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National Conference on Wood Transportation Structures

Abstract

The Federal Highway Administration and the USDA Forest Service, Forest Products Laboratory, jointly sponsored the National Conference on Wood Transportation Structures, October 23-25, 1996, in Madison, Wisconsin. This was a direct result of 5 years of cooperation in conducting research related to timber transportation structures. The objective of the conference was to present state-of-the-art information on wood utilization in transportation applications. The conference included a plenary session, reviewing timber bridges throughout the world, followed by concurrent paper sessions on various topics. This report includes the papers presented at this conference.

Keywords: Timber, bridge, transportation structures, stress laminated, metal-plate-connected (MPC) truss, design, wood, fiber-reinforced plastic (FRP), railing, sound barrier, economics, glulam, glued-laminated timber, laminated veneer lumber (LVL), connector, portable bridge, shear, preservative, grading, pile, non-destructive evaluation (NDE), rehabilitation, load rating, mechanical properties

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Preface

Interest in timber transportation structures has significantly increased in the past several years. Much of this interest has resulted from the programs and activities of the USDA Forest Service Timber Bridge Initiative (TBI), passed by the U.S. Congress in 1989, and the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. The TBI is aimed primarily at stimulating local economies and enhancing rural transportation systems through the use of locally available wood species for timber bridge and highway structural applications. From 1992 to 1997, the ISTEA authorizes expenditures by the Federal Highway Administration (FHWA) for research and information transfer related to timber transportation structures.

After 5 years of cooperation in conducting research related to timber transportation structures, FHWA and the USDA Forest Service, Forest Products Laboratory (FPL), are hosting this conference to further disseminate information related to wood utilization in transportation applications. This information will be useful to practicing engineers in government and private practice and members of the academic and industry communities. The objective of the conference is to provide a forum for the exchange of state-of-the-art information on timber transportation structures.

The conference includes a plenary session, reviewing timber bridges throughout the world, followed by concurrent sessions on various topics. Papers presented in the sessions are included in this report and printed as submitted by the authors. The conference sessions include the following topics:

Emerging Bridge Systems
Timber Bridge Case Studies
Timber Bridge Performance and Design
Non-Bridge Structures
Material Properties and Grading
Reinforced Glulam Beams
Timber Bridge Design Considerations
Wood Preservatives
Inspection
Timber Bridge Design
Timber Bridge Programs
Standardized Design
New Wood Treatments
Load Rating, Maintenance, and Rehabilitation
Timber Bridge Economics and Perceptions

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Timber Highway Bridge Construction Practices in The United States

Thomas G. Williamson, P. E., APA Engineered Wood Systems

Abstract

Wood has been successfully used as a highway bridge material in the United States for hundreds of years. Some of the earliest examples of wood bridge construction, dating back to the mid to late 1800's, are the covered bridges, many of which are still in service today in the Eastern U.S. From the early 1900's to the mid 1960's, thousands of highway bridges were built in the U.S. using a longitudinal wood stringer and transverse nail-laminated sawn lumber deck system. In the late 1960's, extensive research was undertaken to develop more efficient wood bridge systems which would result in lower material and labor costs and which would minimize the maintenance requirements often associated with the older wood bridges.

This research has continued for the past 25 years and has led to the development of many innovative wood bridge systems which are now leading to a rediscovery of wood as a highway bridge construction material in the U.S. These systems include a variety of glued laminated timber framing systems, stress-laminated sawn lumber deck bridges and stressed T and box sections utilizing different combinations of lumber, glulam and laminated veneer lumber (LVL) components. New technology is moving towards the use of a broader spectrum of species, the use of prefabricated metal plate truss systems, new stressing rod technologies, the use of high strength plastic composites for reinforcement of wood members and the application of Load and Resistance Factor Design to bridges.

Introduction

As with many European countries, covered bridges using structural wood framing systems represent an important era in timber bridge construction in the U.S. Many of these bridges constructed in the mid to late 1800's are still in service today although most have undergone renovation to bring them up to current design standards. An example of this is the Cornish Windsor covered bridge which connects the towns of Cornish, New Hampshire and Windsor, Vermont. This bridge consists of two 75m spans which are claimed to be the longest individual spans of any covered bridge in the U.S.

This bridge underwent extensive restoration in 1990 to replace damaged members, re-introduce camber into the structure and increase it's load capacity to current American Association of State Highway and Transportation Officials (AASHTO) specifications. In order to achieve these structural enhancements, the design engineers used glulam components to replace original sawn timber members such as floor beams and truss members as shown in Figure 1. Similar structural improvements have been used to renovate numerous covered bridges in the U.S. thus extending their useful lives for another 50 years or more.

Figure 1 — Rehabilitation of Windsor Cornish Covered Bridge Using Glulam Components

From the early to mid 1900's, the use of longitudinal sawn timber beams and transverse sawn lumber decking was the prevalent timber bridge construction system used in the U.S. The decking consisted of dimension lumber positioned on edge with individual pieces being attached to adjacent pieces by through nailing. Toe-nailing was used to attach these decking pieces to the timber beams. While this system provided a structurally sound and tight deck system when initially installed, the effects of annual moisture cycling combined with the dynamic impact of moving vehicles often resulted in a loosening of the mechanical fasteners.

This in turn permitted moisture to migrate into untreated areas of the wood members with decay often resulting in these areas. This combination of loosening of the mechanical fasteners combined with subsequent decay development led to extensive maintenance requirements for these bridges and often county and state bridge engineers discontinued the use of timber in highway bridge construction in favor of pre-stressed concrete or steel systems.

Use Of Glued Laminated Timber

One of the first significant breakthroughs in timber bridge construction in the U.S. was the expanded use of glued laminated timber (glulam) components. Glulam has been succsssfully used as a structural building material in Europe since the late 1890s. In the United States, it has been used in buildings since approximately 1935. The introduction of wet use adhesives in the mid 1940's allowed the uses of glulam to be expanded to include exposed applications such as highway and raiIway bridges, transmission facilities and other exterior structures. While glulam was accepted as suitable alternative to sawn timbers for bridge construction, its use was limited by the lack of knowledge of this relatively new construction product new at least to the US.

Glulam is a versatile engineered wood material that provides several distinct advantages for bridge construction. Because it is a manufactured product, glulam can be produced in a wide range of shapes and sizes to fit virtually any end use requirements. Most glulam used in bridges in the U.S. has involved straight members, but curved members have also been used successfully in a number of applications. For examp1e, the Keystone Wye bridge, shown in Figure 2, built in 1968 in South Dakota is a unique tri-level interchange using both a straight girder glulam bridge and a long span glulam arch structure. This high visibility bridge structure has performed well for over 25 years with only minimal maintenance being required for the timber components. Numerous other glulam arch bridges have been built during the past 25 years using both suspended and supported deck systems and are a popular type of construction where aesthetics are a key design consideration.

Figure 2 — Keystone Wye Glulam Bridge

Another advantage of glulam as compared to sawn lumber is related to the laminating process which randomly disperses the strength-reducing characteristics (src's) of the lumber laminations throughout the member. This random dispersal of src's, results in reduced material properties variability and increased strength characteristics. Glulam also provides better dimensional stability because it is manufactured from dry lumber. Glulam bending members are manufactured using selective lamination placement with higher quality lumber positioned in the highly stressed zones of the member and lower quality material placed in the inner layers of the beam, where bending stress is lower. This practice, which also permits the mixing of species within a single glulam member, helps to extend the available lumber resource and improves the economy of glulam as a construction material.

The majority of the glulam bridges built in the United States during the past 25 years have used either longitudinal girders with transverse deck or longitudinal deck superstructures. Due to the relatively long span and wide spacing

capabilities that can be achieved with the use of longitudinal glulam girders, the incompatibility of the traditional transverse nail-laminated decks as previously described became a limiting factor in expanding the use of glulam.

Glulam Deck Panel Technology

During the late 1960's, research engineers at the USDA Forest Products Laboratory, in cooperation with USDA Forest Service regional bridge engineers and the glulam industry undertook a research program to develop a glulam deck panel to replace the traditional nail-laminated deck system. The concept was to use a vertically laminated glulam member spanning transversely across the longitudinal bridge girders. The length of the deck panel was equal to the overall width of the bridge with the thickness of the panel being dependent on the grade and species of laminating lumber and the spacing of the deck panels. In order to facilitate handling of these deck panels at the manufacturing facility, during transportation and on the jobsite, an industry decision was made to fabricate these deck panels in widths of approximately 122 cm.

In order to achieve plate action for this deck system along the longitudinal direction of the bridge and to minimize the differential deflection at the joints beyond the individual panels, several alternative load transfer mechanisms between panel interfaces were evaluated. The most efficient was the use of a steel dowel inserted in holes prebored at the mid-depth of each panel face as shown in Figure 3. While hundreds of bridges were successfully constructed using this dowel system, alignment problems were occasionally encountered in the field when attempting to pull the individual panels together.

Figure 3 — **Use of Steel Dowels as Load Transfer Mechanism**

To provide the required load transfer between panel edges but not require the close construction tolerances associated with the steel dowel system and it's associated pre -bored deck panel holes, the Weyerhaeuser Company developed a cast aluminum bracket. This bracket, as shown in Figure 4, is positioned in grooves pre-routed in the side of the glulam girders prior to pressure treating and is then attached to the deck panels with a single through bolt. Since the bracket was manufactured from aluminum, this eliminated concerns of corrosion. The use of this deck bracket essentially replaced the steel dowel and has become the standard connector device for the longitudinal girder and transverse glulam deck panel system.

Figure 4 -- Cast Aluminum Deck Bracket

A natural evolution of the transverse glulam deck panel was to use these members as vertically laminated beams spanning longitudinally between supports where spans were relatively short. However, the load distribution provisions in the American Association of State Highway and Transportation Officials (AASHTO) Specifications for Highway Bridges were not favorable to the use of these longitudinal deck panels. The glulam industry sponsored an extensive test program of this system which was conducted at Iowa State University. The results of this study led to more favorable and realistic distribution factors for this type of deck system which were adopted in the AASHTO Highway Bridge Specifications.

As with the transverse deck panels, the longitudinal deck panels are also manufactured in widths of 122 cm. This created a necessity to develop a mechanism for transferring loads transversely between these longitudinal panels. Thus, in addition to developing new load distribution factors for the longitudinal glulam deck panel system, the Iowa State research led to design provisions for spreader beams (typically small glulam sections) which are positioned transversely beneath the longitudinal deck panels at approximately 2.45 meters on center. These spreader beams can be attached to the deck panels with a variety of mechanical fastening devices, several of which are shown in Figure 5. One of the most successful has been the use of the same cast aluminum deck bracket used to attach transverse deck panels to longitudinal stringers.

Figure 5 — Attachment Techniques for Spreader Beams to Longitudinal Decks

Stress-Laminating Concepts

Stress-laminating has been an evolving technology in both Canada and the U.S. for almost 20 years and has achieved considerable success in highway bridge construction during this time. The idea of using stress-laminating techniques for the rehabilitation of existing timber bridges and for the construction of new timber bridges was frost introduced in Ontario, Canada in 1976. The first use of this emerging technology in the U.S. was in the mid 1980's. Since that time, several hundred stress-laminated vehicular bridges have been constructed in the U.S. using sawn lumber laminations, and a guide specification for the design of this type of timber bridge was published by AASHTO in 1991.

Longitudinal stress-laminated decks are typically constructed by placing sawn lumber laminations (either 2x, 3x or 4x material) on edge and stressing the laminations together transversely with high-strength steel bars installed through the mid-depth of the laminations (Ritter, 1990). Figure 6 schematically illustrates this concept.

The compression stress existing between the laminations due to the pressure induced by the transverse stressing rods serves to transfer load between the laminations by friction, causing the deck to act as a large orthotropic wood plate. The typical force induced in these stressing rods ranges when used in conjunction with sawn lumber laminations varies from 355-445 kN.

Figure 6 — Schematic of a Longitudinal Stress-Laminated Timber Bridge

In order to verify the work done in Ontario, Canada and to develop the necessary design criteria for using this stress-laminating technology in the U. S., extensive testing was undertaken at the University of Wisconsin in cooperation with the USDA Forest Products Laboratory. This testing involved the evaluation of two lumber lamination systems. One system utilized nominal 2x12 laminations and was tested over spans up to 8 m. The second system incorporated nominal 4x16 laminations with test spans ranging up to 16 m. Figure 7 shows an example of this laboratory testing program.

Figure 7 — Laboratory Evaluation of a Stress-laminated Lumber Bridge

This research led to a basic understanding of the behavior of these stress-deck systems including (a) the effect of lamination butt joints on wheel-load distribution and deck stiffness, (b) the mechanism of stress transfer into the deck and related edge effects on wheel-load distribution, (c) the effects of transverse bending on the required level of compressive stress and (d) requirements for the anchorage of the pre-stressing rods. Other research efforts have evaluated time related stress losses due to such environmental variables as moisture cycling and investigated various anchorage systems to transfer the pre-stressing force into the wood laminations without over-stressing the outer pieces.

One of the natural characteristics of wood is that it will creep over time under a continually applied load. As the wood laminations creep under the pre-stress force, the level of pre-stress decreases correspondingly. Thus, typically three stressing operations are required to account for these pre-stress losses. After initial pre-stressing the deck. to a level of approximately 0.70 MPa, a second pre-stressing is applied after approximately one week to bring the stress back to the initial level. Final stressing is applied 4-6 weeks after the second stressing. Studies indicate that a pre-stress loss as high as 50% can occur with no significant loss in system capacity.

As part of the ongoing USDA Forest Service timber bridge program, stress-laminated bridges are routinely monitored to verify pre-stress level, load carrying capacity and deflection characteristics. This involves periodic full scale load testing of many of these structures. While most of this load testing has been conducted using static loads to evaluate bridge performance, a recent innovation in load testing has been to evaluate the dynamic performance of the bridges under moving vehicular loads. This is an exciting new technology which offers considerable promise for the future.

Stress-Laminated Glulam Decks

Due to increasing lack of availability of dimension lumber in the U.S. in wide width sizes greater than 2x12 or 2x14 the use of the vertically laminated longitudinal deck systems was limited to spans of approximately 10 meters. Thus, an alternative systsm was sought which would permit the construction of longitudinal deck systems for spans greater than 10 meters. It was conceived that one such solution would be to apply the concept of stress-laminating, as previously described for sawn lumber members, to a series of longitudinal glulam beams placed side by side.

This concept of stress-laminating glulam beams together to form a longitudinal deck system was first used in the U.S. in 1989 for the construction of the Teal River bridge constructed in Wisconsin (Wacker and Ritter, 1992), Figure 8 shows this bridge during the construction sequence. Since the construction of this prototype bridge. a number of other similar structures have been built.

Figure 8 — Teal River Bridge During Construction

Bridges using glulam in stress-laminated deck applications have demonstrated excellent in-service performance. Because horizontally laminated glulam beams allow for deeper sections, longer bridge spans are possible. Additionally, the glulam beams can be manufactured to be continuous over the bridge length and butt joints, which can reduce the bridge strength and serviceability of sawn lumber stress-laminated decks, are not required. These continuous long length beams can also be used to span across intermediate supports resulting in very high stiffness multiple span bridges, further reducing bridge deck deflections.

One of the most noteworthy advantages of using glulam components has been the force retention in the pre-stressing bars. Because the glulam members are dry when installed (moisture content of 16% or less), the beams typically absorb moisture slowly and the deck swells slightly as it moves toward an equilibrium in-service moisture content. As a result, this minimal swelling offsets force loss in the pre-stressing rods due to stress relaxation in the wood and the net loss in bar force is minimal. Extensive monitoring of these bridges by the U.S. Forest Service has verified this performance characteristic.

Stress Laminated T-Beam and Box-Beam Sections

In the mid 1970's, an extensive test program was conducted at Colorado State University to determine the degree of T-beam action which could be expected in a longitudinal girder and transverse glulam deck system. Full size double T-sections spanning 12 meters were tested under simulated AASHTO truck loading. These test sections used conventional 122 cm wide transverse deck

panels with steel dowels used to provide load transfer between adjacent panels. The deck panels were attached to the stringers using steel lag screws. While there was approximately a 10-15% decrease in stringer deflection, the degree of T-beam action was limited by the effectiveness of the mechanical connections and the associated slip which occurred during loading.

However, the advent of stress-laminating offered new opportunities to achieve composite T-beam action between the deck and stringers without being dependent on the mechanical fasteners between the deck and stringers. The clear span of glulam bridges is typically controlled by design considerations related to the depth of the superstructure and by economical limitations on the bridge depth. Creating T and other composite configurations allowed designers to overcome these limitations, thus permitting much greater span capabilities for glulam bridges.

Two types of composite bridges that have been successfully used in the U.S. are the T-section and box-beam bridges, shown schematically by Figure 9. T-beam bridges can be constructed using vertical glulam or laminated veneer lumber (LVL) web members with flanges constructed of sawn lumber or glulam deck panels. The composite action between the flange and the web is developed through friction by stress-laminating the section with stressing bars through the flanges and webs. The box section is similar to the T-section, but with flanges and stressing bars added to the bottom of the section to create a higher overall section modulus and moment of inertia.

Figure 9 — End View Cross-Sections of a Stress-Laminated T-Beam Bridge (top) and a Stress-Laminated Box-Beam Bridge (bottom)

The concept of stress-laminated T-section and box-beam bridges has been well received and more than 30 bridges have been built over the past 3 years in the U.S. The longest span structure to date is a 27 meter span stress-laminated T-beam bridge, which was built in Arkansas in 1993. Recent research work regarding these glulam superstructure configurations was completed at West Virginia University, in cooperation with the Federal Highway Administration (FHWA) and the U.S. Forest Products Laboratory, and used a modular construction approach (Barger, et al., 1993; Davalos, et al., 1993).

In addition to continued research on stress-laminated T-section and box-beam bridges using glulam or LVL webs and sawn lumber flanges, research is underway at the University of Wisconsin, in cooperation with the U.S. Forest Products Laboratory and the U.S. Federal Highway Administration, to develop systems constructed completely from glulam components (Oliva and Rammer, 1993). It is estimated that glulam bridges built using this technology will be able to clear span over 30 meters with structural sections less than 106 cm in depth.

Research and field evaluation are continuing on the structural performance of these systems. Draft specifications for the design and construction of stress-laminated T-section and box-beam bridges are currently being developed for adoption by AASHTO.

Alternative Species For Glulam

Glulam can be manufactured from virtually any softwood or hardwood species provided the end product meets necessary strength and stiffness requirements. In actuality, most of the glulam manufactured in the U.S. during the past 60 years has utilized either Douglas Fir-Larch or Southern Pine lumber. However, with continuing changes in the availability of worldwide wood resources, and with increased emphasis on using underutilized local wood species, there has been a growing interest in the U.S. towards developing new glulam layups utilizing both hardwood and softwood species. Over the past 5 years, most of the work on alternative species for glulam has centered on the utilization of hardwood lumber, but several secondary softwood species have also been evaluated.

Recent glulam research completed at the Pennsylvania State University, West Virginia University, and the U.S. Forest Products Laboratory has been directed at developing glulam layups using Red Maple, Red Oak and Yellow Poplar (Manbeck, et al., 1993; Shaffer, et al., 1991; Moody, et al., 1993). Recent full scale tests of glulam beams manufactured using mechanically graded Red Maple and Yellow Poplar support the use of bending and modulus of elasticity design values comparable to those achieved with the traditional Douglas Fir-Larch and Southern Pine softwoods.

