabricated, this process will be eliminated in field.

7.3 Cost

Table 7.1 presents a comparison of superstructure materials and fabrication cost between a 50-ft long by 34.5-ft wide double-tee girder bridge and a glulam girder bridge with the same geometry. The materials and fabrication cost for 46-in. wide by 23-in. deep precast double-tee girders is approximately $247 per linear foot based on data provided by the South Dakota Department of Transportation. Nine double-tee girders are used in a 34.5-ft wide bridge. Therefore, the total superstructure materials and fabrication cost for this bridge is approximately $111,150 (Mingo 2016).

<table>
<thead>
<tr>
<th>Bridge System</th>
<th>Glulam Girder Bridge</th>
<th>Double-Tee Girder Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Materials/Fabrication ($)</td>
<td>78,000</td>
<td>111,000</td>
</tr>
<tr>
<td>Total ($/sq. ft.)</td>
<td>45</td>
<td>64</td>
</tr>
</tbody>
</table>

The total material and fabrication cost estimated by the manufacturer for a 50-ft long and 34.5-ft wide glulam girder bridge is approximately $78,000. Therefore, the
materials and fabrication cost of this type of bridges is approximately 30% less than that for double-tee bridges.

Transportation cost for glulam girder bridges is $2.65 per mile at the time of this writing. Additional costs such as assembly, onsite activities, life cycle costs, and substructure fabrication and construction should be included in the total bridge cost.

Overall, the cost of glulam girder timber bridges is estimated to be 15-20% less than that for double-tee bridges, which is the most common type of bridge on South Dakota local roads.
8. Evaluation of Longitudinal Glulam Deck Bridges

The present chapter includes an evaluation of the longitudinal glulam deck bridge for field application. The evaluation includes: (1) structural performance, (2) constructability, and (3) cost of the superstructure.

8.1 Performance under Fatigue II and Strength I Limit States

Based on the average daily truck traffic (ADTT) of 15 for local roads in South Dakota, approximately 411,000 trucks will cross a bridge in 75 years. Since these types of bridges are very short span, each truck will count as two cycles (the maximum moment will occur when each of the rear axles crosses the midspan). The full-scale 16.5-ft long test bridge was subjected to 550,000 load cycles at the midspan, which is equivalent to 50 years of traffic loading. The fatigue test was stopped at this load cycle since there was no damage to the bridge and the stiffness did not degrade. The load at the midspan corresponds to the maximum moment experienced by an interior deck panel of the prototype bridge based on the Fatigue II limit state loading specified in AASHTO (2013).

The test bridge stiffness remained constant throughout the 550,000 fatigue II load cycles (Fig. 8.1). The change in stiffness throughout the fatigue test was less than 1% with respect to the initial bridge stiffness. Furthermore, there was no apparent damage
from the fatigue testing. Therefore, it can be concluded that this bridge system is adequate for the entire service life.

![Figure 8.1: Stiffness Degradation of Slab Bridge under Fatigue II Loading](image)

The equivalent AASHTO (2013) service I limit state load was 44.8 kips and the strength I limit state load was 85.7 kips (Fig. 8.2). The test bridge did not crack up to 270 kips where the test was stopped due to setup limitations. This indicates that the bridge design was adequate (Fig. 8.2). No significant damage was observed in the deck panels and stiffeners under the ultimate loading. The glulam slab timber bridge was found to be a viable option for short spans on local roads.
8.2 Constructability

The constructability of the main components of a glulam slab bridge is evaluated herein. The construction of this type of bridge is generally fast and does not require skilled labor.

8.2.1 Glulam Deck Panels

The glulam deck panels will be prefabricated in a controlled environment at the manufacturer’s plant. The fabrication of the deck panels was discussed in chapter 4.

A two-lane bridge with shoulders on both sides will consist of eight glulam deck panels. Onsite construction will be minimal since the only onsite work will be to place the panels and to anchor them down. The construction can be further accelerated if the entire bridge is prefabricated.
8.2.2 Glulam Stiffeners

The glulam stiffeners will also be constructed in a controlled environment at the manufacturer’s plant. The construction of the stiffeners was also discussed in chapter 4.2.