In addition to developing specifications for glulam produced from hardwood species, efforts to develop high strength and cost efficient glulam layup combinations utilizing secondary softwood species have also been successful. A project in Wisconsin using a combination of red

pine and Southern pine to manufacture glulam beams resulted in the construction of the Teal River stress-laminated deck bridge previously described (Wacker and Ritter, 1992), These beams used Southern pine for the outer tension and compression zones with the red pine being used for the core of the beams. The resultant beams had similar bending strength and stiffness characteristics as beams manufactured from all Southern pine laminations. This further led to the design and construction of a stress-laminated bridge using glulam manufactured exclusively from red pine lumber. Other projects using secondary softwoods, such as Eastern hemlock, Ponderosa pine and cottonwood are underway.

Laminated Veneer Lumber Bridges

Other glued engineered wood products are also being used for bridge construction in the U.S. Laminated veneer lumber, LVL, has long been used in both residential and commercial building construction. LVL is produced by bonding together veneers of wood (with the thickness of the individual veneers not to exceed 6mm) with the grain of all veneer layers approximately parallel to the long direction of the member. This product has very high bending strength and stiffness characteristics as compared to other wood products and is thus ideally suited as a bridge construction material where these properties typically control design. LVL also has a high propensity for absorbing preservative treatment chemicals due to the lathe checks inherent in the individual veneers. This further enhances its characteristics as a bridge construction material.

LVL has been used in numerous bridges in the U.S. One system involves the use of LVL components to create a longitudinal "slab" type deck system. The individual LVL pieces are stressed together using the stress-laminating technique previously described for sawn lumber bridges As with glulam, the use of LVL permits longer spans due to the higher strength characteristics assigned to this product as well as due to the wider range of sizes available.

Another use of LVL in bridge construction has been as the web members of composite stress-laminated T-systems using sawn lumber deck pieces. In this type of application, the LVL is used as an alternative material to glulam. As an extension of this technology, one manufacturer produces a prefabricated all-LVL T-section. These individual T-sections are produced in variable depths depending on design span and loading. After the individual T-sections are positioned side by side, they are stressed together using high strength steel rods. Figure 10 shows a bridge being erected using these LVL T-sections.

Figure 10 — Installation of an LVL T-section

Crash Tested Rail Systems

One ongoing concern expressed by bridge designers in the U.S. has been related to the need for cost efficient crashworthy timber bridge guardrail systems. AASHTO and the Federal Highway Administration have a program underway which will require all highway bridges guardrail systems to be fully crash tested. Several levels of guardrail performance are being considered in this program ranging from resisting the impact of passenger vehic1es to that of large over over commercial trucks. Both the US. Forest Products Laboratory and the Federal Highway Administration have completed full scale crash test programs to evaluate the performance of various timber bridge guardrail systems on both longitudinal glulam deck and longitudinal glulam stringer and transverse deck bridge configurations.

These crash tests have been conducted using a variety of test vehicles ranging from passenger cars to pick-up trucks to larger commercial trucks. Rail system tested have included (a) single glulam rail with wood posts, (b) single steel rail with wood posts, (c) glulam rail, wood wheelguard and wood posts and (d) other combinations of guardrail system components. To date, all of the guadrail systems tested in these two research studies have met the crash test requirements established by AASHTO and the Federal Highway Administration. The availability of fully crash tested guardrail systems will provide a major impetus to the further use of glulam highway bridge systems in the U.S.

Emerging Technologies

Many other innovative technologies for timber bridge construction are being evaluated by U.S. researchers. One of these concepts uses metal gusset plate wood trusses as the main longitudinal superstructure elements. While metal gusset plate wood trusses are used extensively for floor and roof framing in both residential and commercial

building construction, their use in bridge construction is on an experimental basis with only a few prototype bridges having been constructed.

Two possible technologies using these trusses are being evaluated. In one system, a series of identical trusses are placed in direct contact with each other (in the same manner that sawn lumber laminations would be used) and these are then stressed together using the stress-laminating concept. Once stressed together, these multiple truss bridges create a bridge with a very high degree of stiffness.

A second approach using pre-engineered metal gusset plate trusses is to prefabricate girder trusses using several of these trusses mechanically tied together. These girder trusses are then used as the vertical web elements in combination with stress-laminated sawn lumber deck pieces to create a T-beam configuration.

Work is also underway to evaluate alternative stressing rod materials using high strength steel strands in lieu of the traditional steel rods which have typically been used in the stress-laminated systems. Other researchers are evaluating the potential for using high strength structural composites such as reinforced plastics for these stressing elements.

Another evolving technology involves combining high strength structural plastic composites with wood such as by reinforcing the tension zone of glulam beams with fiber reinforced plastic (FRP). In virtually all instances, the bending strength of glulam is controlled by the tensile strength of the lumber or the end joints on the tension side of the beam. The potential for increasing the bending strength of glulam by reinforcing the outer tension zone has been evaluated by many investigators during the past 30 years using a variety of materials. Recent developments in the use of FRP suggest that this high-performance material offers the possibility of being bonded to the wood laminations under factory conditions thus providing this tension reinforcement.

Forming a composite beam by using a relatively small amount of FRP to reinforce the outer tensile zone offers the potential for significantly increasing the bending strength of glulam beams. However, the use of this reinforcement material may have limited effect on increasing overall stiffness when used in the relatively small percentages required to achieve the increased tensile performance yet be cost competitive with alternative materials.

Recent work has been completed using various types of fibers in pultruded plastic products, often referred to as FRP, to reinforce glulam (Tingley, 1990). Tingley and other researchers from Oregon State University report highly favorable results by reinforcing the tension zone of glulam using fiber reinforced plastics and have obtained U.S. building code acceptance of their proprietary technology. This has led to the use of these reinforced glulam in a number of building and highway bridge applications. Cooperative research has also been undertaken between West Virginia University and the U.S. Forest Products Laboratory to investigate similar uses of FRP bonded to either the tension side only or to both the tension and compression sides of beams. Other research evaluations related to reinforcing glulam beams using FRP products has also been undertaken by the University of Maine and by APA-The Engineered Wood Association.

These research efforts could soon lead to the construction of additional bridges using composite glulam and fiber reinforced plastic beams. Reinforced beams appear to have the greatest opportunity for showing economic advantages in applications where either (a) bending strength controls the design, (b) it is critical to minimize beam depth, or (c) the beams are part of a composite structure where the added strength provides substantial benefits.

Another related technology being pursued is to use high strength reinforcing fibers, such as Kevlar, to pre-stress the tension zone of glulam beams in a manner similar to pre-stressed concrete design. In this application, fiber mats are positioned between outer laminations of the glulam beam and then prestressed prior to face bonding the glulam beams. The pre-stressing force is then relaxed after the bonding operation is completed. While this research appears promising, it is only in its infancy and considerable work remains to be completed before it can become a marketable technology.

Summary

Beginning in the late 1960's, extensive research has been ongoing in the U.S. to advance the technology for using timber in highway bridge construction. This research has resulted in many innovative technologies that have been successfully incorporated in numerous timber highway bridge applications throughout the U.S. Continuing research will undoubtedly expand on existing technologies and create new technologies which will enhance the competitiveness of timber as a highway bridge construction material. In addition, these technologies also have applications in other types of bridges such as pedestrian bridges and railroad bridges thus leading to an expanded use of wood in those types of structures.

It is further hoped that much of the timber bridge technology developed in the U.S. over the past 25 years may have application in other countries where the use of timber in bridge construction is a design option. For example,

although not located in the U.S., one of the most striking examples of the innovative use of glulam in highway bridge construction is the cable-stayed glulam bridge constructed in Hiroshima, Japan as shown in Figure 11.

Figure 11 — Cable Stayed Glulam Highway Bridge.

This two lane wide bridge has a total length of 145 meters with a center clear span between support towers of 84 meters. This bridge uses a glulam truss configuration for the suspended superstructure. Although constructed in Japan, the glulam components for this unusual timber bridge were all rnanufactured, pre-fabricated for all connections and pressure preservatively treated at manufacturing facilities in the U.S. Perhaps this and other innovative timber bridge systems as described in this paper will once again lead designers to timber as a modern highway bridge construction material.

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Timber Bridges in the Nordic Countries

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Abstract

The Nordic Timber Bridge Programme is a co-operative programme for Sweden, Finland, Denmark and Norway, with the head of the working group and the chairman from Norway. The participants in this programme come from the Timber Industries, the Timber Research Institutes and Public Roads Administration in the various Nordic countries. The paper will describe the most essential points in "why and how" this Timber Bridge Programme was established. The objective was to increase the knowledge of timber used in bridges, and timber bridges vis-á-vis architects, engineers and politicians regarding material and construction preference and disadvantage, environmental aspects, construction costs and service life costs. The aim is to construct more timber bridges at the sacrifice of steel and concrete bridges.

Background

Bridges in a historic perspective started with different forms of natural bridges, see figure 1.

Figure 1-Historical bridge.

In the Nordic countries, as in most part of the world, timber and stone were common constructing materials for bridges in the past. The most common timber bridge types were probably:

- Beam bridges
- Cantilevered timber bridges
- Strut frame bridges
- Suspended frame bridges
- Truss bridges

These bridges could also be covered.

Figure 2-Bridge from the Viking period.

In Norway the number of timber bridges increased during the last part of the nineteenth century, but around the turn of the century the building of timber bridges stopped almost completely. Most likely because steel bridges became cheaper and simpler to construct.

During a period in the 1960's we constructed in Norway a number of glulam bridges for pedestrians. These bridges were competitive concerning cost and aesthetics. After a period of 15 years we almost stopped constructing these bridges because of the supposed service life not being as expected. It seemed

that the bridges decayed much sooner than anticipated. But this was only an assumption.

In connection with the design of the indoor arenas for the 1992 Olympic Winter Games in Norway there was a competition between steel and timber for the roof constructions. Timber was chosen because of the cost and the aesthetics. These arenas are huge constructions compared to previous Norwegian timber constructions. The roof structure consists of several truss girders. The largest span is more than 100 m and the load carrying capacity of two such girders is high enough to facilitate a two-lane road bridge.

If we construct big halls in timber, why don't we also make timber bridges? It was decided to establish a timber bridge group with participants from the timber research centre, the timber industry, and the Public Roads Administration. The task was to work out a conceptual design for timber bridges in Norway with emphasis on aesthetics, service life, design rules and competitive economics. The result of the conceptual design was satisfactory regarding these issues and the work with timber bridges continues. However, the report is written in Norwegian.

About the same time groups from all the Nordic countries worked on or had the intention to start working on national timber bridge projects. This lead to a study trip to the Forest Products Laboratory in Madison, Wisconsin, USA. The laboratory was chosen because it is well-known that it possesses the engineering environment and experience to produce research work and to construct timber bridges. On this study trip the whole Nordic group decided to make an effort in establishing a project for timber bridges with financial support from the Nordic Industrial Fund, the timber industry, national timber research institutions and the national Roads Administration.

The Nordic Project in General

During the visit the delegates decided to co-operate in the production of relevant facts to encourage timber bridge structures in the Nordic countries.
The Nordic countries are Sweden, Finland, Denmark and Norway.

Figure 3-Map showing the Nordic countries as a part of Europe.

The objectives of the project were to promote timber bridges by

- removing some of the myths about these bridges that are obstacles in the choice of timber bridges in competition with concrete or steel bridges.
- increasing the participants' knowledge of timber bridges.
- doing research on timber bridges.
- providing relevant information about timber bridges to be announced for decision-makers by means of
  - publishing reports and participating in exhibitions with results from this project
  - announcements aimed directly at the decision-makers
  - presentations at various conferences
  - newspapers and television.

The success rate is not manifested in figures, but in terms which mean that the trust in timber bridges must be re-established, and that construction of timber bridges should increase substantially in specific areas in the Nordic countries and in neighboring countries. Both pedestrian and road bridges are included in this project, but the increase in road bridges is the most encouraging.

We also expect that this project will result in increased knowledge of timber bridges and start a process with continuous improvement of the timber bridge concept.

Table 1-The sub-project each country deals with. * shows the responsible country.

The Nordic timber bridge program is organised as a partnership with an executive group consisting of two representatives from each Nordic country. In addition the project has a project group which co-ordinates the activity on behalf of the executive group.

The complete project is composed of the thirteen sub-projects as indicated above.

Each sub-project has one national responsible person from one of the countries. The other nations take part in each sub-project as agreed upon. Not all sub-projects have partners from all countries. A few of them are actually individual, national sub-projects. However, they are still managed by the project and the executive group.

For the Nordic timber bridge project NOK 8,000,000 ($ 1,200,000) is allocated from the following funds:

- 50 % from the timber industry and from national road administrations
- 20 % from national research funds
- 30 % from Nordic Industrial Fund.

The project will be closed in October this year and has at that time lasted for two years.

There are many advantages with a Nordic project like this.

Firstly, the various partners from the Nordic countries have different experience from their own areas. This means that the store of knowledge based on what all partners know about this subject is greater than the individual knowledge.

Secondly, the basic experience that each of the partners has obtained prior to starting the project is the basis for aiming directly at the problems that this timber bridge project involves.

Finally, the knowledge coming from a varied group of members and the issues which are raised are the bases for being creative in this study and to indicate excellent solutions.

The project has an on-going research programme for fatigue study of dowel connections. About 20 samples of dowel joints were made. These joints were tested in a fatigue testing machine with up to 10^7 cycles. As the test is still in process, I have to come back to the result in the oral presentation.

Examples from Sweden

Klockarbergsvägen

A new road outside the Swedish city Skellefteå was opened in the autumn of 1994. Two glulam bridges are included in this project. Both of them are based on the principle of stress-laminated timber decks, and are designed to carry the normal traffic loads according to the Swedish regulations “Bronorn 88”. Bridge number 1 has a total length of 35,2 m and a width of 10,7 m. The main load carrying structure consists of 4 rows of poles with crossbeams and 4 parallel three-hinged arches, as indicated in figure 4. The span of the arches is 15 m. The stress-laminated deck is built up by glued laminated beams with a depth of 360 mm.

Figure 4-Klockarbergsvägen bridge 1.

Bridge number two has a total length of 16,2 m and a width of 10,5 m. The deck is supported at the ends and by two crossbeams on the top of the poles, see figure 5.

Figure 5-Klockarbergsvägen bridge 2.

The producer of both bridges was Martinsons Trä AB and the design was done by Trätek in co-operation with Martinsons Trä AB.

Enköpingsbron

In 1995 a new timber bridge for light-weight traffic was opened in the Swedish city Enköping. The bridge has two spans, each 28 m long, see figure 6.

Figure 6-Enköpingsbron.

The deck is supported by a CCA-treated roundwood structure with zinc coated steel parts. The stress laminated deck is built up by CCA-treated solid timber planks and is divided into 5 sections, with lengths of 5 to 6 m. The deck is covered by a membrane and an asphalt top layer.

Examples from Denmark

Pedestrian arch bridge Roskilde

Just outside the Danish city Roskilde a new pedestrian glulam bridge was opened in 1995. The bridge has a total length of 35,8 m and a width of 4,0 m. The main load carrying elements are two three-hinged arches with a span of 32,4 m, as shown in figure 7.

Figure 7-Arch bridge Roskilde.

The arches are circular glulam elements with inner radius of 25 m and a constant cross section area of 300 mm x 1000 mm. The timber deck planks (150 mm x 60 mm) are supported by glulam crossbeams (200 mm x 500 mm) and 7 secondary beams in the longitudinal direction in solid timber (140 mm x 220 mm). The upper surfaces of the glulam arches are protected with aluminium covering.

Pedestrian crossing Hørsholmvej

This timber bridge was built as a temporary bridge in 1955, but is still in service. The total length of the bridge is 39 m which includes 4 spans with maximum spans of 11,6 m, see figure 8.

Figure 8-Pedestrian crossing Hørsholmsvej.

The main structures are "lattice trusses". (This system was patented by the US architect Ithiel Town in 1820, and a great number of covered bridges based on "Town lattice trusses" were built in many parts of the United States for railroad and highway traffic.) The cross sections of the trusses are described in figure 9.

Figure 9-Cross section lattice trusses.

The web members have a cross section of 1,25" x 6", the lower chord members have a cross section of 3" x 9" and the upper chord members have a cross section of 2,5" x 5" with a horizontal 2" x 8" on the top. After having been in service for 40 years, the bridge in still in good shape with very low maintenance costs.

Examples from Finland

Myllysilta bridge in Nurmijärvi

This roundwood timber bridge is a so called "museum bridge" and was built in 1966. The total length of the bridge is 47 m, and the two main spans are 16 m. The main structures are 7 roundwood systems, as shown in figure 10.

Figure 10-Myllysilta bridge.

The diameter of most of the heavy loaded members are 200 mm, and all the timber pieces are pressure treated with creosote.

Kruununmylly bridge in Hämeenlinna

This bridge was opened in 1993, and was the first Nordic timber and concrete composite bridge. The span of the bridge is 8 m and the width 12 m, see figure 11.

Figure 11-Kruununmylly bridge.

The main structure consists of 11 impregnated glulam beams with cross sections of 140 mm x 630 mm and with a concrete slab as deck structure. The shear connections between the glulam beams and the concrete slab consist of round steel bars. Concreting may be executed without temporary supports, and permanent plywood panels between the girders are the only formwork needed.

Examples from Norway

Figure 12-Evenstad bridge.

Evenstad bridge

Evenstad bridge crosses Glomma river in Hedmark county about 250 km north of Oslo.

A preliminary study for the Evenstad bridge was to compare the cost, the aesthetics and the functionality of one steel, one concrete and one timber bridge in the same situation. Especially for the timber alternative the technical challenge should not be too extensive to handle. The study concluded that the estimated cost for these three alternatives would be about the same, the aesthetic challenge was easier to handle in the timber alternative, and the functionality of the three alternatives would be the same. However, because of the flood condition the superstructure must be situated above the deck. It was decided that a timber bridge should be built. This will be the first modern Norwegian road bridge made of timber.

Technical data and dimensions of the timber bridge – Evenstad bridge will be 180 m long and the carriageway 6,5 m wide. The bridge will consist of five 35,5 m simply supported spans. The superstructure is composed of two glulam truss girders with a curved upper chord. The girders support the deck by means of crossbeams underneath the truss girders. The substructure will consist of three concrete piers on steel piles and two directly founded abutments. The deck will be a stress laminated timber deck with damp proofing and asphalt.

The project team faces two main challenges for this timber bridge project.

Figure 13-Timber truss girder with joints of multi-section dowel.

The joining of big glulam structures is both feasible, economical and practical to do by using multi-section dowel joint. This will also be the case here, as shown in figure 13. The evaluation is based on experience gained both from the construction of the Olympic Arenas in Norway and from Switzerland in particular. As mentioned, some fatigue studies for this joint are in progress in the Nordic timber bridge project.

Figure 14-Stress-laminated timber deck.

It will be the first time stress-laminated decks are utilised in a large bridge in Norway. Other nations as Sweden, Switzerland, Australia, Canada and USA have used them for several years. *On* the topside of the deck will be placed a membrane with an asphalt layer top, se figure 14.

Protection system — For this bridge a preservation treatment with MT Creosote with BAP less than 50 ppm will be used. This product is assessed to be the best long-term treatment also in relation to the environmental aspect. Creosote with low benzopyrene used for wood preservation is no longer classified as a carcinogenic product according to EU-classification.

The first timber bridge for road traffic in many years in Norway will be instrument to monitoring moisture content and stress.

From a **maintenance** aspect the use of timber should not be problematic in such a structure. Both the experience from countries that have used modern timber bridges in the last decade, and the use of timber in outdoor constructions show that the maintenance cost for timber structures is very low during the first 40-70 years. The reason for this is both the pressure-treatment with creosote and the fact that the wood itself, with its good characteristics as a bridge construction material, is able to resist impact and shock load.

I will now mention some recently constructed pedestrian bridges.

Stien bridge, Rogaland county.

Stien bridge is a pedestrian bridge crossing the highway about 15 km south of Stavanger leading into the city. It was constructed in 1993.

Stien bridge is 31 m long and 3.4 m wide with a 3 m carriageway. The bridge consists of two glulam arches with 24 m span which support a deck of wood and steel grating. All timber is preserved with CCA and mordant of oil.

Figure 15-Stien Bridge.

Holmen bridge, Nord-Troendelag county

Holmen bridge is a pedestrian bridge crossing Namsen river in Nord-Troendelag county. It was constructed in 1994.

Holmen bridge is 60 m long and 3.65 m wide with a 3 m carriageway. The bridge consists of two glulam arches with 40 m span supporting a deck of wood and steel grating.

All timber is preserved with creosote. The only problem up to now is that some creosote leaked from the wind bracing to the pedestrians on the bridge. The problem was solved by removing redundant creosote.

Figure 16-Holmen bridge.

Oeya bridge, Oppland county

Oeya bridge is a pedestrian bridge crossing a small creek in a park in Lillehammer. It was constructed in 1993.

This is a one-span 13 m long and 2 m wide bridge. It is made of one arched glulam girder with steel ribs which also function as parapet. On each post is a timber hand rail. The timber girder and the ribs support a CCA-treated deal deck.

The timber girder is treated with creosote.

Figure 17-Oeya bridge.

0s bridge, Hedmark county

0s pedestrian bridge is only in a preliminary stage, but is planned as a 80 m long, two-span covered bridge.

Figure 18-0s bridge.

A dream

Will it be possible to design and build a really big timber bridge? Rune Abrahamsen, a student at the university of Trondheim, questions this in his postgraduate thesis. He concludes that it would be possible to construct a truss arch bridge with a span of 150 m. This is based on experience from the construction of the before-mentioned Olympic Arenas in Norway. The bridge will sustain the traffic load described in the Norwegian Traffic Load Regulation.

The bridge consists of six truss arch timber girders, se figure 19. The top and bottom flange will be 645 x 645 mm and the height of the arch 4 m. The wooden deck will consist of 600 m^3 timber and the girders of 1234 m^3.

Figure 19-A dreambridge ?

The timber part of the bridge is estimated to about 15 MNOK (M$ 6.2). At that amount the structure will not be competitive with a similar bridge in concrete or steel.

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Timber Bridges in Australia

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Abstract
This paper presents an overview of the history of timber bridges in Australia and examines applications of timber technology related to transportation structures and the implementation of technology in Australia.

Keywords: Timber, Bridges, Australia

Introduction
European presence on the Australian mainland has only existed in a permanent format for approximately 200 years. Prior to this, the indigenous aboriginal population lived in harmony with the often harsh environment.
Australia's significant timber resources are located in the coastal zones of the South West, Eastern and South Eastern regions of the continent. The remainder of the continent is semi arid to arid with little to or no significant commercial timber resource.
The Indigenous population had little commercial interest in the forests other than to provide a refuge and environment in which they and their food resource resided.
Europeans had a different perspective on the forest resource. Australian forests provided them with an untouched natural resource which provided materials of strength, size and durability.
Unlike the Indigenous population, Europeans had little understanding of the environmental management issues in this new continent which would enable a sustainable commercial use for the forest resource.
It took more than a century for Europeans to realise that the timber was a finite resource unless managed.
Australia now has a well managed hardwood and softwood forest management strategy in practice. Native Eucalyptus forests are managed on a sustainable basis with specific management practices developed for the different species and environments. Hardwood plantations are being developed to supplement the managed native forest resource.
Commercial softwood plantations, mainly Radiata Pine, have been planted over the previous 70 odd years. Softwoods have provided the economical alternative resource for the value added side to the timber industry.

History of Timber Bridges in Australia
The earliest recorded timber bridge built in Australia was the Bridge Street bridge over the Tank Stream in Sydney constructed in 1788 but was washed away soon after construction. The Stream rapidly became a polluted drain as Sydney developed and was finally fully covered as part of Sydney's expansion by 1870.

Professor Colin O'Connor, in his book "Historical Bridges of Australia" records that by 1805, there were 10 timber bridges on the Sydney to Paramatta Road constructed of Iron bark or Blue Gum timber stingers and decking.

As the Colony developed bridges were constructed, many using timber superstructures and masonry piers. None of

these early bridges remain.

There are few records of the actual numbers of timber bridges constructed during the early years in the Colonies.

As the Colonies expanded, and the need to transport people and commodities increased, there was a need to build bridges with larger spans and greater load capacity.

The timber truss is generally considered as an American innovation. During the early and mid period of the nineteenth century, many consider that, as a result of the Gold rushes and rapid movement of people, this technology was transferred internationally.

New South Wales was the Australian leader in the use of timber truss bridges to overcome the span requirements. Professor O'Connor indicates the development of truss bridges in Australian can be classified into so called old type trusses prior to 1886, McDonald trusses 1889-1894, Allan trusses 1893 to approximately 1920 and the de Burgh composite steel and timber and Dare composite trusses in the period 1900 to 1912.

Old Style Truss (Public Works Department)

McDonald Truss

Allan Truss

De Burgh Truss

Dare Truss

Timber Truss Types for Bridges Constructed 1860-1912

One of the oldest type timber truss bridges still in existence is the Karuah River bridge at Monkerai which was built about 1877.

Laminated Bow Staring arch bridges were constructed in the period 1870 to 1900, but many developed problems as a result of separation of laminates due to the large amount of shrinkage of Australian hardwoods. The shrinkage created gaps that enabled fungal and termite attach in these areas. Replacement of individual laminates was not possible.

The larger and more impressive truss structures attract the attention of the general public.

Livingston Road Bridge
Morwell River, Victoria

Many of the innovations in timber bridge technology were developed through necessity with the expansion of the railway network and the requirement for greater load capacity

It was not these structures that provided the significant tool to the development of the infrastructure network, but rather the humble timber stringer beam bridge with timber trestle substructure. These simple structures with spans in the range 5 to 12 metres provided the backbone to the road and rail network. Timber for these structures was sourced and milled locally and bridges constructed by the local communities.

Timber bridge construction was overtaken by more modem materials and technologies in the early portion of this century. Timber appeared to have lost the appeal to designers and those managing the road network.

Old Timber Bridge - Eastern Victoria

Materials

Durability is perceived by many to be the down side to using timber in significant infrastructure projects.

An understanding of the materials is essential to the correct utilisation of the material, whether timber, concrete or steel or combination of all in composite structures.

Australia was fortunate in that it had many readily available hardwood species suitable for structural applications. The most notable are the Eucalyptus species of which many provided both durability and strength.

Much of the old growth Eucalyptus species were large trees and very suitable for bridge construction applications. Today in Tasmania, the Southern most state of Australia, it is not difficult to obtain clear round logs in excess of 1.2m diameter and 15m plus length.

The availability of this timber is reducing rapidly as the community appreciates the old growth mature forest for its heritage and environmental values.

Many of the Australian Eucalypts have been high strengths with round logs from varieties such as Grey Box, Black Butt and Iron Bark being classified in strength groups of F27 and high durability classifications. These timber species enabled early bridge builders to construct basic bridges easily, cheaply with the benefit of strength and long life. Many of these basic bridges are in service on minor roads today, with little or no maintenance and in excess of 80 years service.

Designers of bridges in Australia today cannot economically obtain old growth mature forest sawn timber in sizes necessary to construct bridges.

Technology has enabled plantation timbers to be utilised through the manufacture of structural timber sections such as Glu-Laminated beams. Preservative treatment of lower durability timbers, both hardwood and softwood, have enabled previously undesirable species to find application in bridge building.

Manufactured structural section with appropriate durabilities are available for bridge building. The imagination of the designer will be the limitation to the application of these products in the competitive market place.

Perceptions of Timber Bridges

Timber bridge construction effectively ceased in the early 1950s due to availability of suitable timber sizes, perceived durability and the advances in steel and reinforced concrete technologies.

Concrete in particular was perceived as providing long term maintenance free structures similar to many old and existing masonry structures. Advances in steel technology provided opportunities for longer span structures with increased load capacities.

Timber bridge technology had not advanced beyond the trusses of the late nineteenth century. Construction and maintenance of these bridges was becoming a problem particularly where the ratio of material cost to labour cost was altering with labour cost proportions increasing over material costs.

The timber bridge was seen as a solution on low traffic volume, small span and remote location sites where there was a local material source.

New Timber Stringer Bridge
Savage River - Tasmania

Poor detailing on many of the short span timber stringer with timber trestle substructures combined with low durability local resourced timber, resulted in the perception

of timber bridges being short term solutions to bridging problems. This perception is still held by many Engineers practicing today.

The timber bridge was being delegated to the boutique market scene where cost was not the driving consideration. This effectively excluded timber bridges from the arterial road networks, with targeted programs to replace existing timber bridge stock.

Timber bridges are delegated to a solution for small pedestrian bridges and low load capacity vehicle structures on the likes of golf courses and private rural farm properties.

The timber supply industry concentrated on the rapidly growing residential scantling market where large volumes of repetitious section sizes would be utilised.

Timber was not seen as exciting in the context of materials available in the market. The result was stagnation in the advancement of technologies which would enhance the use of natures most friendly renewable resources in transport infrastructure projects.

Advancements in Technology

During the 1970s, the extent of the timber bridge asset was being appreciated, not through an appreciation of its functionality, but rather through the cost of maintaining it. Neglect of the asset over decades had resulted in this situation. Replacement of the timber bridge asset was prohibitive in the short term.

The necessity to prolong the serviceability of these assets was required. Like so many progressions in technology, necessity for a solution progressed the cause.

The residential and commercial building market with the potential for large volume turnover lead to the development of manufactured timber products such as Glu-Lam and Laminated Veneer Lumber structural sections.

Developments in preservative treatment of timber products opened the field for use of timber products to previously hostile environments.

Innovation by practitioners in the maintenance field resulted in a better appreciation of the detailing requirements for timber bridges.

Advancements such as “stress laminated timber decks utilising preservative treated or high durability sawn timber has provided a new and exciting tool to maintain existing bridge structures while also increasing their load capacity through. The Canadian initiated “Stress Laminated bridge system has been successfully further developed in the United States to provide a tool to practitioners maintaining and building timber bridges.

Through the generous contribution of the United States Forest Industry, the technology associated with stress laminated timber bridges has been shared with Australia.

The Road and Traffic Authority of New South Wales in association with the University of Technology, Sydney, have worked in adapting this technology to Australian conditions. The timber industry have been supportive of the research.

Advancements in the development of stress laminated technology to new structures is being carried out by the Road and Traffic Authority of NSW and the University of Technology, Sydney. Consultation and technology transfer during the project is occurring between the Australian project personnel and the US Forest products laboratory.

The use of laminated timber beams has been used successfully in Australia to construct pedestrian bridges. These bridges range from single span stringer beam through to large arch structures.

The Maribyrnong River pedestrian bridge is a two-pinned arch restrained by continuos off ramps. The clear span of the arch is 66 metres.

Interest in timber bridges technology in Australia has lead to innovation in small span bridge design utilising composite timber stringer and concrete decks, cable stayed stress laminated pedestrian bridges and stress laminated cellular deck structures.

Timber Arch Pedestrian Bridge
Manbyrnong River - Victoria

Conclusion

The future use of timber in bridges in Australia will ultimately be driven by economics. The solutions

developed by designers to satisfy client requirements will generally be cost driven. The market in volume terms is relatively small when compared to other commercial uses. The small span bridge market is very competitive with solutions using alternative materials such as concrete and steel being perceived by many as providing the more appropriate economic and durable solution.

For timber bridges to achieve success in this competitive market will require the persistence of the dedicated, commitment of suppliers and an appreciation by the consumer of the benefits of using natures most friendly renewable resource.

Timber will not be the appropriate solution for all situations.

With the appropriate commitment from all shareholders in the timber technology community, timber will remain a key resource material for bridge designers and managers of the road network infrastructure asset.

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Timber Bridges in Central Europe, yesterday, today, tomorrow

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Abstract

Historically, limber bridges have played an important role. The decline of timber bridge building was triggered by the development of competing materials: steel and concrete. Governing factors for the revival of limber bridge building in the eighties has been the development of timber engineering, of glued laminated timber (treated and untreated), as well as of the laminated bridge deck.

Keywords: Bridges, competition, concrete, development, design, history, innovations, steel, tradition, timber, timber engineering, trends.

General Remarks

Building bridges is a matter - among other things - of topography, transportation needs, economic potential, cultural heritage, availability of suitable materials and experienced craftsmen, fashions, prestige, preferences and prejudices. Bridges often have a symbolic character and reflect the spirit of the times in which they were built.

Nowadays bridges are usually engineered structures - designed solely by engineers. Architects have only been involved in bridge designs in very few cases - and this has not always been beneficial.
While craft skilled timber structures have shown more or less a steady appreciation and demand on the building market through the last decades, the engineered timber structures, including timber bridges have demonstrated a significant increase over the last fifteen years. This has been especially true for Switzerland and later on to some extend also for neighboring countries.

The background of this development, the reasons and the perspectives shall be explained: Europe - in spite of its small size - is characterized by many climatic, topographical and cultural differences, as well as a considerable variety of building traditions. The following considerations are mainly valid for the alpine region with its major through roads overcoming many obstacles, rivers, streams, gullies, ravines, gorges etc.

Bridges in Retrospective

Bridges certainly played an important role in the transport systems of old cultures, but amazingly little is known about them. Exceptions are Caesar's bridge over the Rhine or Trojan's bridge over the Danube.

Considering the entire history of men, timber has been undoubtedly the most frequently used material for bridge building Even the Romans, who developed a highly sophisticated building technology with natural stones, used a lot of timber not only for bridges but also for other major structures like the Coliseum in Rome. After the collapse of

the Roman Empire their amazing road system broke apart due to the lack of maintenance and many of the building skills of the Remans were lost. More simple methods were applied later on, techniques which had been used at all times by craftsmen for less important crossings. These techniques can be characterized by short span, low load bearing capacity, cheap material, little workmanship and low durability.

Very little is known about the art of bridge building between the Roman times and the Middle Ages (approx. 1200 a.D.). By then the road systems had gained importance again and included safe and durable bridges. From that period, we have considerable evidence of timber bridges, mainly of covered ones. The roof obviously was recognized as being a most efficient means of protecting the bearing structure from the adverse influence of the weather. These bridges mostly had an abutment of stone to separate the timber structure of the bridge from the moist soil. Those builders knew perfectly well that - besides hostile men - rot is the most dangerous enemy of timber bridges - and can be prevented by keeping the wood reasonably dry.

The evidences of those timber bridges are not restricted to pictures and written documents, for the durability achieved with the technology of that time allowed many of these structures to stay in service until now. One of the famous examples is the Chapel-bridge in Lucerne. which lasted until 1993 when most of it was destroyed by a fire.

By the way, this has been the fate of most of the historical timber bridges: They were not destroyed by the ravages of time, but were burnt down in the many quarrels and wars of European history.

The most famous bridge of the builder Hans Ulrich Grubenmann was built in Switzerland in 1758 over the Rhine at Schaffhausen and had a length of 120 m (400ft). Even though longer bridges were built by other builders. the Grubenmann family represents a certain climax of craft skilled timber bridges up to the 19th century.

Then new techniques were introduced, triggered by a increased availability of iron at a considerably reduced price due to new manufacturing methods. New types of structures were introduced in Central Europe, especially of the truss type, like lattice truss or Howe's truss. However, they never became as popular as in North America, where they had been invented and were built by the thousands.

Decline of Timber bridge building

Promoted by decreasing prices the iron and later steel, the technology developed quickly.. Iron and steel became increasingly popular for bridge building and the use of timber declined rapidly in the second half of the 19th century.

Steel and - fifty years later - reinforced concrete represented modern times and technology. The seemingly unlimited new possibilities have edged timber completely out of the minds of ridge-) engineers. Besides the image of being outdated - and not offering any superlatives (strongest, biggest, longest), - timber as a bridge material was disregarded for several other reasons:

The traffic loads had multiplied, urging the use of high strength materials. Scientific research and development resulted in a high level technology of steel- and concrete production. Large and powerful steel and concrete industries developed, absorbed and implemented new know-edge. Tough competition kept (and still keeps) the price of steel and concrete low, and professional training in steel- and concrete technology and design is thorough and well recognized.

In the same period wood and timber engineering remained a research topic of low significance. Production - except that of the wood derived panel products - remained in the hands of numerous local saw-millers and craftsmen, lacking a substantial capital base and innovative drive. Professional training at the crafts-men's level was (and is) satisfactory but traditional - and at the engineers level either modest or non existent.

For at least half a century the art of timber bridge building remained in a state of hibernation, fifty years during which an enormous increase of all building activities took place in the industrialized part of the world - a period strictly dominated by concrete and steel.

Revival of Timber bridge building

A gradual revival of timber bridge building started about 1980. It was based mainly on earlier developments in timber engineering and of glued laminated timber. Numerous innovative projects resulted in some spectacular structures, particularly roof structures for sport stadiums. All these structures have one advantage bridges don't have: The carrying structure is normally well protected from the weather and presents few complications regarding durability.

Even though adequate durability is one of the major tasks in timber bridge construction, there were and

certainly still are more dominating obstacles to a more frequent use of timber. It is well known that, not only in Central Europe, but in most countries the limited knowledge and experience of most design engineers and officials in highway departments leads to a generally negative attitude towards the use of timber for bridges.

A US-American investigation revealed in detail the uncertainties and knowledge gaps of the responsible engineers (1). The study certainly also reflects the situation in Central Europe and many other countries.

The renaissance of the timber bridge demonstrates, that technical as well as mental obstacles can be overcome. Today there arc a considerable number of arguments to encourage the use of timber for bridge construction:

Technical arguments -

- the availability of new types of wood based products with engineered properties

- the fun of rediscovering a "new-old" fascinating material with many interesting properties

- the better knowledge of the properties of wood and the increasing ability to take advantage of specific characteristics of wood in an appropriate manner

new possibilities of modifying wood and combining it with other materials to fulfill special tasks within the timber structures

- new ways and means of manufacturing, quality, control, transporting and erecting wood structures

Other arguments -

- the rediscovery of timber as an environmentally friendly, renewable resource with virtually unlimited supplies

- a fashion-like preference for architectural features expressing the characteristics of timber structures

- the potential of wood and timber structures to promote a certain design quality which cannot be achieved with other materiels

and last but not least

- some loss of image of the materials competing with wood. particularly the increasing awareness that concrete is by far not the everlasting material it was expected to be.

Focal points of Innovations

If we analyse critically the circumstances leading to the current increase in the use of wood as construction material for bridges in Central Europe, wc come to the conclusion that three dominating factors are responsible for it, two of which are of a technical nature:

The *first* factor is the development and use of pressure treated glulam for which a base was laid in extensive tests in the seventies. This has allowed the use of glulam elements in the more hazardous circumstances normally occurring in bridge constructions.

The *second* factor has been the development of the stress laminated bridge deck, a development which started also in the seventies in Ontario. A later development was the glued laminated bridge deck. Compared to former techniques, such bridge decks are characterized by a certain monolithic solidity, allowing an efficient plate action of the deck, i.e. a better load distribution and providing also a solid base for the moisture barrier and the wearing surface (pavement). In view of the everyday problems of bridge maintenance and repair this factor should not be underestimated. Today laminated bridge decks are important for new timber bridges, as well as for retrofitting existing ones.