Glulam stiffeners can be installed in the field very rapidly. The construction workers need to clamp the stiffeners to the deck panels followed by installing lag bolts to hold them together. The onsite installation of stiffeners can be eliminated if the bridge is prefabricated in line with accelerated bridge construction (ABC).

8.3 Cost

Table 8.1 presents a comparison of superstructure materials and fabrication costs for a double-tee bridge and a slab bridge with a length of 16.5 ft and a width of 34.5 ft. The materials and fabrication cost for double-tee girder bridges is approximately $64 per square foot based on data provided by the South Dakota Department of Transportation. Note that the estimated cost of the double-tee bridge was the cost per square foot for a bridge with a length of 50 ft and a width of 34.5 ft.

<table>
<thead>
<tr>
<th>Bridge System</th>
<th>Slab Bridge</th>
<th>Double-Tee Girders</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total ($/sq. ft.)</td>
<td>30</td>
<td>64</td>
</tr>
</tbody>
</table>

The total material and fabrication cost estimated by the manufacturer for a 16.5-ft long by 34.5 ft wide glulam slab bridge is approximately $17,000. Therefore, the materials and fabrication cost of this type of bridge is approximately half of that for double-tee bridges.
Transportation cost for glulam slab bridges is $2.65 per mile at the time of this writing. Additional costs such as assembly, onsite activities, and substructure fabrication and construction should be included in the total bridge cost.

Overall, the cost of glulam slab bridges is estimated to be 50% of that of double-tee bridges, which is the most common type of bridge on South Dakota local roads.
9. Design and Construction Recommendations for Glulam Timber Bridges

This chapter includes design and construction recommendations for glulam timber bridges. The design recommendations are based on the experimental data of full-scale bridge test models. The construction recommendations are based on literature review, fabrication, and assembly of the test bridges in the Lohr Structures Laboratory, and engineering judgment.

9.1 Design and Construction Guidelines for Glulam Girder Bridges

Design and construction guidelines for different components of glulam girder bridges (Fig. 9.1) including: (1) glulam girders, (2) glulam deck panels, (3) diaphragms, (4) wearing surface, (5) railing system, and (6) abutments, as well as the inspection and maintenance recommendations are discussed in this section.

Figure 9.1: Typical Glulam Girder Bridge
9.1.1 Glulam Girders

Glulam girders shall be designed fully non-composite meeting the requirements of current AASHTO LRFD Bridge Design Specifications. The base material type and properties shall be according to AASHTO. AASHTO does not specify the spacing between glulam girders. Nevertheless, a girder spacing of 3 to 6 ft generally results in the most cost effective design.

The type, rating, treatment, and geometry of the wood shall be verified and approved by the designer before fabrication of the girders. The girders shall be precision milled to allow the deck panels to form the crown and to meet the minimum camber requirement by AASHTO.

9.1.2 Glulam Deck Panels

Glulam deck panels shall be at least 6-in. deep as required by AASHTO. The width of the deck panels can cover either the full width of the bridge with one grade or one-half of the bridge width with two grades (Fig. 9.2).
There would be a longitudinal joint directly over the middle girder when installing the bridge with two grades. The panel edges should be cut and prepared to minimize the gap at the joint. The gap should be filled with epoxy.

A weaker wood compared to that of glulam girders is usually used for the deck panels to minimize the costs. The edge of each deck shall be straight then covered with epoxy to complete the deck-to-deck connections in the longitudinal direction of the bridge as shown in Fig. 9.3.
The deck panels shall be connected to the girders using epoxy at the interface. Two rows of screws spaced no more than 18 in. along the length of the girder shall be incorporated to hold the panels and to activate the epoxy (Fig. 9.4).