The *third* and last factors has been a slowly changing attitude of building owners, politicians and especially the responsible officials in highway departments, which lead to a more positive attitude towards the use of timber.

Types of new Bridges

Trying to characterize the timber bridges of the last fifteen years, we can distinguish three quite different categories:

The first type - which I would call *'utility bridge'* - is expected to be cheap, simple to build and to maintain. It is standardized and restricted to the most favourable circumstances for timber (e.g. short to moderate spans). It is durable and has little beauty or design ambitions, but can compete economically with any other bridge material.

The second type is just the opposite and could be called *'designer bridge'*. Its characteristics are bigger spans, a higher level of sophistication, innovative design and the implementation of new and/or unusual ideas. It has esthetic ambitions, but the economy is usually less favorable.

The third type is the *'modernized classical bridge'*. It was the first one of the timber bridge revival. Its appearance bears features of traditional bridge design, especially of covered bridges, but technically it is taking advantage of all modern developments, such as glulam, microlam and other wood based products, pressure treatment, newest types of connectors, prestressing of bridge decks, etc.

Following this simplified classification in reference to highway bridges we may say that the *designer bridge* is at the moment the preferred type in Central Europe. The *modernized traditional bridge* is also popular, but of decreasing importance, not only for price reasons. Up to now the *utility bridge* plays a secondary role in Europe, but this could change rapidly according to local initiatives.

For obvious reasons railway bridges in timber are virtually non existent in Europe.

Pedestrian and other low duty bridges are quite popular in most regions. The far lower loads allow a much greater degree of design freedom, and their development and application follows a different pattern.

Many people have mixed feelings about the construction of new covered bridges. It is a controversial topic. but should be handled from a rational point of view. This supposes a clear distinction between the technical, economical and architectural aspects. Covered bridges are traditional in some regions and fit well in some landscapes and surroundings. They tend to have a heavy appearance, if they carry more than one lane. Even though various samples of this type were built, the majority of covered bridges are rather narrow. To like the architecture of covered bridges or not, is a matter of taste, but all the historical bridges still in service were and are - if not of natural stones - *covered* timber bridges.

Design Trends

There is no doubt that technically the roof is an extremely efficient protection against the adverse impact of the weather. It increases the bridge construction costs noticeably but, on the other hand, clearly cuts expenditures for maintenance - nearly by half according to German investigations (2). This may result in competitive annual costs or life cycle costs.

Undoubtedly the improved durability image of outdoor timber structures - among them also bridges - is based to a considerable extent on chemical treatments for preservation. However, durability is becoming more and more a dominating design factor.

In the last ten years environmental concerns have clearly spoken increasingly against the use of toxic chemicals for wood preservation. This has recently prompted some less experienced designers to omit chemical treatments completely, without taking into account the far reaching consequences: Some design configurations just don't work without the help of chemical treatment, and completely different designs may be necessary. The borderline between feasible and non-feasible is not well defined and depends on a number of factors, but improper approaches may result in substantial damage within a short time. This could quickly and thoroughly destroy the 'durability image' of timber bridges.

The requirement to minimize or to avoid completely chemical preservation clearly reduces the multiplicity of design possibilities and enhances the importance of constructive measures, i.e. structural detailing (see contribution of F. Kropf). Nevertheless one hopes that the innovative drive in timber bridge design observed in the last few years will not lose its momentum.

Looking at other perspectives of Central European timber bridge design - beyond durability considerations - it seems that the engineers have acquired a new freedom of combining the various materials available to them: timber, steel, concrete, wood derived materials, and last but not least high performance fibres, which are increasingly the subject of R&D and full scale implementation also in the building domain.

Composite structures will certainly be one of the distinct trends in the next few years, based on newly developed technologies for joining various materials by gluing or more conventionally with mechanical fasteners. Concrete bridge decks on glulam girders are in many cases a viable alternative and the monolithic deck can provide an efficient weather protection for the underlying timber structure. One of the important details still in need of improvement is the efficient and economic shear transfer between deck and girders.

Closing Remarks

In retrospective there has been a remarkable innovative drive in timber bridge building in Central Europe. Starting from a lower level than North America, there is a clear potential for a quantitative development. However, substantial efforts are necessary to maintain and consolidate the current

rather favourable general attitude towards timber bridge building.

For such an objective it is highly desirable that many engineers with enthusiasm and solid knowledge of timber construction create and try out new attractive designs.

On the other hand it may be less spectacular but still of higher importance regarding the marketing potential that the respective industries get interested in developing, producing and marketing technically and economically convincing utility bridges.

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Timber Bridges in South America

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Abstract

Timber bridges in South America predate the 19th century. This paper provides an introduction of the many types of timber bridges currently used in South America. The five basic types used, which are, the longitudinal beam, frame, truss, arch and suspension superstructures, are presented. Research on new bridge designs, using tropical and reforestation wood species, has been developed in the Laboratory of Wood and Timber Structures, Sao Paulo University in Brazil, on prestressed timber bridge and decks composed with two diagonal layers of sawn wood connected with wood dowels over composed longitudinal beams. We will construct the first prestressed timber bridge in South America in this year using Eucalyptus Citriodora specie.

Keywords: timber, briges, South America, Brazil

Introduction

This paper provides an introduction to the many types of timber bridges currently used in South America. Although decks are technically part of the superstructure, they are addressed separately because of their varied application on many superstructures types.

Beam Superstructures

Longitudinal beams superstructures are the simplest and most common timber bridge type and consist of a deck system supported by a series of timber beams between two or more supports. Bridge beams are constructed from logs and sawn lumber, single or composite elements.

Log Beams

The simplest type of timber bridge in South America is the log beam. It is constructed by placing round logs alternately tip and butt sections. The span of log beam is limited to sizes and truck loads. The clear span of 5 to 12 meters are most common (Figure 1).

In order to support high truck loads, LaMEM developed a longitudinal beam with composite configuration of two logs alternately tip and butt sections and connected with split rings and bolts (Figure 2). This configuration give a stiffness property of 80% of the theoretical value, and was used in several bridges construction (Figures 3 and 4).

Figure 1. Single log beam bridge over Corumbatai river. This bridge is 22 meters long, 11 meters span, consists of eight longitudinal single log beams of eucaliptus specie and was built in 1985.

Figure 2. Composite log beams.

Figure 3. Composite log beam bridge over Ribeirao dos Porcos river. This strut frame bridge is 21 meters long, 15 meters span, consists of four longitudinal composite log beams of eucalyptus specie and was built in 1974.

Figure 4. Composite log beam bridge over Vespasiano river. This strut frame bridge is 32 meters long, 20 meters span, consists of four longitudinal composite log beams of eucaliptus specie and was built in 1983.

Sawn Lumber Beams

Sawn lumber beam bridges that are commonly 20 cm wide and 30 cm deep. Solid timber blocking is placed between beams for alignment and lateral beam support. Sawn lumber beams bridges are limited in span by the availability of lumber beams in the required sizes. The clear span of 5 to 12 meters are most common (Figure 5). Longer crossings are achieved by using a series of simple spans with intermediate supports (Figure 6).

Figure 5. Single sawn lumber beam bridge over Passacinco river. This bridge is 22 meters long, consists of eight longitudinal single sawn lumber beams of eucaliptus specie.

Figure 6. Single sawn lumber beam bridge over Jauru river. This bridge is 60 meters long, 5 meters span, consists of five longitudinal single sawn lumber beams of aroeira specie.

Other type of sawn lumber beam is the longitudinal multi-lattice boarded nailed I beam with diagonally crossed web boards with 2.5 cm thick, nailed together and straightened with stiffening ribs. The chord of the longitudinal beam consists of boards of 2.5 cm thick arranged in two layers with boards on each side of the web in each layer(Figure 7).

Figure 7. Multi-lattice boarded nailed system over Sorocaba river. This bridge is 28 meters long, consists of two longitudinal nailed I beams with diagonally crossed web of Macaranduba specie and was built in 1990.

Truss Beams

Truss beams consists of straight members connected to form a series of triangles. A typical truss superstructure consists of two main trusses. Timber truss beams are constructed in many geometric configurations but the three of the most popular are the bowstring truss, parallel-chord truss and strut frame truss (Figures 8, 9 and 10). The top and bottom chords are constructed of a series of straight sawn lumber members.

Figure 8. Bowstring truss bridge over Tiete river. This bridge was 52 meters long, consisted of two longitudinal bowstring truss beams of Peroba Rosa specie and was built in 1930.

Figure 9. Parallel chord truss bridge over Chinchina river, Colombia. This bridge was 30 meters long, consists of two longitudinal parallel truss beams of Culifierro specie and was built in 1987.

Figure 10. Strut frame truss bridge over Tiete river. This bridge was 60 meters long, consists of four longitudinal strut frame truss beams of Peroba Rosa specie and was built in 1930.

Deck Superstructures

The deck is the portion of the bridge superstructure that forms the roadway and distributes vehicle loads to supporting elements of the structure. The type, thickness, and material of the deck are based on the weight and volume of traffic it must support. Timber decks are typically constructed with sawn lumber planks and composite logs-concrete snabs.

Sawn Lumber Plank Decks

Sawn lumber plank decks are the oldest and simplest type of lumber deck. They are constructed of lumber planks of 3 to 16 cm thick and 25 to 30 cm wide, that are placed flatwise and connected to supporting beams with spikes or wood dowels. The planks are laid in the transverse, longitudinal or diagonal direction and are attached directly to closely spaced beams with spikes or wood dowels (Figure 11 and 12). The planks in laids in diagonal direction are generally composite in two layers of 45 degrees with longitudinal axis of the bridge and 90 degrees among them and attached in the beams with wood dowels (Figure 13).

Figure 11. Transverse sawn lumber plank deck attached to longitudinal beams with spikes.

Figure 12. Longitudinal sawn lumber plank deck attached to longitudinal and transversal beams with spikes.

Figure 13. Diagonal crossed sawn lumber board deck attached to longitudinal beams with wood dowels.

Composite Logs-Concrete Decks

Composite logs-concrete decks are constructed with logs of 20 cm medium diameter fixed in the londitudinal beams using screws and a steel strip of 25 mm wide and 4 mm thickness making a zig-zag between the logs. A concrete and asphalt cover were put over the deck in order to give a uniform surface on the bridge (Figure 14).

Figure 14. Composite logs-concrete deck attached to longitudinal beams with steel strip and lag screws.

Suspension Bridges

Timber suspension bridges consist of a timber deck structure suspended from flexible steel cables that are supported by steel, concrete or timber towers (Figures 15,16, 17). They are capable of long clear spans (over 100 meters) and are normally used only when other bridge types are impractical because of span requirements or when the use of intermediate bents use not possible.

Figure 15. Suspension bridge over Paranapanema river. This bridge is 132 meters long, 80 meters span, consists of two longitudinal parallel truss beams of Itauba specie, was built in 1920, reconstructed in 1985 and repaired by LaMEM in 1989.

Figure 16. Suspsnsion bridge over Tiete river. This bridge is 62 meters long, 31 meters span, consists of two longitudinal composite log beams of Eucaliptus specie, was designed by LaMEM in 1977.

Figure 17. Suspension pedestrian bridge over Piracicaba river. This bridge is 105 meters long, 78 meters span, consists of two longitudinal parallel truss beams of Eucalyptus specie and was designed by LaMEM in 1992.

Concluding Remarks

LaMEM is one of the most important laboratories of timber structures research in Brazil. We developed research, design and construction on the subject of timber bridges. In this year we are constructing the first prestressed timber bridge in South America in Sao Paulo state in Brazil. Our recent draft code in limit states design give the new guidelines to be followed in design and construction of timber bridges. Our reforestation specie recommended to bridges construction is the Eucalyptus Citriodora. Prospectives species are pinus.

Fundamental Structural Behaviour of "Built-up" Stress Laminated Timber Bridge Decks

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Abstract

Following the successful implementation of stress laminated timber bridge deck technology for decks spanning up to 9m, a 3 year research initiative was commenced early in 1995 at the University of Technology, Sydney (UTS), to undertake fundamental research and development of cellular bridge decks using stress laminated timber technology.

This work is funded collaboratively by the Australian Research Council and industry, and has involved both full scale serviceability and ultimate load testing (up to 12.2m) as well as an examination and analytical investigation into fundamental behaviour of cellular and "T" beam structural forms constructed from timber elements. Research is currently being undertaken to extend this technology to applications with clear spans of 25 to 30m.

This paper presents the findings of on-going research at UTS and discusses the implications of it for timber bridges. Research undertaken to date indicates that the technology is structurally and economically viable.

Introduction

The research and development program for Stress laminated timber (SLT) bridge decks commenced at the University of Technology, Sydney in early 1990 and has focused on a technology transfer of the current state of the art from overseas into an Australian context. The purpose of this testing program has been to develop an "engineered system" for both new and replacement bridge decks to the AUSTROADS and Standards Australia design codes, for use by both state and local government authorities in maintaining some 10000 timber bridges that are known to be in service on the east coast of mainland Australia.

This program resulted in the first prototype hardwood bridge spanning 9.6 metres, being constructed over the Yarramundi Lagoon on Main Road 570 in December of 1991.

Subsequent laboratory testing[1] and field monitoring of prototype bridges, has now established SLT technology in Australia. Design, construction and maintenance procedures have now been documented,[2] culminating in the recent publication of a limit states design code and commentary.[3]

However, it has been found both in Australia and overseas, that the application of stress laminated orthotropic plate decks are limited to bridges of up to about 9m span (refer to Table 1). Approximately 60% of bridges in New South Wales exceed this span limit and as such, a definite need exists to be able construct spans in the 11m to 15m range, for the rehabilitation and maintenance of the existing timber bridge asset.

Similarly, in North America, the desirability of larger spanning timber bridges has led to an extensive research and development initiative, investigating

Table 1 - Maximum Possible Spans for Plate decks

Type of timber used: (290x35mm laminates)	Maximum Span: for single span bridge	Maximum Span: for double span bridge	Maximum Span: for multiple span bridge
softwood (F8)	6.8 - 7.0m	7.5m	8.5m
hardwood (F17)	7.0 - 7.2m	8.0m	9.0m
hardwood (F27)	8.0 - 8.2m	9.0m	10.0m

NOTES:
1) softwood data based on F8 Radiata pine, using ingrade stiffness of 12200 MPa
2) all spans are assumed to be continuous over the supports
3) multiple span deck based on 5 spans, with the end spans 1m shorter than the 3 middle spans
4) the deflection limits for the above decks are span / 400 for T44LL and span / 350 for T44LL with DLA.
5) using limits of span / 500 and span / 400 respectively for (4) would reduce spans in Table 1 by approx. 300 to 500mm
6) decks have been modelled using AUSTROADS T44 and HLP320 loads

alternative structural forms other than orthotropic plates. An overview of some of these options, which includes steel wood composite decks and prefabricated deck trusses, parallel cord trusses, "T" beams, box beams and cellular decks, has been reported by Taylor and Ritter.[4] The latter three of these options are generally referred to as "built-up" timber sections.

Of these systems, the "T" beam and the cellular bridge have particular interest for application in Australia, as they utilise material which is readily available from the timber industry and for which reasonable quality assurance and reliability of material properties is available,[5] making the material acceptable for use in timber bridge structures.

"Built-Up" Timber Bridge Decks

Built-up Sections

Experimental work investigating behaviour of "T" beams (figure 1) and "box" beams has been undertaken in the United States at the Forest Products Laboratory in Madison, Wisconsin, and at the University of West Virginia.[6] An alternative structural form to the box is the timber cell, which uses thinner, closer spaced webs (usually made from LVL - laminated veneer lumber) with two transversely laminated flanges, typically constructed from sawn material of between 150 and 200mm in nominal depth, as shown in figure 2.

Recent investigations (as a part of study tour by the authors to North America in 1994), have indicated that the "T' beam form of this construction also has considerable potential for application in Australia.

"T" beam Decks

The first stressed "T" beam was introduced in West Virginia in 1988 following an extensive research end development program, involving laboratory testing at the University of West Virginia. This program resulted in the construction of a 22.3 metre span prototype near Charlestown, West Virginia, which utilises 150mm x 1140mm LVL (laminated veneer lumber) beams and 230mm deep decking.[7]

Figure 1 - General details of "T" beam bridge deck

Subsequently, glulam beams were used in lieu of LVL beams. The timber research program at the University of West Virginia, has as one of its primary aims, the promotion of local species hardwoods. Details of the "T" beam deck are shown in figure 1.

Box Beam Bridge Decks

As a part of the Timber Bridge Initiative, the Forest Products Laboratory (Madison) end the University of West Virginia are also developing a low profile box system using widely spaced thick web members with spacings typically at about 1.2 metres. Several prototype bridges have been constructed in the last few years as a part of an extensive construction program, involving some twenty new bridges in West Virginia.

Cellular Timber Bridge Decks

The cellular concept received some preliminary evaluation by the Ontario Ministry for Transportation in 1988, but the results of this preliminary research were not published and the testing program for this particular work did not proceed.

A prototype bridge based on this preliminary research was constructed for the Ministry of Natural Resources over the Chippewa River in 1993. This bridge has a clear span of 25 metres and a single lane width of 5.3 metres and has an overall depth of 1200mm. The deck consists of 38 x 184mm flange laminates of sawn timber with 65 x 1200mm deep LVL webs at approximately 500mm centres. The bridge was essentially designed as a series of plywood web beams which do not take into account the shear and torsional interactions of the cellular form of construction.

Conceptual work on cellular decks as presented in this paper was commenced by the authors in late 1991, as a response to the need to develop longer spanning SLT bridge decks to meet Australian needs for spans up to 12.2m. This work referenced the research then being undertaken in the U.S. to develop a structurally efficient form which could utilise both sawn timber and LVL / glulam beams being produced from softwood plantations in Australia.

The cellular deck, whilst similar in concept to the box beam, essentially differs, in that it uses more closely spaced and thinner web members, with the webs typically being made from LVL in thickness' from 45 to 63mm and spaced at centres not exceeding 5oomm.

Details of the cellular concept developed by the authors and tested at the University of Technology, Sydney, are shown below in figure 2. The test decks are full scale, single lane bridge decks, spanning up to 12.2m (40ft). These are almost identical to the prototype bridges which have been constructed subsequent to the R & D work.

Figure 2 - Section detail of the UTS cellular deck, utilising Radiata pine flanges and pine LVL webs

Research And Development For Cellular Decks

General Considerations

The first major research program into the structural performance of cellular stress laminated timber bridge decks is currently being undertaken by the authors at UTS. Initial research in 1993 by Crews[8] indicated that decks with cellular cross sections have significant structural advantages provided the stiffness of the flanges and webs are such as to enable the balanced utilisation of their respective strength. In cases where the stiffness of the web beams precludes the effective utilisation of the strength of the flanges a "T" beam form may be more appropriate.

To calibrate analytical models and to determine design parameters it is necessary to understand the torsional and orthotropic flexural characteristics of these structural systems. The testing program at UTS has been tiered, to enable implementation of the technology as soon as possible with a series of prototype bridges, which will then be monitored under field and in service conditions. The field testing of prototype bridges will complement the continuing laboratory based research and development work.

Three full size cellular deck configurations and a series of partial deck component cells have been tested at UTS to date. This 'proof testing' of full size decks has enabled the validation of the cellular bridge concept and has enabled the construction of several prototype bridges in NSW to proceed.

The first cellular bridge deck tested in the laboratory consisted of a 12.2 metre clear span with 140 x 35 hardwood laminates and 63 x 450 deep LVL webs. Whilst preliminary analysis had indicated that the design depth for this particular span should be 600mm, a depth of 450mm was adopted in order to maximise the induced curvatures in the deck. The underlying purpose of this was to quantify the extreme performance characteristics of this particular structural form.