![Figure 9.4: Deck-to-Girder Connections for Glulam Girder Bridges](image)

### 9.1.3 Diaphragms

AASHTO allows the use of two types of diaphragms to be installed between the girders to improve the stability of the bridge: (1) solid glulam diaphragms (Fig. 9.5a) and (2) steel cross braces (Fig. 9.5b). Another type of diaphragm, glulam cross braces (Fig. 9.5c), was utilized in the full-scale bridge test model of the present study and was found to be a viable alternative. All three options are recommended for field applications.
Figure 9.5: Three Types of Diaphragms for Glulam Girder Bridges

(a) Solid Glulam Diaphragm (Hosteng 2013)

(b) Steel Cross Braces (etraxx.com)

(c) Glulam Cross Braces
9.1.4 Wearing Surface

The use of four types of wearing surfaces shall be allowed for glulam timber bridges: (1) asphalt overlay, (2) asphalt chip seal, (3) aggregate overlay, or (4) epoxy with embedded grit. Of the four, the latter may be preferred to other options since the wood cracks and joints will be filled with epoxy, which is compatible with glulam. Long term performance of the first three options confirms that they are adequate as long as they are well maintained.

Figure 9.6: Different Types of Wearing Surfaces for Glulam Girder Bridges
9.1.5 Railing System

According to AASHTO, a bridge railing system must be positioned to safely contain an impacting vehicle without allowing it to pass over, under, or through the rail elements. Furthermore, a proper railing system must be free of features that may catch on the vehicle or cause it to overturn or decelerate too rapidly.

Any crash-tested railing configuration or those designed according to AASHTO LRFD (2013, Article 13.7) can be used for timber bridges. The rail material can be timber, metal, or concrete. Timber railings are recommended for aesthetic reasons.

![Figure 9.7: Timber Bridge Railing](image)

9.1.6 Abutments

Timber bridge abutments are typically constructed using either timber or concrete as shown in Fig. 9.8. The connections should be designed to resist appropriate design loads as stated in AASHTO. It is recommended that the existing abutments, if any, be
modified for the reuse as this can save time and money. Bearing pads designed according
to AASHTO shall be used to allow the girder to freely rotate.

Figure 9.8: Glulam Girder-to-Abutment Sample Connection

9.1.7 Inspection and Maintenance

It is necessary to perform routine maintenance to keep the wearing surface and
other exposed areas of the timber bridge in good condition. It is also highly
recommended that timber bridges be inspected every 2 years and any wood that is
exposed be retreated every 6 years (Ritter, 1990). Retreatment can be done by spreading
the preservative on the wood using a brush.
9.2 Design and Construction Guidelines for Longitudinal Glulam Deck Bridges

Design and construction guidelines for different components of glulam slab bridges (Fig. 9.9) including: (1) glulam deck panels, (2) glulam stiffeners, (3) wearing surface, (4) railing system, and (5) abutments as well as the inspection and maintenance recommendations are discussed in this section.

Figure 9.9: Typical Glulam Slab Bridge

9.2.1 Glulam Deck Panels

Glulam deck panels shall be at least 6-in. deep as required by AASHTO. The panel thickness shall be determined according to the AASHTO strength I limit state. The deck can be either sloped in one direction or crowned in the middle (Fig. 9.10). The strongest wood available is recommended to be used for the deck panels due to high shear demand. All four edges of deck panels shall be cut and prepared with high precision to minimize the number and the width of fabrication joints.
There would be a longitudinal joint directly at the center of the bridge when installing the bridge with two grades. The panel edges should be cut and prepared to minimize the gap at the joint. The gap should be filled with epoxy.

9.2.2 Glulam Stiffeners

According to current AASHTO LRFD (2013), the product of the wood adjusted modulus of elasticity ($E'$) and the moment of inertia ($I$) of a stiffener must be greater than 80,000 k-in$^2$. The minimum width of the stiffener is recommended to be 5 in. Each stiffener shall be made with the same material utilized in the deck panels.