The second series of laboratory tests involved testing 600 deep decks which were constructed of 63 x 600 LVL webs at 485mm spacings, with 190 x 35 treated Radiata Pine flanges (as shown in figure 2). This particular deck configuration was tested for serviceability loading conditions with a span of 9.8 metres and for serviceability and ultimate limit strength state loading conditions on a 12.2 metre span.

Objectives of the Research Program

The primary objectives of this research and development program are to study the behaviour of cellular stress laminated timber bridge decks and to develop appropriate analytical models which represent this behaviour within the service design load range.

The program includes experimental work, analytical modelling and interpretation of the data, and calibration of the analytical models, concluding with the formulation of appropriate design methodologies for the application of this new technology.

The essential components of the project include the following:

- Experimental investigation of serviceability and ultimate limit states for the cellular stress laminated timber deck system. These tests have been completed for 3 full scale decks tested to destruction under laboratory conditions, establishing specific structural behavioural patterns for this technology.

- Identifying the various limit states for the components, e.g. longitudinal shear, interlaminate slip and identification of the limit states for the particular structural systems.

- Deflection serviceability limits for structural components such as the flange laminates, webs and stressing bars.

- Stability and strength limits for the component cells and then for the structural system as a whole.

Scope

The global issues being addressed within the scope of this project include:

- Characterizing material properties.

- Experimental investigation of single cell behaviour such as shear lag effects and strain distributions within a cell and relationship to multi-cell behaviour, based on full scale testing.

- Characterizing the structural behaviour of individual cells up to service load limit stage.

- Theoretical and analytical investigations into the effect of geometric parameters such as centre to centre spacing of webs, aspect ratio of the cell i.e. breadth to depth ratio, based on the characteristics ofa 'typical cell'.

- Analytical investigation of the influence of geometric parameters on the structural behaviour of cells.

- The effects of changing prestress levels.

- Descriptive reporting of web behaviour within a cell, e.g. load distribution width for cells, effects of load distribution before and after web failure.

- Identification of load transfer mechanisms and contributions of the web to structural response of the cell before and after web failure occurs.

- Dynamic responses and long-term cyclic load effects i.e. the effects of sustained and repeated loads on serviceability and the effect of duration of load on strength, taking account of discontinuities and "shakedown" effects.

Results of Full Scale Tests

At the time of writing, the tests completed or currently underway for the cellular stress decks are:

- Full scale tests (both serviceability and ultimate limit state destructive) on 2 single lane cellular decks spanning 12.2m, with depths of 450mm and 600mm.
- Full scale tests (both serviceability and ultimate limit state non-destructive) on 1 single lane cellular deck spanning 9.8m, with a depth of 600mm.
- Serviceability and ultimate limit state non-destructive tests on single component cells, spanning 9.8m with depths of 600mm, 900mm and 1200mm.
- Cyclic T44 serviceability loading of single component cells, spanning 9.8m with a depth of 600mm.
- Testing of deck segments to quantify slip behaviour at web to flange interfaces, between flange laminates and around / at butt joints.

The full scale flexural tests and the proof load shear tests give an indication as to overall structural response of this cellular system in an empirical sense, but do not in themselves permit a clear understanding of the fundamental behaviour.

In order to quantify the latter, component testing of individual cell and various deck segments commenced in May 1995 and is due for completion in August 1996.

Figure 3 - Cross section deflection profiles - 12.2m span deck with 2 point loads of 540 kN, 1200mm apart and symmetrical about midspan

Basic Load Response

Whilst there was a need to undertake tests of lateral and torsional stiffness on the plate decks, the tests undertaken on the full scale single lane cellular bridge decks constructed at UTS, indicate that load eccentricity does not form a major problem for the structural deck system.

The load response behaviour of a cellular deck in the linear elastic range (and even into the non-linear range) is much more akin to that of a wide beam, rather than the orthotropic plate behaviour of the plate decks. This is illustrated in figure 3 below.

In applying the various loading conditions on the cellular decks at UTS, a number of loading scenarios have been modelled including both two point and four point loading in order to effectively model the bending moment envelopes induced by T44 and HLP320 vehicles for both the serviceability limit states and the ultimate limit states, as defined in the AUSTROADS Bridge Design Code 1992.

Serviceability load testing was undertaken at a variety of prestress levels, (1000, 700, and 550 kPa) whilst testing to failure loads was undertaken with a prestress of 700 kPa. For the 600mm deep decks, the design ultimate limit state loading (HLP320) was satisfied at a load level well below the actual "failure" load capacity of the deck.

Additional testing has also been undertaken on short span segments of the cellular decks to quantify punching shear responses and to apply ULS proof shear loads, to ensure that shear capacity of the decks are adequate to meet the AUSTROADS design standards.

Ultimate Load Capacity - Flexure

As indicated above, the first hardwood deck tested in the laboratory, had intentionally used a very small depth to span ratio to induce significant curvature and facilitate the development of clearly defined limit states and to enable the determination of the ultimate load carrying capacity. Because such testing had not been previously undertaken, the failure mechanism and ultimate load carrying capacity were not readily predictable.

The shakedown failure modes were also unknown. The normal design depth for this 12.2 metre span would have been 600mm rather than the 450mm as tested. One interesting phenomenon, which was noted during the tests was the fact that even when all the LVL webs had fractured in flexure, the deck still retained significant load carrying capacity (still carrying 580 kN per wheel path), albeit with very large deflections.

Only one of the hardwood flange laminates exhibited any signs of failure, although interlaminate slip was noticeable at some butt joints, at the higher load levels.

Figure 4 - Load vs. Deflection plots for a 12.2m span Cellular deck (pine flanges, 600mm deep)

Even at this reduced depth, the hardwood cellular deck had an ultimate bending moment capacity of 1622 kNm, which was considerably greater than the 1000 kNm required to meet the T44 ULS for a single wheel path. However, it fell slightly short of the HLP320 wheel path loading, which required a ULS moment capacity of 1740 kNm. The hardwood deck had no problems meeting the AUSTROADS shear load requirements for both T44 and HLP loading.

The tests on the Radiata Pine decks were commenced some six months after completion of the hardwood tests. This enabled certain refinements to be made and the authors had more specific expectations as to the likely performance of the cellular system. Summary results for the 9.8 metre span and 12.2 metre span pine decks are presented in Table 2.

Table 2 - Summary of ULS Tests on Cellular Decks (under flexure)

Deck Type:	actual ultimate BM (kNm)	Design T44 ULS BM (kNm)	Design HLP320 ULS BM (kNm)
12.2m hardwood (450mm)	1622	1000	1740
12.2m softwood (600mm)	3000	1000	1740

Two points of importance should be noted. Firstly, at the "design depth" of 600mm the essential load deflection response is linear up to the HLP320 ULS requirement, as is seen in figure 4. Secondly, the ultimate load capacity of these decks was significantly in excess of (70% above) the design requirement for the HLP320 ULS.

Observed Failure Mechanisms

The failure mechanisms on these particular decks are quite interesting. From the tests and from readings taken at butt joints using highly sensitive LVDT's, it appears that the load distribution mechanism which occurs, is one of horizontal slip of the laminates, particularly at the interface with the LVL webs.

The observed responses show that the theoretical outer fibre stresses in the flange material were well in excess of the fifth percentile MORs for the pine laminates. Interestingly no failures were observed to occur in any of the laminates, both during and then after the test in a post mortem of the deck.

The failures which did occur were essentially localised tensile failures in the lower tensile region of the two LVL webs which failed. The load sharing mechanism is fascinating. It appears that the cellular system continues to take increasing load until such point as the horizontal shear overcomes the frictional resistance induced by the prestressing forces between the flanges and the web elements. When this occurs microslipping appears to occur resulting in significant load redistribution, and a settling in of the flanges with the webs.

Ultimately however, the "distribution width" converges, resulting in extreme stress concentrations in the outer fibres of the central (under load

application points) LVL webs, which in turn ultimately led to minor compression and then tensile bending failures in the lower tensile region of the LVL webs.

Post Critical Behaviour

As can be clearly seen from figure 4, the system behaviour of a stress laminated timber cellular deck is non linear in the lead up to ultimate flexural capacity and over the entire loading range exhibits marked ductility. When failure finally occurred the extent of wood damage was highly localised at midspan in the LVL webs, immediately under the load application points.

However, despite the fact that the principle elements had failed and discontinuity existed at midspan, significant load carrying capacity still existed in the deck well in excess of the design ULS requirements, specified by the AUSTROADS Bridge Design Code.

This was verified by post failure testing of the deck, which was observed to produce essentially the same linear load response up to the AUSTROADS design ULS loads despite the fact that web failure had occurred. This highlights the extraordinary load sharing characteristics of this particular structural system whilst providing excellent serviceability stiffness characteristics to minimise deflections.

Current Status Of R & D

Component Testing

Research being undertaken at the time of writing involves testing on individual component cells as shown in figure 5, with depths of 600mm, 900mm and 1200mm and prestress levels of 1200, 1000, 700, and 500 kPa. Additional load tests have been undertaken on the 600mm deck for prestress levels of 300 kPa (static loads) and 500 kPa cyclic load tests. These series of tests not only characterise the material and component system behaviour for bending, shear, and torsion, but also focus on quantifying the slip interaction mechanism between web and flange laminates.

This slip mechanism is believed to be the critical limiting factor for effective serviceability structural performance and the cause non-linearity at high load levels for cellular sections. The same slip mechanism is believed to be the governing factor for the "T" beam bridges currently in use in North America and a series of "T" beam tests is also planned as a part of the current R&D program at UTS.

The component test cells (as shown in figure 5) have been manufactured using flange laminates which have been finger jointed to avoid the inclusion of butt joints, which were included in the full scale, single lane deck tests.

Figure 5 - Typical Section of cell component beam used in Component Tests. Arrows indicate positions of vertical deflection measurements

This form of construction has been incorporated to deliberately remove the discontinuity effects of the butt joints and to directly relate slippage to the release of strain energy in the deck.

Discussion of Preliminary Results

Analysis of the results will continue until at least the end of 1996. Initial results from the 600mm deep deck indicate that a strong correlation exists between web to flange (horizontal) slip, the commencement of non-linear load deflection behaviour and the onset of any permanent deflection in the deck (vertical slip) after removal of the loads. This is illustrated for a 600mm deck at 700 kPa in figures 6 & 7.

The prestress level has a marked effect on the "stiffness" of the deck and the commencement of the slip mechanisms discussed above. On the basis of the preliminary analysis of data obtained from the component tests, it is strongly recommended that the prestress pressure in cellular decks should not be allowed to fall below 700 kPa, with an absolute lower bound level of 500 kPa. Below 500 kPa the structural behaviour of the deck changes markedly, both in terms of stiffness and ultimate strength.

A linear elastic finite element model has been developed as the major part of a student research project supervised by and with significant input from the authors. This model is being calibrated with test data from the full scale laboratory tests and forms the first stage of development of a more comprehensive analytical model (incorporating the slip mechanism) which will be undertaken as the component testing is completed.

Figure 6 - load vs. deflection at midspan for 6 load cycles ranging from 40 kN / jack to 160 kN/ jack

The T44 SLS occurs at 40 kN per jack, whilst the HLP320 ULS occurs at 95kN per jack

Figure 7 - load vs. Horizontal slip for 6 load cycles ranging from 40 kN / jack to 160 kN/ jack

note the "accumulated damage" after each loading / unloading cycle

Figure 8 - Strain vs. Load charts for laminates and webs in the critical moment zone.

The system behaviour of a cellular deck in the post design ULS loading range is essentially non-linear as a result of interlaminate slip, despite the fact that at these load levels the material behaviour is still essentially linear elastic (certainly up to the design ULS load level). This latter phenomenon is confined by strain gauging of laminates in the critical moment zones of the deck, results of which are presented in figure 8.

Implementation Of The Technology

To date four prototype bridges utilising the cellular timber technology have been constructed in Australia since 1994, details of which have been presented elsewhere.[9] The most recent of these structures has been shown to be very cost effective, despite the fact that the cellular technology is being currently used at the lower end of its potential span capacity. Two of these bridges (designed by Crews) have won Engineering Excellence awards from the Institute of Municipal Engineers in 1995 and 1996. In all cases, the decks span up to 12m and are based on the 600mm deep prototype decks developed and subjected to full scale laboratory testing at UTS.

In two cases cellular decks have been used to rehabilitate existing reinforced concrete bridge decks (at Lansdowne and Ashlea on the mid north coast of NSW) which had deteriorated to the extent that bridge replacement was required. Both sites had decks of almost identical design and age, except the one at Lansdowne had only 4 spans (each about 11.5m), whilst the Ashlea bridge has seven spans, with an overall length of 85 metres. A schematic cross section of the rehabilitation solution is shown below in figure 9.

Figure 9 - Typical Cross Section - Cellular Deck overlay over existing concrete decks.

The cellular deck overlay used to upgrade the Ashlea bridge is almost identical to the one at Lansdowne, except that with hindsight on the first bridge some minor design refinements have been included and the decks bear on composite timber beams rather than poured insitu concrete. The cellular decks were prefabricated in a council depot about 25 km from the bridge and transported to site The total road closure time was 3.5 weeks and final cost of the total deck rehabilitation is expected to be about $450,000 or about 20% of the cost of the replacement structure.

Conclusions

Stress laminated cellular bridge decks constructed from Australian hardwoods and plantation grown softwoods have been shown to respond linearly to short-term static loading within the serviceability and

strength limit state load ranges specified by the AUSTROADS Bridge Design Code 1992.

The transverse flexural-torsional stiffness of the cellular decks provided effective distribution of internal actions. All the decks tested exhibited considerable ductility when subjected to a range of loads substantially greater than the design service loads.

After extensive web failures at ultimate loads all three decks tested retained the capacity to carry the design service loads with only slightly increased deflections.

Component testing of cellular beams has identified various mechanisms and it is expected that analysis of the results from these tests will quantify critical design and limit states performance parameters. It is believed that these results will have application for "T" beam SLT decks and a program of full scale testing of a partial "T" beam deck will be used to validate the applicability of the slip models. This will also be linked with the results from deck segment tests (smaller specimens) aimed at quantifying shear flow around butt joint discontinuities.

Funding which has been recently made available through an Australian Research Council Grant and industry sources, will further extend this technology through the component testing, currently being undertaken at the University of Technology, Sydney. It is anticipated that the major testing program will be completed by the end of 1996, with results available by the end of 1997.

It is anticipated that the results of this testing will quantify the shear flow and slip mechanisms in "Built-Up" decks and permit the development of rational design procedures for cellular and "T" beam stress laminated timber bridge decks spanning up to 30m.

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Pole Creek Metal-Plate-Connected Truss Bridge

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Abstract
This paper summarizes the performance of the Pole Creek metal-plate-connected wood truss bridge constructed in the Fall of 1992 in rural Tuscaloosa County, Alabama. This two-lane bridge consists of two simple spans. Span 1 is a bolt-laminated transverse deck supported by multitruss girders; span 2 is a stress-laminated truss system. A monitoring program on the Pole Creek bridge, initiated shortly after construction, has provided information on seasonal variations in lumber moisture content, stressing bar force, static-load test behavior, and overall condition, After 3 years, the monitoring program indicates that the Pole Creek bridge is performing adequately with no structural deficiencies.

Keywords: Metal-plate-connected trusses, stress laminated, bridge testing

Introduction
In 1988, the U.S. Congress passed legislation known as the Timber Bridge Initiative (TBI). The objective of this legislation was to establish a National program that provided effective and efficient utilization of wood as a structural material for highway bridges. The USDA Forest Service was assigned responsibility for the development, implementation, and administration of the TBI. The three primary program areas established by the Forest Service are demonstration bridges, technology transfer, and research.

In 1992, as part of the demonstration bridge program, a metal-plate-connected (MPC) wood truss bridge was designed and constructed on Old Fayette Road in Tuscaloosa County, Alabama. The bridge crosses Pole Creek, approximately 16 km (10 miles) north of the city of Tuscaloosa. Traffic consists of passenger vehicles and heavy trucks with an estimated average daily traffic of approximately 100 vehicles.

Background
In many areas, the local timber supply is limited to relatively small-diameter trees that can produce structural lumber of limited dimensions. The wide lumber required for solid-sawn longitudinal decks is often available only at a premium price. Thus, design options that allow for smaller lumber sizes can potentially improve bridge economy and utilization of the forest resource.

The Pole Creek bridge is an innovative use of MPC wood trusses, and we believe this bridge is the first application of such components in roadway bridges. The bridge is 12.2 m (40 ft) long with a 8.5-m (28-ft) curb-to-curb width. It has two 6.1-m (20-ft) spans, and structural elements are MPC wood trusses fabricated from nominal 50- by 100-mm (2- by 4-in.) and 50- by 150-mm (2- by 6-in.) Southern Pine lumber treated

Figure 1—Cross-sections of bridge spans.

with chromated copper arsenate (CCA) wood preservative. Span 1 utilizes a bolt-laminated transverse timber deck over multitruss girders; span 2 consists of stress-laminated trusses that form the deck and transfer loads to the substructure (Fig. 1).

The Pole Creek bridge was designed for standard HS20-44 truck loading, based on load provisions of the American Association of State Highway and Transportation Officials (AASHTO), *Standard Specifications for Highway Bridges* (AASHTO 1992). Because bridge design using MPC trusses is not specifically addressed by AASHTO, conservative assumptions were made regarding the distribution of wheel loads. For the girder span, wheel loads were assumed to be distributed according to the provisions for nail-laminated decks on timber stringers. The stress-laminated trusses were designed assuming that the wheel load was not distributed beyond the tire width.

The individual trusses were designed using provisions of the Truss Plate Institute Design Specification (TPI 1985). A series of designs was completed, with the appropriate fraction of the wheel loads located at different locations along the bridge, in order to find the maximum wood stresses and joint forces. The final truss design used the largest plates and lumber sizes required for wheel loads positioned at any location on the bridge.

The design for the Pole Creek bridge is experimental, and the information gained from monitoring this bridge and similar bridges (Dagher and others 1995) should lead to improvements in future bridges of this type. This will, in turn, lead to the development of design specifications for consideration by AASHTO.

This paper summarizes the field performance of the Pole Creek bridge after 4 years of service. Compete details on the development, design, construction, and cost of this bridge will be presented in a document published by the USDA Forest Service, Forest Products Laboratory (FPL).

Research Methods

A 3-year monitoring plan was established to evaluate the field performance of the Pole Creek bridge. In December 1992, approximately 2 months after the bridge was constructed, it was load tested and load cells were installed on three of the seven stressing bars. Since that time, visits to the bridge have been made at least once a month to measure stressing bar force, obtain moisture content readings, and perform visual inspections. The monitoring program concluded with a second load test that was conducted in June 1996.

Bar Force

For the stress-laminated span, calibrated load cells were installed between the anchorage and bearing plates on three of the seven stressing bars (Ritter and others 1991). The stressing bars are 15.88 mm (5/8 in.) in diameter and have high strength steel bars with an allowable load of 129.0 kN (29,000 lb). As part of the construction process, the bars were tensioned four times during the 2 months prior to the installation of the load cells. Load cell readings were taken monthly with a portable strain indicator.

Moisture Content

Moisture content readings were taken using *an* electrical resistance meter at 10 locations. These locations were distributed throughout the bridge and included the trusses on either edge of the bridge, the bottom chord of the trusses beneath the bridge, and the bridge deck of the girder span.