Zinc-coated lag bolts shall be installed from the underside of the bridge to connect the stiffeners to the deck panels (Fig. 9.11). The lag bolts shall not penetrate beyond 75% of the depth of the deck panels (Fig. 9.11b). The lag bolts shall be at least 12-in. long with a diameter of 0.75-in. Two lag bolts shall be placed per panel on the stiffener.
9.2.3 Wearing Surface

The use of four types of wearing surfaces shall be allowed for glulam timber bridges: (1) asphalt overlay, (2) asphalt chip seal, (3) aggregate overlay, or (4) epoxy with embedded grit. Of the four, the latter may be preferred to other options since the wood cracks and joints will be filled with epoxy, which is compatible with glulam. Long term performance of the first three options confirms that they are adequate as long as they are well maintained.
9.2.4 Railing System

According to AASHTO a bridge railing system must be positioned to safely contain an impacting vehicle without allowing it to pass over, under, or through the rail elements. Furthermore, a proper railing system must be free of features that may catch on the vehicle or cause it to overturn or decelerate too rapidly.

Any crash-tested railing configuration or those designed according to AASHTO LRFD (2013, Article 13.7) can be used for timber bridges. The rail material can be timber, metal, or concrete. Timber railings are recommended for aesthetic reasons.
9.2.5 Abutments

Timber bridge abutments are typically constructed using either timber or concrete as shown in Fig. 9.14. The connections should be designed to resist appropriate design loads as stated in AASHTO. It is recommended that the existing abutments, if any, be modified for the reuse as this can save time and money. It is important that the abutment be completely flush with the deck panels to prevent point loads at the end of the panels. Continuous bearing pads designed according to AASHTO shall be used. The bridge panels should be connected to the abutment using a maximum of two anchor bolts per panel each with a minimum diameter of 0.75 in.
9.2.6 Inspection and Maintenance

It is necessary to perform routine maintenance to keep the wearing surface and other exposed areas of the timber bridge in good condition. It is also highly recommended that timber bridges be inspected every 2 years and any wood that is exposed be retreated every 6 years (Ritter, 1990). Retreatment can be done by spreading the preservative onto the wood using a brush.
10. Summary and Conclusions

The present study was conducted at South Dakota State University to explore the feasibility and performance of glulam timber bridges for local roads. Two types of timber bridges were identified in the literature: (1) transverse glulam deck on glulam girders and (2) longitudinal glulam deck. The performance of both types of glulam timber bridge systems was experimentally investigated in this project through full-scale testing. A summary of the project and the conclusions are presented herein.

10.1 Summary

10.1.1 Glulam Girder Bridge

A glulam girder bridge incorporates glulam deck panels placed on glulam girders with glulam diaphragms. A full-scale 50-ft long, 110.75-in. wide girder bridge was constructed and tested under fatigue, service, and ultimate loadings. The bridge test model, which was the same as a prototype bridge but covering only one lane of traffic, consisted of three girders representing the interior girders of the prototype bridge, 13 deck panels, and 10 cross-braces (five per length) to improve the stability of the bridge. The glulam deck panels were connected to the glulam girders using an epoxy.
The glulam girder bridge was first tested under 500,000 cycles of the AASHTO Fatigue II loading using two point loads applied at the midspan. Stiffness tests were performed at every 50,000 load cycle interval for the fatigue test. Finally, the bridge was monotonically loaded to failure to investigate the ultimate capacities.

Construction of this type of timber bridges was evaluated and a cost estimate was presented. Furthermore, design and construction guidelines were developed, and a design spreadsheet was provided to further aid bridge engineers.

10.1.2 Glulam Slab Bridge

A glulam slab bridge incorporates glulam deck panels placed longitudinally connected by stiffeners. A full-scale 16.5-ft long by 8-ft. wide slab bridge was constructed and tested under fatigue, service, and ultimate loadings. The glulam deck panels were connected to stiffeners using 3/4-in. diameter lag bolts. The test bridge represented two interior deck panels from a prototype bridge.

The glulam slab bridge was first tested under 550,000 cycles of the AASHTO Fatigue II loading using two point loads applied at the midspan. Stiffness tests were performed at every 50,000 load cycle interval for the fatigue test. Finally, the proposed bridge system was monotonically loaded to failure to investigate the ultimate capacities.

Construction of this type of timber bridges was evaluated and a cost estimate was presented. Furthermore, design and construction guidelines were developed, and a design spreadsheet was provided to further aid bridge engineers.
10.2 Conclusions

Based on the findings of the two full-scale bridge tests, the following conclusions can be made for each bridge type.