Load Testing

The first static-load test of the Pole Creek bridge was completed in December 1992, and is reported elsewhere (Triche and others 1994, Triche and Ritter 1996). The second static-load test was conducted June 10, 1996, approximately 3.5 years after bridge construction. The testing consisted of positioning fully loaded trucks on each of the spans and measuring the resulting deflections at a series of transverse locations at midspan. Measurements of bridge deflections were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded). At the time of load testing, the average bar force was approximately 53.4 kN (12,000 lb). This bar force produces 275.8 kPa (40 lb/in^2) of transverse compression, which is the

Figure 2—Load test truck configurations and axle loads. The track width of each truck was 1.83 m (6 ft), measured center-to-center of rear tires.

minimum level of prestress recommended for stress-laminated deck bridges in service (Ritter 1990).

Load test vehicles consisted of two fully loaded dump trucks: truck 126 with a gross vehicle weight (GVW) of 358.2 kN (80,520 lb) and truck 132 with a GVW of 331.7 kN (74,580 lb) (Fig. 2). The vehicles were positioned longitudinally on each span so that the two rear axles were centered at midspan and the front axles were off the bridge. For the stress-laminated and girder spans, the vehicles faced west and east, respectively. Transversely, the vehicles were placed for six load cases (Fig. 3). For load cases 1, 2, and 3, the vehicles were placed at the center of the bridge width, with the inside wheel lines 610 mm (2 ft) from centerline. For load cases 4, 5, and 6, the vehicles were placed at the edges of the bridge, with the inside wheel lines 1.83 m (6 ft) from centerline. Deflection measurements from an unloaded to loaded condition were obtained by placing calibrated rules on the underside of the deck and reading values with a surveyor's level to the nearest 0.2 mm (0.01 in.).

Results and Discussion

This paper presents selected results of the monitoring program. These include bar force, moisture content, condition assessment, and load test behavior. Comprehensive analyses of the performance and load testing of the Pole Creek bridge will be published at a later date.

Figure 3—Load test transverse positions (looking east). For all load cases, the rear truck axles were centered at midspan and the front axles were off the bridge.

Bar Force

Figure 4 shows the bar force compared with time after the fourth bar tensioning. Cell 136 measures the force in the bar located at midspan, and cells 135 and 137 are located on the bars adjacent to the midspan bar. Approximately 5 months after the load cells were installed, the stress was removed from the bars, a new load cell zero balance was obtained, and the bars were

Figure 4—Average trend in bar tension force.

Figure 5—Seasonal variations in moisture content from measurements taken on the underside of the stress-laminated span.

retensioned. This was the fifth bar tensioning and is the reason for the sharp increase in bar force between January and August 1993. Since that retensioning, the bar force has continued to decrease but at a much slower rate. As a result, a sixth bar retensioning (not shown in Fig. 4) was performed in June 1996, shortly before the second load test.

The significant and continued loss in bar force is not typical of stress-laminated deck bridges. The loss in bar force for these stress-laminated trusses is primarily the result of two factors: (1) gaps between the trusses caused by nails used to fasten the individual trusses into bundles for handling purposes and (2) small gaps under individual metal-connector plates that were gradually reduced under the transverse stress.

Moisture Content

During the monitoring period, unadjusted moisture content readings ranged from 16%-27%. These values converted to an approximate range of 13%–24% when adjustments for temperature and CCA treatment effects were considered. Moisture content readings were calibrated to ovendry samples and, according to these data, meter readings averaged about 3% greater than those determined by the ovendry method.

Figure 5 shows the variation in moisture content over time for a single location on the underside of the stress-laminated span. The initial moisture content of the lumber was approximately 12% and has gradually increased to an average of approximately 18%. In 1995, Alabama experienced an unusually hot and dry spring and early summer. During this period, the moisture content decreased to approximately 13%. Since that time, the moisture content has increased and is currently near the 18% average. Referring to Figure 4, there was also an increase in bar force loss during the hot and dry period; bar force began to stabilize for the last two readings as moisture content increased.

Load Test Behavior

Results for the second load test of the stress-laminated and girder spans follow. In each case, transverse deflection plots are shown at the bridge midspan as viewed from the west side (looking east). For each load test, no permanent residual deformation was measured at the conclusion of the testing. In additional, there was no detectable movement at the supports.

Stress-Laminated Span— Transverse deflections for the stress-laminated span are shown in Figure 6. As shown, the deflection profiles are approximately symmetric for corresponding load cases. For load cases 1 and 2, the maximum measured deflections occurred under the outside wheel lines and measured 2.0 mm (0.08 in.) for load case 1 and 2.2 mm (0.09 in.) for load case 2. For load case 3, with both vehicles on the span in the same relative positions, the maximum measured deflection of 2.5 mm (0.10 in.) occurred under the inside wheel line of truck 126. Maximum deflections for load cases 3 and 4 were measured under or adjacent to the outside wheel line and were 3.6 mm (0.14 in.) for load case 4 and 2.7 mm (0.11 in.) for load case 5. For load case 6, the maximum deflection was the same as load case 4 and measured 3.6 mm (0.14 in.) under the outside wheel line of truck 126.

Assuming linear elastic behavior, uniform material properties, and accurate load test methodology, the sum of the bridge deflections for the individual vehicles

Figure 6—Measured transverse deflections for the stress-laminated span measured at centerspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

Figure 7—Comparison of results for the stress-laminated span: load cases 1 and 2 compared with load case 3 (top); load cases 4 and 5 compared load case 6 (bottom).

should be the same as the deflections for both vehicles placed simultaneously. Using superposition, the measured deflections for load cases 1 and 2 compared with load case 3 and load cases 4 and 5 compared with load case 6 are given in Figure 7. As shown, the deflections are virtually identical, with only slight differences along the edges of the bridge for load case 6.

Truss Girder Span— Transverse deflections for the truss girder are shown in Figure 8. As with the stress-laminated span, the deflection profiles are approximately symmetric for corresponding load cases. For load cases 1 and 2, the maximum measured deflections occurred in the girder near the outside wheel line and measured 4.7 mm (0.19 in.) for load case 1 and 5.0 mm (0.20 in.) for load case 2. For load case 3, with both vehicles on the span, the maximum measured deflection of 5.6 mm (0.22 in.) occurred near the inside wheel line of truck 126. Maximum deflections for load cases 3 and 4 were measured at the outside girder and measured 8.4 mm (0.33 in.) for load case 4 and 6.0 mm (0.26 in.) for load case 5. For load case 6, the maximum deflection measured 8.6 mm (0.34 in.) at the outside girder near truck 126.

Measured deflections for load cases 1 and 2 compared with load case 3 and load cases 4 and 5 compared with load case 6 are shown in Figure 9. As with the stress-laminated span, the deflections are nearly identical.

Load Test Comparison

In comparing the load test results for the stress-laminated span and the truss girder span, it is evident that the maximum deflections for the stress-laminated span are approximately half those measured on the truss girder span. This response was expected because there are substantially more trusses in the stress-laminated span and the longitudinal stiffness is greater. The plots also indicate that the deflections are much more localized for the stress-laminated span and there is a significant increase in deflection along the edges of the bridge for the truss girder span. Again, this was expected due to the difference in the relative transverse stiffness of the two spans. For the stress-laminated span, transverse stiffness results from the effects of stress laminating. For the truss girder span, transverse stiffness is a function of the stiffness of the nominal 150-mm (6-in.) bolt-laminated deck, and the transverse stiffness is considerably less than that of the stress-laminated span.

Condition Assessment

The general condition of the Pole Creek bridge was assessed at the time of installation and approximately every 3 months thereafter. These assessments involved visual inspections, bridge and component measurements, and photographic documentation, specifically the condition of the asphalt wearing surface, stressing bars and anchorage systems, and the metal-connector plates. Visual inspections revealed only a few minor deficiencies.

Cracks in the asphalt wearing surface were common on the girder span. This span utilized a bolt-laminated deck, and movement, both within and between panels as a result of vehicle loading, caused reflective cracking. The wearing surface for the stress-laminated span, which experienced the same vehicle loading, showed no cracking.

Regarding suitability of MPC trusses for components of a stress-laminated bridge system, several problems are apparent. Most of these are due to the use of a crowned pile cap. When the bridge was placed on the crowned abutment, the bottom of the trusses followed

Figure 8—Measured transverse deflections for the girder span measured at centerspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

Figure 9—Comparison of results for the girder span: load cases 1 and 2 compared with load case 3 (top); load cases 4 and 5 compared with load case 6 (bottom).

the slope of the abutment cap. When the bars were tensioned, the trusses became horizontal and the trusses along the edge of the bridge lifted above the cap. Modified design details, including the use of a flat pile cap and modified stressing bar positioning, could eliminate these problems in future bridges. There are also some instances of local crushing of the lumber under the bearing plates at the stressing bar anchorages.

No instance of corrosion on the metal-connector plates was detected. Corrosion was addressed in the design by using a galvanized coating and providing an impermeable membrane above the trusses. To date, this design decision seems to be adequate in preventing corrosion in this environment.

Concluding Remarks

The innovative MPC Pole Creek wood truss bridge has been in service for approximately 4 years. Bar force loss on the stress-laminated span was substantial at the beginning of the monitoring period but has remained relatively stable for the past 2 years. The bolt-laminated deck used for the girder span resulted in extensive cracking on the wearing surface. Results of this study indicate that this type of deck is not suitable for bridge decks intended to have asphalt wearing surfaces. The use of a glulam panelized deck would greatly reduce the cracking problem on the wearing surface.

Aside from the relatively minor deficiencies previously noted, both spans are performing well. However, the simplicity of the girder span makes it an attractive alternative. The girder span system eliminates the need for stressing bars and the associated maintenance required to restress the bars periodically. It more closely resembles traditional bridge systems, and the girder span is easier and less costly to design and construct. With an alternative deck design, this system has immediate potential for widespread usage.

Further investigation of the stress-laminated span is needed to determine the required positioning of stressing bars to prevent unequal compression between the top and bottom chords of the truss that cause out-of-plane bending. The requirement for periodic bar tensioning is a disadvantage of this system.

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Fatigue Design Criteria of MPC Wood Trusses for Bridge Applications

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Abstract

Lightweight metal plate connected (MPC) wood trusses are commonly used in residential and commercial roof construction. These trusses also offer advantages for low-volume rural bridges such as ease of construction, high stiffness and wide availability. MPC trusses can be used in bridge applications provided that proper consideration for fatigue, corrosion protection, and detailing to prevent connector plate 'back-out' are incorporated into the MPC joint design. Using fatigue test results of 172 individual MPC joints and 34 fill-scale trusses, recommendations for fatigue design are given. Two MPC truss bridges, 46 ft and 39 ft long, built in Maine in 1993 and 1994 are described briefly.

Keywords: bridges, timber, trusses, fatigue, connectors, metal, corrosion

Introduction

In many parts of the United States the highest-capacity wood structural member, fabricated locally, is the metal plate connected (MPC) truss. MPC trusses are lightweight wood trusses assembled using dimension lumber and metal connector plates. They are commonly used for framing roofs and floors in both commercial and residential buildings.

MPC trusses are cost-effective because they are constructed using widely available dimension lumber, are simple to fabricate, structurally efficient and easy to handle. MPC trusses also offer high stiffness, which is particularly important in bridge construction. Until recently, however, these trusses have not been used for constructing timber bridges (Trish et al., 1994; Dagher et al., 1992, 1994, 1995; Altimore, 1995). Several ways in which MPC trusses can be used in timber bridges are shown in Dagher et al. (1994).

In 1991, the University of Maine and the USDA Forest Products Laboratory initiated a feasibility study to investigate the use of MPC trusses in low-volume rural bridge applications. The one-year study examined a number of issues including economics of MPC truss bridges, potential structural systems, and fatigue and corrosion of MPC joints. The positive results of the feasibility study warranted a more detailed investigation (Dagher et al., 1992). Through the detailed investigation in 1993-94, fatigue design criteria for MPC joints in timber bridges were developed (Dagher et al., 1994). In addition, two demonstrations stress-laminated MPC

truss bridges were constructed in Maine and tested (Altimore, 1995). In 1995 and 1996, the proposed fatigue design criteria have been refined through fatigue testing of full-scale trusses.

This paper summarizes the research results and addresses the following design concerns:

1. Fatigue: Truss joint connector plates are commonly made from 16, 18 or 20 gage steel. Plates are punched from sheets with a die to form teeth that are an integral part of the plate. The punching process causes stress concentrations in the thin plate and makes them more susceptible to fatigue.

2. Corrosion: Steel sheets are commonly galvanized before plates are punched. After punching, unprotected steel is exposed in the critical areas around the roots of the teeth. Stainless steel plates would reduce corrosion concerns, but are prohibitively expensive.

3. Plate 'back-out': Like nails, connector teeth can 'back-out' (i.e., partially withdraw) when the wood in which they are embedded is subjected to repeated cycles of wetting and drying.

Fatigue Testing of Individual MPC Joints

A limited amount of high-cycle fatigue test data on MPC joints is available in literature. In the United States, Sletteland (1976) conducted experiments on the fatigue of metal plate connectors. Hayashi and colleagues (1979, 1980, 1981, 1983 and 1984) conducted fatigue studies on truss connector plates at the Wood Research Institute in Kyoto, Japan. Significant differences in the results obtained by Sletteland and Hayashi made it difficult to draw solid conclusions on fatigue design of MPC joints based on the available test data (Dagher et al., 1992). As a result, the University of Maine embarked on a comprehensive study of the fatigue of MPC joints. One hundred seventy-two high-cycle fatigue tests on MPC wood joints were conducted. The tests were typically carried out to two million cycles, the residual strength of the connector in tooth holding, plate tension and plate shear were determined (TPI 1978, 1989; CSA, 1980).

Test parameters included:
a) load and plate direction with respect to the grain of the wood;
b) plate manufacturer;
c) galvanized versus stainless steel plates;
d) plate thickness;
e) plate confinement (various levels of mechanical confining pressure were applied to the joints)
f) size and geometry of the joint, including the number of rows of teeth in the connector;
g) speed of fatigue testing; and
h) magnitude of fatigue load.

Also, some specimens were moisture-cycled prior to fatigue testing. Moisture cycling was conducted by R. Wolfe at the Forest Products Laboratory. The effects of moisture cycling on the static strength of MPC truss joints has been studied by Wilkinson (1966), White (1977) and Groom (1994).

In addition to the fatigue testing, 155 static strength tests were conducted to provide a reference point to determine the residual strength of the fatigue-tested joints. Details of the fatigue testing study are given in Dagher et al. (1992, 1994).

Fatigue Design Methodology for MPC Joints

Using the results of the individual joints testing program, a fatigue design methodology for MPC truss joints for use in bridge construction was developed. The fatigue criteria are simple to use. It is proposed that fatigue design of MPC truss joints be conducted as follows:

First, the joints are designed for total stress using the plate manufacturer's allowable static design values.

Check 1: Static design for total stress:

(DL + LL + Impact + etc....) stresses < allowable stress of metal plate connector stated by maker

Next, empirically-derived reduction factors $ø_{th}$, $ø_{pt}$, and $ø_{sh}$ for fatigue strength are applied as follows:

Check 2: Fatigue design for LL + Impact only:

(LL + Impact) stresses < $ø_{th}$ (allowable static tooth holding)

< $ø_{pt}$ (allowable static plate tension)

< $ø_{sh}$ (allowable static shear values)

in which:

$ø_{th}$ = reduction for manufacturer's allowable static tooth holding value(s)
$ø_{pt}$ = reduction for manufacturer's allowable static plate tension value(s)
$ø_{sh}$ = reduction for manufacturer's allowable static shear values

The net effect of the fatigue reduction factors $ø_{th}$, $ø_{pt}$ and $ø_{sh}$ is to increase the size of the MPC joints above and beyond what is required in static design, thereby increasing the fatigue resistance of the trusses.

For timber bridge applications, it is proposed that the three fatigue reduction factors $ø_{th}$, $ø_{pt}$ and $ø_{sh}$ be selected to provide adequate strength of the MPC joints after the application of one to two million cycles of the design truck loading. This is a severe fatigue design requirement for most timber bridge applications, particularly since most timber bridges are on low-volume rural roads with low Average Daily Truck Traffic (ADTT). According to AASHTO LFD (American Association of State Highway and Transportation Officials - Load Factor Design) requirements for fatigue design, the two million cycles fatigue design requirement applies to ADTT counts comparable to interstate highway bridges. Most timber bridges intended for this application would have ADTT counts that would place them in the 300,000 cycles or less AASHTO fatigue design category. The one-to-two million cycles design suggestion used here accounts (indirectly) for the interaction between fatigue and corrosion of the MPCs.

Fatigue Testing of Full-Scale Trusses

Using the fatigue test results of the individual MPC joints, preliminary empirically-derived reduction factors $ø_{th}$ = 0.60, $ø_{pt}$ = 0.35 and $ø_{sh}$ = 0.35 were selected for further evaluation. The evaluation program included fatigue testing of full-scale trusses designed using the proposed fatigue reduction factors, as well as the fatigue testing of static-designed trusses. The general configuration of the trusses used in the foil-scale laboratory fatigue study is given in Figure 1.

The fatigue testing was conducted at a rate of five Hz. Clearly, in the interest of time, it was desirable to conduct the testing as quickly as possible since it requires 4.6 days to reach two million cycles at five Hz. However, five Hz was the fastest value that could be used while keeping the dynamic effects related to the inertia of the truss within acceptable limits. To study stiffness degradation over time, the test control equipment was programmed to interrupt the fatigue testing at regular intervals and to conduct a quasi-static, data-taking load cycle. LVDTs were used to measure maximum truss deflection during the data-taking cycles. All fatigue testing was conducted up to two million cycles or up to failure, which ever occurred first. For trusses that reached two million cycles, the fatigue testing was interrupted and the static residual strength of the trusses was determined.

Figure 1 -Configuration of Truss Fatigue Test

Table 1 - Results of full-scale fatigue testing of trusses.

Truss Group	Number of Trusses	MPC Joint Design	% Trusses that Exceeded 2 Million Cycles	% Trusses that Exceeded 1 Million Cycles	Mean Stiffness at 50,000 Cycles (lbs/in)	Mean Stiffness at 2 Million Cycles (lbs/in)	% Stiffness Loss at 2 Million Cycles
1	15	Fatigue, tension chord splices at panel points	66	86	5,020	3,515	30
2	9	Fatigue, tension chord splices between panel points	78	100	4,930	3,765	24
3	10	Static design	0	0	4,610	0	100

The 34 trusses that were fatigue tested were divided into three groups as shown in Table 1. All joints were sized to resist the two 900 lb. concentrated loads shown. Therefore, every joint was critical for the load configuration shown. The only difference among the three groups of trusses is how the joints were designed.

The MPC joints in the fifteen Group 1 trusses were designed for fatigue using the reduction factors $ø_{th}$ = 0.60, $ø_{pt}$ = 0.35 and $ø_{sh}$ = 0.35. Also, the tension chord splices were placed at the panel points as is often done in practice today. The trusses in Groups 1 and 2 were identical except that the tension chord splices in Group 2 were relocated between the panel points to reduce the stresses in the joint at the panel points. This was done because the fatigue failures in Group 1 trusses tended to occur at the tension chord splices located at the panel points. The joints in the Group 3 trusses were not designed for fatigue, In other words, all the reduction factors used in designing the joints in the Group 3 trusses were $ø_{th} = ø_{pt} = ø_{sh}$ =1.0, as is commonly done in static truss design today. The size of the MPC joints in the Group 3 trusses were therefore smaller than those in Groups 1 and 2 and less resistant to fatigue.