10.2.1 Glulam Girder Bridge

- Construction of a glulam girder bridge is fast and does not require any advanced technology or skilled labor.
- The girder bridge did not exhibit any signs of deterioration through the 500,000 AASHTO Fatigue II load cycles (equivalent to 91 years of service life) and the bridge overall stiffness essentially remained constant throughout the fatigue testing.
- Damage of male-to-female deck-to-deck connections was observed at 250,000 load cycles (equivalent to 45 years of service life). The damage can be eliminated by connecting flat deck panels with epoxy instead of using a male-to-female connection.
- Although there was partial composite action, it was not sufficient to warrant composite design. The girders should be designed fully non-composite.
- The epoxy connection for the deck to girder connection in the girder bridge performed adequately throughout testing.
- The girder bridge did not meet the AASHTO service and strength limit state requirements under ultimate testing because a wrong grade of wood was used in the fabrication by mistake.
• A calculation of the bridge capacity assuming non-composite behavior and as-
  built material properties and bridge geometry led to accurate estimation of the
  bridge test model capacities. Therefore, current AASHTO method of design for
  this type of bridges is valid.
• The superstructure cost for a 50-ft long by 34.5-ft wide glulam girder bridge is
  70% of that for a double-tee bridge with the same bridge geometry.

10.2.2 Glulam Slab Bridge

• Construction of a glulam slab bridge is fast and does not require any advanced
  technology or skilled labor.
• The slab bridge did not exhibit any signs of deterioration through the 550,000
  AASHTO Fatigue II load cycles (equivalent to 50 years of service life) and the
  bridge overall stiffness essentially remained constant throughout the fatigue
  testing.
• No damage was observed at an actuator load of 270 kips, which was three times
  higher than the AASHTO Strength I limit state load of 85.7. The test was stopped
  due to setup limitations.
• The superstructure cost per square foot for a 16.5-ft long by 34.5-ft wide glulam
  slab bridge is only 50% of that for a typical double-tee bridge.

10.2.3 Glulam Timber Bridges

Overall, it can be concluded from the design, construction, testing, and cost data
that both types of the glulam timber bridges are viable alternatives to the precast double-
tee girder bridges, which are common on South Dakota local roads. AASHTO method of design for timber bridges can be utilized for the design of these types of bridges.
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Appendix A: Shear and Moment Envelopes

The moment envelopes for a typical interior girder for the girder bridge and an interior panel for the slab bridge were determined in CSI Bridge software and exported to excel. The moment envelopes include the effects for HL-93 loading according to AASHTO LRFD (2013).
A.1 Girder Bridge Strength I Limit State HL-93 Design Truck

Figure A.1: Moment Envelope for Strength I Limit State HL-93 Design Truck

Figure A.2: Shear Envelope for Strength I Limit State HL-93 Design Truck
A.2 Girder Bridge Strength I Limit State HL-93 Design Tandem

Figure A.3: Moment Envelope for Strength I Limit State HL-93 Design Tandem

Max Moment = 675.49 k-ft

Figure A.4: Shear Envelope for Strength I Limit State HL-93 Design Tandem

Max Shear = 43.87 kips at 7.5 ft
A.3 Girder Bridge Service I Limit State HL-93 Design Truck

Figure A.5: Moment Envelope for Service I Limit State HL-93 Design Truck

Max Moment = 464.49 k-ft

Max Shear = 31.12 k at 7.5 ft

Figure A.6: Shear Envelope for Service I Limit State HL-93 Design Truck
A.4 Girder Bridge Service I Limit State HL-93 Design Tandem

Figure A.7: Moment Envelope for Service I Limit State HL-93 Design Tandem

- Max Moment = 443.42 k-ft

Figure A.8: Shear Envelope for Service I Limit State HL-93 Design Tandem

- Max Shear = 28.31 k at 7.5ft
A.5 Girder Bridge Fatigue II Limit State HL-93 Design Truck

Figure A.9: Moment Envelope for Fatigue II Limit State HL-93 Design Truck

Figure A.10: Shear Envelope for Fatigue II Limit State HL-93 Design Truck
A.6 Slab Bridge Strength I Limit State HL-93 Design Truck

![Moment Envelope](image1)

Max Moment = 149.30 k-ft

Figure A.11: Moment Envelope for Strength I Limit State HL-93 Design Truck

![Shear Envelope](image2)

Max Shear = 31 k at 2.75 ft.