The results of the fatigue testing are summarized in Table 1. The Table gives average test results for each of the three groups of trusses. The variability of the results was significant, as one would expect in fatigue testing. In general, the Table shows that the proposed design criteria, using the reduction factors $ø_{th}$ = 0.60, $ø_{pt}$ = 0.35 and $ø_{sh}$ = 0.35, give a mean fatigue strength between 1 and 2 million repetitions of the static design load. Smaller values of the reduction factors increase the fatigue strength. As indicated by comparing Groups 1 and 2 trusses, attention to joint detail to minimize stresses within the joints will also improve the fatigue design strength.

Developing rigorous probability-based fatigue reduction factors is difficult at this time considering the high variability of the fatigue strength data and the limited amount of test data currently available. It should also be pointed out that other more rigorous approaches to fatigue design were considered. For example, a fatigue design approach may attempt to estimate the maximum stress ranges within the MPC connectors, accounting for factors such as stress concentrations, eccentric moments, shear tension and compression stress combinations within the joints.

Such an approach, while more proper, would be too cumbersome for everyday design, particularly since the proposed semi-empirical methods appear to work well.

The example shown in Table 2 illustrates the application of the proposed fatigue design criteria. The example uses actual values for 20 gage plates in southern pine from two MPC manufacturers, Alpine and MiTek. The first two rows of the Table show allowable static design values for the plates as stated by Alpine and MiTek. The third row shows the proposed fatigue reduction factors. The fourth and fifth rows show the reduced allowable design values to account for fatigue. Values in the first two rows are then compared with total stress (i.e. with stresses resulting from any working stress design load combination). Values in rows four and five are compared with live and impact stresses only.

It is important to note that the proposed fatigue design criteria were developed primarily by testing Alpine and MiTek's 20-gage connector plates. It is likely that the reduction factors proposed here for fatigue will change from one type of plate to another and from one manufacturer to another. This is because the stress distribution in the plate is a function of the location and geometry if the punched teeth, which vary from plate type to plate type (Mirsa 1964, 1966). Before utilizing these criteria, it is therefore essential that plate manufacturers verify the applicability of the criteria to their plates. Standard fatigue tests similar to the ASTM D1761 fastener static test (ASTM, 1986) should be developed in the future to accomplish this.

Demonstration Bridges

Two demonstration stress-laminated MPC truss bridges, funded through the ISTEA (Intermodal Surface Transportation Efficiency Act) Federal Highway timber bridge demonstration program, were constructed in Maine. The first bridge, located in Byron, is 46 ft. long, 32 ft. wide and has an 18 degree skew. Located in North Yarmouth, the second is 39 ft. long, 31 ft. wide and has a 12 degree skew. Both bridges were designed for AASHTO H25 loading and have similar configurations. Therefore, only the Byron bridge is described below.

Table 2 - Example of proposed fatigue design criteria

	Design Values	Tooth Holding (lb/2si)[a]		Plate Tension (lb/2li)[b]		Plate Shear (lb/2li)	
		AA	AE	AA	AE	AA	AE
Alpine 20 Gage (Net Area)	Static. Compare with any load combination	444	324	813	797	622	418
MiTek 20 Gage (Gross Area)	Static. Compare with any load combination	258	216	1,406	875	758	607
Fatigue Reduction Factors		0.60	0.60	0.35	0.35	0.35	0.35
Alpine 20 Gage	Fatigue. Compare with LL and Impact	266	194	285	279	218	146
MiTek 20 Gage	Fatigue. Compare with LL and Impact	154	128	492	306	265	212

[a] Lbs of load per square inch x two plates
[b] Lbs/linear inch x two plates

The Byron bridge is a stress-laminated, solid truss design, with equal numbers of structural and spacer trusses. The trusses were lifted onto the abutments in pre-assembled modules held together by nails and metal straps. Placed parallel to traffic, structural and spacer trusses are alternated side-by-side to form the width of the bridge. They are stressed in the transverse direction using epoxy-coated post-tensioned rods. A spacer truss is a 'stripped down' truss with only enough webs so that it can be handled safely as a unit. Spacer trusses prevent metal-on metal contact of the connector plates in adjacent trusses and prevent distortion of the bridge during stressing. Side-by-side placement of the trusses and post-tensioning also prevents the plates from backing out due to moisture content fluctuations in the lumber and fatigue loading. Metal connector plates were galvanized before punching and brush-coated with an epoxy paint after the trusses were manufactured. This protection system is one of five recommended for use on metal plate connectors by the Steel Painting Council to yield a 70 year life in a "sheltered marine environment" (Bruno et al., 1989). Additional protection of both the plates and wood against water is provided by a membrane placed on top of the trusses and paved with asphalt. The bridges have no separate deck since traffic runs directly over the top chords of the trusses.

The Byron bridge was load-tested and opened to traffic in November 1993. The North Yarmouth bridge was load-tested and opened in June 1994. The load tests verified the high stiffness of this bridge system. Under two 70,000 lb. trucks positioned so as to maximize deflection, the measured deflections in both bridges were less than L/2300 (Altimore, 1995).

Concluding Remarks

Metal plate connected wood trusses offer an attractive alternative for low-volume rural bridges. Their advantages include wide availability, ease of handling, speed of construction, and high stiffness. Proper fatigue prevention of plate 'back-out' and corrosion protection, as described in this paper, are essential to ensure longevity of the system. The fatigue design methodology described in this paper is based on tests conducted on individual joints and has been verified by conducting tests on full scale trusses. While the proposed design methodology applies to different makes of plates, the actual values of the fatigue reduction factors will vary from one plate manufacturer to another and will depend on the level of safety required in the design.

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Stress-Laminated / Steel T-Beam Bridge System

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Abstract

The stress-laminated timber bridge deck has been successfully used for short span bridges. A design philosophy, procedure, and related equations have been developed for a timber-steel composite deck design which may offer improved economy and performance where longer spans are required. The design incorporates a series of inverted steel T-beams within the stress-laminated timber deck. A significant number of currently used design specifications and construction methods for stress-laminated bridges along with a design procedure familiar to most bridge engineers have been included in this design procedure. Large shear forces between the deck and steel beams require steel dowels. A grid system composed of the steel T-beams and wooden diaphragms provides additional load distribution, and may facilitate the construction process. Standard lumber sizes and relatively unskilled labor may be used to construct the bridge. It is anticipated that the design's lightweight nature may result in significant cost savings for the substructure and construction equipment required. Efficient use of labor, materials and existing design practices make this design advantageous for many applications in the 12.2m (40 ft) to 24.4m (80 ft) span ranges.

Keywords: Stress-laminated, timber bridges, prestressed timber, dowel, composite, T-beam.

Introduction

The timber-steel composite bridge deck is a new type of bridge superstructure being developed by the Civil, Agricultural and Geological Engineering Department at New Mexico State University. The system utilizes a stress-laminated timber deck with inverted steel T-beams for added stiffness (Figure 1). The added stiffness provided by the T-beams will allow longer spans than those possible with the conventional stress-laminated deck. The design process described in this report will utilized as much of the existing technology used to design stress-laminated decks as possible. The design methodology for concrete-steel composite bridge systems is familiar to most bridge designers. Therefore, the design of the steel T-beam and the method of shear transfer between the timber deck and the steel T-beam is similar to currently used design procedures.

This report describes the results for Phase I of this project. Phase I consisted of developing design procedures and relationships for a simple span bridge. Phase II & III, if funded, will involve testing a model bridge to verify and modify design parameters, and finally, a demonstration bridge will be built and monitored.

The philosophy used in developing this design procedure and related methodology centers on utilizing as much of

the currently used bridge design practices as possible. This will insure that development costs are minimized and that bridge design engineers will be familiar with most of the concepts used in this design procedure, Additionally, the procedure developed will allow the bridge designer considerable latitude in adapting this type of bridge to a particular application. This will promote designs which are safe, reliable, and cost effective.

Figure 1—Timber-steel composite bridge deck.

Scope of Study

This study was limited to developing a bridge system for a simple span approximately 12.2m to 24.4m (40 - 80 ft) in length. T-beam spacing is in the range of 1.22m - 1.83m (4 - 6 ft) with deck thicknesses ranging between 203mm to 305mm (8 to 12 in.) nominal. These spacing and thickness ranges allow the designer sufficient latitude in developing an efficient design while restricting the design to predictable behavior.

Overview of Design Procedure

The design procedure will focus on designing an individual T-beam unit which consists of one steel "T" section and its associated deck section as, shown in Figure 2.

The recommended basic design procedure is as follows:

1. Specify initial parameters.
   a. Span length.
   b. Design vehicle.
   c. Wearing surface.
   d. Number of lanes.
   e. Clearance, if applicable.

2. Select material types and calculate allowable stresses.
   a. Wood species and grade.
   b. Steel.
      1) T-beam.
      2) Dowels.

3. Calculate maximum shear and moment acting on bridge.

4, Design composite T-beam unit for maximum moment.

Figure 2— T-beam design unit.

a. Choose deck thickness.
b. Choose T-beam spacing.
c. Determine distribution factor.
d. Determine effective flange width.
e. Select trial size for steel T-beam.
f. Determine non-composite stresses due to dead load.
g. Determine composite stresses due to live load.
h. Repeat steps d-g for outer T-beam units.
i. Check diaphragm spacing.
j. Check web shear.
k. Revise as necessary.

5. Check global deflection at midspan.

6. Check local effects.
   a. Deck.
      1) Transverse deflection.
      2) Transverse bending stress.
      3) Punching shear.

7. Design dowels for horizontal shear.
   a. Check bearing stress on the web.

8. Design diaphragms.
   a. Check longer spacings.
   b. Size the diaphragm.

9. Design prestressing system and guard rails.

10. Check camber.

Development of Design Equations

Previously developed equations for predicting the effective flange width and distribution factor for other types of prestressed timber deck systems were found to be inadequate for the timber-steel deck system. Therefore, design equations of effective flange width and transverse distribution factor were developed for this system. A butt joint factor of 1 in 4 is recommended for all designs. Butt joint factors higher than this are probably not economical since deck forces usually do not control the design.

Effective Flange Width

A number of finite element models were generated and evaluated to determine the effective flange width. These models were evaluated for decks of 203mm, 254mm, and 305mm (8", 10", and 12") nominal thickness, and T-beam spacings of 1.22m, 1.53m, and 1.83m (48", 60", and 72"). Two layers of solid elements were used to model the deck. The appropriate orthotropic elastic constants for Douglas-fir were used in the solid element models. Frame elements were used to model the steel T-beam.

Results of the finite element studies indicated that effective flange width varied between 0.79 and 0.86 of the T-beam spacing. In the design process, the effective flange width will be transformed into an equivalent steel width so that stresses may be calculated. In most cases a modular ratio corresponding to a butt joint frequency of 1 in 4 would be used to calculate the transformed effective flange width, B_e. The slight variation in effective flange width from 0.79 to 0.86 will become insignificant after transformation using the modular ratio. Thus, to simplify calculations, an effective flange width of 0.80 is recommended for all deck thicknesses and T-beam spacings.

Distribution Factor

When a vehicle is placed on a bridge, there is a lateral distribution of its weight. This lateral distribution depends in part on the stiffness of the deck and girder system. In the bridge design process, a single T-beam unit is designed and loaded with a single wheel line. The distribution factor is intended to account for the lateral load distribution when using the single T-beam as the design unit.

Finite element models were constructed for complete composite T-beam bridges and for the corresponding single T-beam design unit. The model for the entire two lane bridge was loaded with two design vehicles so that the maximum moment was produced in an interior girder. Similarly, the single T-beam design unit was loaded with a wheel line of the design vehicle such that the maximum moment was produced. The ratio of these maximum moments is the distribution factor.

Table 1—Recommended distribution factors.

T-beam Spacing	Deck Thickness	Distribution Factor
1.22m (48")	203mm - 305mm (8 - 12 in.)	0.8
1.53m (60")	254mm - 305mm (10 -12 in.)	1.0
1.83m (72")	254 mm - 305mm (10-12 in.)	1.1

Finite element studies using nominal deck thickness of 203mm, 254mm, and 305mm (8", 10", and 12") with T-beam spacings of 1.22m, 1.53m, and 1.83m (48", 60", and 72") and loaded as described above were conducted to determine the distribution factor. The 203mm (8 in.) deck was found to be unsatisfactory for a T-beam spacing of

1.83m (72 in.) due to excessive transverse bending deflection and other local effects and was, therefore, not considered further. Table 1 below summarizes the findings. A linear interpolation is recommended for T-beam spacings between those listed in the table.

Horizontal Shear

A large horizontal shear force exists between the steel T-beam and the wood deck. The friction resistance which results from the prestressing force is of insufficient magnitude to resist this force. Steel dowels were selected to carry the additional shear force not carried by friction. The design for these dowels is similar to that used to design the shear studs for a concrete-steel composite section. Following AASHTO (AASHTO, 1993) procedures, the total shear to be resisted by the dowels and the friction force is the compression force in the flange when loaded for maximum moment. This shear force is distributed from the point of maximum moment to the point of zero moment. Since the maximum moment occurs near the centerline of a simple span bridge, one-half the span length may be used. An appropriate number of dowels are selected and spaced uniformly within this portion of the span. The same number of dowels are used in the remaining half-span.

The design of the dowels utilizes current yield limit theory for bolted connections (NDS, 1991). A nut and washer may or may not be used on the ends of the dowels depending on the construction methods used. However, the design strength used for these dowels assumes that a nut is placed on each end of the dowel since the prestressing force will serve as the clamping force for those cases where nuts are not used.

The design of the dowels assumes a double shear type connection with a steel center member. With this conjuration only NDS equations 8.3-2, 8.3-3, and 8.3-4 cm-responding to modes I_s, III_s, and IV apply (Breyer, 1993, NDS, 1991). Pending experimental studies, the thickness of the side member may be taken as one-half the effective flange width, B_e, since data is not available for prestressed layered side members. The larger side member thickness effectively eliminates all failure modes except mode IV. Thus, NDS equation 8.3-4 is used for dowel design:

$$Z = \frac{D^2}{1.6\,K_\theta}\sqrt{\frac{2\,F_{em}\,F_{yb}}{3\,(1 + R_e)}} \qquad (1)$$

where Z is the nominal bolt design value(lbs); D is the nominal bolt diameter (inches); $K_\Theta = 1 + (\Theta_{max} / 360°)$; Θ is the maximum angle of load to grain; F_{em} is the dowel bearing strength of main member (psi); F_{yb} is the bending yield strength of bolt (psi); $R_e = F_{em} / F_{es}$; and F_{es} is the dowel bearing strength of side members (psi).

Steel "T" Section

Preliminary studies have indicated that standard WT sections may not be economical in many cases. This is due to the relatively shallow depth of the web, thereby requiring the use of sections with large web and flange thicknesses to obtain an adequate stiffness. Although cost studies are not part of this project, it appears that a built-up "T" sections may be the most economical for most cases, A standard WT section with a web extension is an alternative, but it appears to be more costly than the built-up section. A final possibility which was not investigated is a standard W section with the top flange removed and welded to the bottom flange creating a "T" section with a relatively large bottom flange. In any case, these decisions should be left to designer so that he/she may take advantage of the local material supply.

Diaphragms

Diaphragms are recommended for the design for three reasons. First, sidesway web buckling (Summers, 1982) is a possible failure mechanism for the T-beams. To prevent the possibility of this type of failure, diaphragms should be used at spacings equal to or less than those given by the AISC-ASD formula C-K1-1 (AISC, 1989):

$$\frac{d_c / t_w}{l / b_f} > 2.3 \qquad (2)$$

where d_c is the clear unsupported distance between flanges (inches); t_w is the web thickness (inches); l is the largest laterally unbraced length along either flange at the point of load (inches); and b_f is the width of bottom flange of the steel T-beam (inches).

Second, diaphragms are useful for increasing the lateral distribution of loads. Finite element studies indicate that this lateral distribution increase may be substantial in some cases. Finally, diaphragms are useful in the construction process as discussed later in this paper.

Wood diaphragms were chosen for use in this design for two reasons. First, the possibility of fatigue cracks developing at the web-diaphragm connection when wood is used for diaphragms is substantially reduced compared to steel. The fatigue cracks which often form when steel diaphragms are used result from out-of-plane bending of the girder web and from metallurgical changes caused during the field welding process. This out-of-plane bending is the result of differential vertical movement of adjacent girders, Wood diaphragms avoid both of these problems since no welding is required, and because the connection will be semi-rigid in nature, out-of-plane bending in the steel T-beam web is minimized (Fisher and Mertz, 1985).

Figure 3—Diaphragm or end block assembly.

Second, wood diaphragms have the same coefficient of thermal expansion as well as the same rate of shrinkage and swelling with moisture content changes as the wood deck has. Thus, there are no transverse differential dimensional changes induced in the bridge due to moisture or thermal changes.

The wood diaphragms should be connected by one prestressing rod of the same size as those used in the deck (Figure 3). They should also have one dowel in them to carry the excess shear not carried by the friction force. Wood diaphragms connected in this manner are semi-rigid in nature and minimize out-of-plane bending in the T-beam web.

Transverse Deflection

Pending experimental studies, the transverse deflection may be calculated using the equation developed by West Virginia University for the stress-laminated T-system timber bridges (Davalos and Salim, 1993):

$$\delta_{max} = \frac{P S^3}{4 K_\delta E_t t^4} \tag{3}$$

$$K_\delta = -10.9 + 7.8 \frac{S}{t} + 0.27 \frac{E_L'}{E_t} \tag{4}$$

where δ_{max} is the maximum local transverse deflection (inches); P is the rear axle wheel load (lbs); S is the T-beam spacing (inches); E_t is the transverse modulus of elasticity of the deck (psi); E_L' =E' is the allowable longitudinal modulus of elasticity of the deck (psi); and t is the deck thickness (inches).

Maximum transverse deflection of the deck between the steel T-beams should be limited to approximately 0.1 to 0.2 inches (Davalos and Salim, 1993).

Transverse Bending Stress

The transverse bending stress may be calculated using (Davalos and Salim, 1993):

$$\sigma_{max} = \frac{3 P S}{2 K_\sigma t^3} + f_p \tag{5}$$

$$K_\sigma = 3.0 + 3.1 \frac{S}{t} + 0.15 \frac{E_L'}{E_t} \tag{6}$$

where σ_{max} is the maximum transverse bending stress (psi); and f_p is the operational prestress level (psi).

Punching Shear

The punching shear for the deck is calculated by the following equation (AASHTO, 1993, AASHTO 1991, and Davalos and Salim, 1993):

$$V = \frac{P h}{(b_f + 2t) C_{bj}} \tag{7}$$

$$C_{bj} = \frac{j}{j + 1} \tag{8}$$

where V is the calculated punching shear force (lbs); h is the thickness of the deck lamina (inches); and b_f is the tire contact length perpendicular to span (inches).

Bearing Stiffeners

Steel bearing stiffeners are required for welded girders (AASHTO 10.34.6.1) and for rolled girders when the shear stress in the web adjacent to the bearing exceeds 75 percent of the allowable shear stress (AASHTO 10.33.2), These stiffeners serve to prevent failure in the web due to local web yielding and web crippling, which is a form of web buckling (Salmon and Johnson, 1990). The stiffeners are required to extend from the bottom flange to the top flange and to be welded to the web and to each flange, Since the wood deck serves as the top flange in this design it would be very difficult for the designer to design a functional

connection between the deck and the bearing stiffener. For this reason, wood end blocks are recommended for this application. These end blocks should be constructed of the same material as the deck and should fit tightly between the bottom of the deck and the top of the bottom flange of the "T" section. They should extend from one "T" section to the adjacent one (Figure 3). A prestressing rod of the same size and with the same applied force as that used for the diaphragms should be used.