Figure A.12: Shear Envelope for Strength I Limit State HL-93 Design Truck
A.7 Slab Bridge Strength I Limit State HL-93 Design Tandem

Figure A.13: Moment Envelope for Strength I Limit State HL-93 Design Tandem

Max Moment = 175.60 k-ft

Figure A.14: Shear Envelope for Strength I Limit State HL-93 Design Tandem

Max Shear = 38 k at 2.75 ft
A.8 Slab Bridge Service I Limit State HL-93 Design Truck

Figure A.15: Moment Envelope for Service I Limit State HL-93 Design Truck

Max Moment = 77.27 k-ft

Figure A.16: Shear Envelope for Service I Limit State HL-93 Design Truck

Max Shear = 15 k at 2.75ft
A.9 Slab Bridge Service I Limit State HL-93 Design Tandem

Figure A.17: Moment Envelope for Service I Limit State HL-93 Design Tandem

Max Moment = 92.30 k-ft

Figure A.18: Shear Envelope for Service I Limit State HL-93 Design Tandem

Max Shear = 19 k at 2.75ft
A.10 Slab Bridge Fatigue II Limit State HL-93 Design Truck

Figure A.19: Moment Envelope for Fatigue II Limit State HL-93 Design Truck

Max Moment = 43.05 k-ft

Figure A.20: Shear Envelope for Fatigue II Limit State HL-93 Design Truck

Max Shear = 9 k at 2.75ft
### Non-Composite Design of Girders

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### Slab Bridge Design

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Appendix C: Wood Epoxy Bond Strength

C.1 Introduction

The bond behavior of wood-to-wood connections through epoxy was experimentally investigated in the Lohr Structures Laboratory at South Dakota State University.

C.2. Test Specimen and Matrix

Seven samples each made of three single-lamination pieces (Southern Yellow Pine) connected by epoxy at the interface were tested under pure shear in a universal testing machine to investigate the bond behavior (Fig. C-1). The middle lamination was thicker than the outer two laminations to prevent compressive failure of the wood.
Table C-1 presented the test matrix. The test variable was the epoxy bonded area between the wood pieces. The wood pieces were initially epoxied together with a total bonded area of 12 in². The bonded area was then reduced to 4.5 in² by placing a layer of wax paper in between the pieces of the wood to increase the demand on the epoxy. The specimens were monotonically loaded in compression at a rate of 0.0004 in./sec to failure.
Table C-1: Bond Strength between Timber and Epoxy

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C3. Test Results

Figures C-2 shows the mode of failure of the specimens. In all of the specimens except SP-1, the wood failed by separation of the wood grains at approximately the same peak stress. In SP-1, the epoxy did not cure even after 24 hours. The failure mode indicated that the epoxy is stronger than the wood itself thus the bond strength between the wood laminations depends on the strength of the wood.
Figure C-2: Failure Modes

Figure C-3 shows the measured stress-strain relationship of the test specimens. The stress was the ratio of the applied force to the bonded area. The strain was the ratio of the machine moving head displacement to the total height of the specimen. It can be seen that the average bond strength of all samples was approximately 1250 psi. It can be concluded that the epoxy is adequate.
C.4 Summary and Conclusions

The connections were first tested with a bonded area of 12 in² and then with a bonded area of 4.5 in². For both scenarios, the peak stresses were similar and the wood failed by separation of the wood fibers parallel to the grain. It was concluded that the epoxy is stronger than the wood, therefore the connection is adequate.
Appendix D: Mechanical Properties of Epoxy

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**Thermal Properties**

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**Adhesive Epoxy**

**Advantage: ADV-276**

**Advantage: ADV-176**

**Technical Data**

**Pro-Set**