The wood end blocks serve to prevent local web buckling by supplying lateral support to the web of the T-beam at the reactions. This lateral support of the web in effect creates the equivalent of a column on an elastic foundation, thereby substantially increasing the buckling resistance of the web. Representative calculations of the stress at which buckling would occur for a typical T-beam, 762mm (30 in.) deep with a 12.7mm (½ in.) web indicated that the buckling stress would be approximately 6 times higher than the allowable compression stress for the section. Thus, buckling is not possible and need not be considered further.

Local web yielding may be prevented by following ASSHTO 10.33.2 which limits the shear stress in the bearing area to 75 percent of the allowable. In the unlikely event that the applied shear stress exceeds 75 percent of the allowable the tightly fitting wood end blocks will serve to carry part of the bearing load making failure unlikely. Other remedies are available to the designer if needed or desired. First, the thickness of the web could be increased, lowering the applied stress in the web for a given loading. Secondly, a flat plate could be welded to the web to increase the web thickness in the bearing area with the end result of lowering the stress in the web. If used, this steel plate should be shop welded in lieu of field welding, to minimize the possibility of fatigue cracking in the area of the weld.

Construction Methods

Construction of this bridge system may be accomplished using methods similar to those used for the concrete deck-steel girder bridge. With this type of construction the T-beams are placed on the abutments first and the deck is added later. Here diaphragms are used to stabilize the steel beams while the deck is being installed. This process may prove difficult since the diaphragm connections must be left "loose" so that the wood deck and dowels may be placed between the T-beam webs.

The construction of the bridge may be more efficiently carried out by preassembling a single T-beam unit before it is placed on the abutments. In this case a nut and washer may placed on the ends of the dowels to hold the unit together while it is being set on the abutments. After setting preassembled units, prestressing rods may the be placed in the deck and tensioned.

Conclusion

The stress-laminated / steel T-beam bridge system described in this paper utilizes a procedure familiar to most bridge engineers. The design addresses the effective flange width, transverse load distribution, horizontal shear, diaphragms, and local effects in accordance with many existing and proposed AASHTO procedures. Lightweight and efficient steel sections may be incorporated within the stress-laminated timber deck to allow for longer spans, and a cost effective alternative to current designs.

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Rehabilitating Historic Timber Transportation Structures At Burnt Cabins Grist Mill, Burnt Cabins, Pennsylvania

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Abstract

Burnt Cabins Grist Mill is an operating, water-powered grist mill on Little Aughwick Creek in Fulton County, Pennsylvania. The mill and ten acres of the privately owned site are listed on the National Register of Historic Places. Two heritage timber transportation structures were redesigned to combine traditional timber construction techniques with modern materials, treatment and construction methods, including:

1. a timber mill race flume; and
2. a timber weigh station canopy and timber scale bed.

The project was partially funded by the USDA Forest Service Rural Development Program.

Keywords: Burnt Cabins Grist Mill, National Historic Register Site, Fulton County, USDA Forest Service, Southwestern Pennsylvania Heritage Preservation Commission, Mill Race

History

The Village of Burnt Cabins enjoys a colorful transportation history, in which the historic mill site plays a significant part. Burnt Cabins Grist Mill is located in a narrow pass on the western side of Tuscarora Mountain, approximately three miles from the Pennsylvania Turnpike Tuscarora Tunnel. The Tuscarora ridge, and the pass through Burnt Cabins, were major Native American thoroughfares. European migration is first documented through Burnt Cabins by Scottish-Irish immigrants traveling west from Maryland.

Originally called Sidneysville, the name Burnt Cabins comes from timber structures which early settlers had built upon land not yet purchased from the natives. In 1750, the provisional governor ordered that these cabins be burned in an attempt to appease the natives and prevent bloodshed. White settlers soon returned.

Figure 1 - Location Map

Figure 2 - Burnt Cabins Grist Mill, West Elevation, 1995 (Note: scale bed at lower left.)

During the French and Indian War, the British General John Forbes constructed his military road over Tuscarora Mountain and through Burnt Cabins. He erected Fort Littleton (now a turnpike exit) five miles to the west. Tax records indicate that the first grist mill at Burnt Cabins was operating by 1770. Ruins of that first mill still remain to the east of the existing mill, which was built in 1840.

"Vanderbuilt's Folly," the historic South Penn Railroad, was to be aligned directly across the Burnt Cabins Grist Mill site. The railroad grade was completed to the west side of the Little Aughwick, but the stream culvert was never constructed, nor was the grade constructed at the mill site. This railroad right-of-way was purchased and partially used by the Pennsylvania Turnpike Commission, which opened "America's. First Superhighway" in 1940.

The new turnpike alignment bisected the grist mill site, isolating the mill pond from the mill, except for a culverted mill race under the raised highway alignment. The final turnpike alignment miraculously spared the mill and most of the village from an earlier plan to locate an interchange at the mill site. The section of Forbes Road through the mill site was rerouted as Allens Valley Road, a state highway, and cut between the mill race and mountainside to pass under the turnpike.

Recent Development

Today, Burnt Cabins Grist Mill has been identified as a "Heritage Gateway" into Fulton County and the southwestern Pennsylvania region. It is located along Route 522, on a portion of an auto tour route through nine counties in southwestern Pennsylvania called the "Path of Progress" by the federal Southwestern Pennsylvania Heritage Preservation Commission (SPHPC.)

In 1994, SPHPC funded a master plan for the heritage preservation and economic development of Burnt Cabins Grist Mill. Over the twenty five years prior to this master plan study, owners Jack and Sonja Blattenberger had restored the mill and developed a successful gourmet mill products business. However, due to costs of historic reconstruction, many other

Figure 3 - Eastern Elevation showing Existing Mill Race, Head Wall and Trash Screen.

parts of the heritage site required attention beyond the capabilities of a single owner.

Simone and Jaffe Incorporated (SJ) developed the master plan which recommended physical and interpretive improvements throughout the site. The recommendations were based on a financial strategy developed by E.L. Crow Inc. of Lafayette Hill, PA. Priority was given to improvements that would increase tourism and the sale of mill products. Improvements identified as high priority included the reconstruction of two historic transportation structures:

1. *a timber mill race flume; and*
2. *a timber weigh station canopy and scale bed*

In 1995, the Burnt Cabins Grist Mill was awarded a grant from the USDA Forest Service Rural Development program through its community partner, the Burnt Cabins Civic Association, to design and reconstruct the existing timber flume; and the missing timber weigh station canopy and scale bed. Also in 1995, the Grist Mill was awarded a SPHPC grant for site improvements; a historic structures report for an on-site log cabin; and a site-wide interpretive plan. The Owners are instituting these improvements as part of a coordinated program to increase tourism in Fulton County. The working grist mill will be one of the premier heritage destinations for regional visitors.

General Description - Timber Reconstruction

Timber Mill Race Flume - A 40-foot flume which transports water between an earthen mill race and the waterwheel inside the mill was failing and required replacement. The rehabilitation plan included utilizing traditional timber joinery combined with modern techniques and preservation treatment. Reconstruction is scheduled for the Summer of 1996.

Timber Weigh Station Canopy / Scale Bed Originally, the weigh station was used to weigh the grain transport wagons for calculating quantities. The new weigh station will serve as a visitor orientation and interpretive feature. The original refurbished scale will be used to weigh visiting groups en masse, as a practical demonstration of historic technology. The canopy will provide shelter for these activities. Reconstruction of the weigh station is scheduled for September, 1996.

The original 12’ X 22’ weigh station is documented in one early photograph. The new weigh station canopy was designed to match the original scale, roof angles and facade. The new canopy structure will also incorporate mortise and tenon joinery - similar to the timber details in the grist mill. Several simple timber construction techniques were designed to protect the canopy against deterioration of exterior timber, without the use of preservatives.

Figure 4 - Existing Timber Flume, Pedestrian Bridge, and Trash Screen.

The existing timber scale bed was designed to be replaced with treated timber.

1. Timber Mill Race Technical Details

This water transportation structure was constructed between a concrete headwall of the earthen mill race and the stone mill foundation. The timber flume was originally set into an earthen embankment, almost entirely below grade - except for the tops of the posts and headers, and the side of several bays on the downhill side near the waste gate.

Original Construction

The existing timber mill race flume was approximately 40 feet long by 48 inches deep by 48 inches wide on the inside. The flume was framed by a series of rectangle bents which consisted of a sill, two posts, and a header. The bents were traditionally constructed with mortise and tenon joinery and fastened with tapered wooden pins. By Summer of 1995, about half of the original timber bents had been temporarily repaired with pressure treated Yellow Pine.

Flume Bents - The new bents were redesigned to be very similar to the original structure. All joinery was designed to allow drainage. All joinery and pre-drilling was specified to be performed prior to preservative treatment to prevent perforation of the protective treatment shell. Shaved white oak timber pegs were specified as bent fasteners. Tops of the headers were sloped to prevent standing water and discourage their use as "balance beams."

Figure 5 - Existing Timber Flume Bent Detail.

Flume Liner - The original trunk lining was constructed of tongue and groove boards nailed longitudinally to the insides and bottom of the bents. The existing liner boards had been coated many times with tar to seal the wood and joints.

Flume Invert - The flume opening into the mill determined the invert of the flume. The flume invert included a five inch curb which diverts water out through a waste gate to bypass the mill during periods when it is not in operation.

Concrete Headwall - The formed concrete headwall includes flared upstream walls and a short section of concrete flume and apron. Most of the existing headwall was in good condition, except for the uphill wingwall which was cracked and heaving.

Control Gate - Between the parallel walls of the headwall, a slot was formed to house a wooden control gate which is raised and lowered with vertical screw rod to regulate the amount of water which flows through the flume to the waterwheels.

Waste Gate - The waste gate is constructed in the downhill side of the timber flume with openings in the top and bottom of the wall boards in one bay near the concrete headwall. The top opening is an overflow. The bottom opening is controlled with a simple board on a stick which uses water pressure to hold the board against the opening. The gate can be raised and lowered to adjust the flow of wastewater.

Trash Screens - The Owners have installed two wooden vertical-slatted trash screens within the existing flume and headwall. These devices catch debris before it enters the mill and require periodic raking to keep the flume open.

Required Demolition - The entire timber flume was found to require demolition and replacement. One wing of the concrete headwall was cracked and shifted, and required removal and replacement. The existing overflow spillway was a series of deteriorated stone wall terraces stepping down to an overflow pond, which drains into the tail race. Demolition and reconstruction of these stone walls was incorporated into this project. Two wooden foot bridges over the flume were specified to be removed and reconstructed.

Figure 6 - Existing Timber Flume, Waste Gate and Stone Spillway.

Figure 7 - Plan - New Timber Flume and Spillway

Figure 8 - Section through new Timber Flume and Spillway

Structural Design Improvements - A system of structural concrete walls and slabs were designed as the foundation for the new timber flume and spillway. The structure was designed to be hidden below grade and to support an exposed native field stone facade. The design provided for surface and subsurface drainage from the uphill sides of the flume and retaining walls.

New timber bents were designed to sit on a new 4" reinforced concrete slab placed under the flume alignment between the headwall and mill foundation. The bents and structural liner were surrounded by gravel backfill to promote drainage and air movement around the below-grade timbers. A porous geotextile fabric was specified to separate the gravel backfill from the earthen grade.

Two inch thick structural liner boards were designed to be nailed to the insides of the bents in the traditional way, but not to hold water. An impermeable liner was specified to be installed against the inside of the structural flume walls and floor, as one piece, between the mill foundation and the concrete headwall. A "facade" liner was designed as removable sections of longitudinal flooring and siding on sleepers which cover and protect the membrane. The facade wall sections were designed to wedge the floor sections in place without nailing through the membrane below the waterline. In fact, the membrane is employed to prevent leaks, not to protect wooden members. The liner was designed to be sealed to the concrete head wall with battens and to continue inside the mill to line the water distribution box.

Waterproof Flume Liner - The waterproof membrane liner was specified as a reinforced, 45 mil polypropylene polymer geomembrane with a non-woven polyester fiber geotextile bonded onto both sides.

Species Selection and Preservative Treatment Red Oak was selected for the structural liner and the below-grade bent members because it is a local hardwood which has an open cell structure capable of accepting preservative treatment. Untreated White Oak was specified as the bent headers because the members will be above grade and in contact with the public; and the species has a natural resistance to rot.

CCA-treated Yellow Pine was specified as the facade liner and all control structures. Creosote was chosen as the optimum timber preservative for the below-grade members. Tar was specified as an on-site joint sealer.

Hardware - All hardware was specified to be hot-dipped galvanized.

2. Weigh Station Technical Details

The previous weigh station canopy was a roofed, drive-through structure with portals under both gables. The structure was asymmetrical with an enclosed shed section toward the north.

An existing iron scale carriage is mounted on a formed concrete foundation in a below grade well. The carriage is connected by a lever arm to the scale mechanism which is housed in an existing wooden structure above grade.

Original Foundation - The tops of the concrete well walls are flush with the grade and are approximately twelve inches thick. The original canopy posts were mounted directly into concrete piers located just outside these concrete foundation walls. However, the existing concrete walls were chosen as the foundation for the new structure, to eliminate the need to construct new piers.

Canopy Structure Type - The original canopy was used to protect grain wagons. The weigh station canopy was probably "stick" framed, however no evidence exists outside of an original photo. The new canopy was designed to invite visitors, and protect them from the elements during orientation to the mill. It could not exactly replicate the original design.

The new weigh station canopy was designed for observation, as an open-sided timber frame with roof. The design includes mortise and tenon timber joinery, similar to the connections which exist inside the mill and the mill residence.

Horizontal siding was specified on the gable ends to protect roof members and cover knee braces, as siding did on the original structure. Siding exposure matches the mill siding.

New Post Details - The posts were designed to bear on standard metal post pedestals, to prevent the end

Figure 9 - Previous Weigh Station Canopy (date unknown.)

grain from directly contacting masonry. Posts were desigined to be anchored by new threaded anchor rods drilled and grouted into the existing concrete foundation. Posts were designed to be drilled from the bottom to allow the anchor rods to be secured with a typical "bedpost" detail. A mortise opening for the anchor washer and nut was designed into the outside face of the post. All outside faces of posts were designed with a protective one inch vertical sheathing, which extends as a drip edge below the bottom of the posts.

Bent Construction - The structure design employs typical "bents" which are post, girt, rafter, and brace assemblies, connected with mortise and tenon joinery and fastened with timber pegs or "pins." Bents are typically assembled on the ground and lifted to their vertical positions. Connecting girts or "beams" connect the bents. The new canopy design employs four bents - creating three "bays" between the bents.

The bay sizes were determined by the existing concrete foundation which is asymmetrical. A structure of this length could be built with only three bents, however a fourth bent was added to allow the two center bent rafters to extend as an outrigger roof for protection over the housed scale mechanism.

The bents at both gable ends were designed in the original fashion with knee braces inside the portal openings, and sheathed with horizontal siding. Studs between the bent girts and rafters were framed on two foot centers to carry the siding.

The two interior bents were designed with triangular bracing in the roof trusses between the bent girt and rafters. Standard knee braces below the girts were not used on the interior bents to maintain the sense of one undivided space. The outrigger roof does employ knee braces outside the structure, between the posts and rafters.

Roof Overhangs - The roof includes substantial overhangs on the sides and gable ends which are designed to protect the structure and space under the roof from normal, but not occasional driving rains, The side overhang drip lines are two and a half feet from the outside of the posts and we designed to help shade and direct rain and snow away from the posts.

Purlins span longitudinally between rafters, and were designed to mortise into, but sit two inches above the rafter tops to create airspace under the roofing, The

gable end overhangs were designed to use outrigger purlins to extend the drip line of the roof twelve inches beyond the face of the portal.

Interpretive Features - The new weigh station itself will be the subject of interpretation for visitors. It will serve visually as a major structure at the entry to the mill from the west. A dated commemorative plaque will be mounted in each gable. The eastern portal will serve as the visitor entry.

A structural railing was designed between bents to control pedestrian movement and to support interpretive display panels. The display panels will be weatherproofed and mounted at an angle against this railing. Lighting inside the structure will be available by running an underground conduit through a channel milled in the outside of one post.

Self Draining Joinery - All joints which are not covered by siding or roofing are designed to be self draining. This is accomplished by sloping normally horizontal mortise surfaces, and installing "weep" holes. These typical details are employed in the knee braces and railing joints.

Species Selection - White oak was selected as the primary local hardwood species for all canopy structural members because of strength and natural resistance to rot. Siding was designed to be Western Red Cedar.

Timber Treatment - The timbers in the weigh station canopy were designed for use without preservative treatment. The plan includes painting the entire structure the same color as the mill.

Scale Bed - The existing timber scale bed was rotten and cordoned off from the public. Before demolition, sizes of existing members were documented. The new scale bed was designed using the original member sizes.

The timber bed structure was designed to bear on the three longitudinal steel I-beams of the existing scale carriage, each measuring 12 1/2" deep x 5" wide. Timber transverse beams measuring 6" deep x 8" wide spanned the three carriage beams at 16" centers. Longitudinal sleepers measuring 2" deep x 5" or 4" wide were used to span the transverse beams as nailers for the decking. Three inch thick transverse decking was used at random widths between 7" to 12".

Figure 10 - Existing Scale Bed (view to east.)

The original 1 1/2” space was left between the steel edge on the concrete foundation and the outside edge of the scale bed deck. A 4” x 2 1/4” curbing was designed to fasten to both sides of the deck with 4” hex head lag bolts and washers as in the original deck.

All replacement deck members were specified as creosote treated red oak, except for the deck and curb timbers which were specified as CCA-treated Southern Yellow Pine.

Scale housing - The wooden scale housing measures 58” high x 62” long x 12” wide and requires only minor repairs to the southern sill and vertical siding.

Two inch thick decking boards were specified to be replaced between the bed and the scale housing with 2” CCA treated Southern Yellow Pine.

PROJECT SIGNIFICANCE

The significance of the Burnt Cabins Grist Mill project promotes a wider recognition of “non-bridge” heritage timber transportation structures. The Weigh Station and Timber Flume are part of a comprehensive federal/local rehabilitation plan in a highly visible, historic context.

The projects integrate local timber use, historic timber detailing and craftsmanship, and modem timber treatment.

Information about these heritage recreation structures will be part of a comprehensive interpretive plan for the entire historic visitors' site. The traditional canopy frame raising for the weigh station will be video documented for interpretive displays during mill tours.

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Figure 11 - Gable Elevation of New Weigh Station Canopy.

Performance of Red Oak and Red Maple Glued-Laminated Bridges

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Abstract

A three year program to monitor the performance of a red oak longitudinal girder, transverse deck glued-laminated (glulam) highway bridge is presented. The results indicate that the predicted and observed live load beam deflections agree to within 2% when the stiffness of the individual beam laminations and ten percent increase in beam stiffness due to composite action between the deck panel and longitudinal girders are incorporated into the design. Significant reflexive cracking of the asphaltic wearing surface was observed at the interface between each red oak deck panel. This was attributed to the gap provided between each panel during construction, to the placement of the waterproof membrane directly over the creosote treated deck panels, and to improper mating of the deck panels to the beams during installation of the lag bolts. Long term (three year) dead load deflection measurements indicated that after approximately one year, dead load deflections remained nearly constant for the interior beams. Elevations of the lower surface of the two exterior beams fluctuated considerably and varied seasonally. There was no evidence of delamination of the girders or deck panels after four years. However, there was some evidence of delamination of the curbs and to tops of rail posts. Preliminary observations of a red maple glulam bridge, the design of which addressed the three shortcomings of the red oak bridge, suggest no delamination of the glulam components and no reflexive cracking of the wearing surface after nine months of service. Live load testing of the red maple bridge is scheduled for the summer of 1996.

Keywords: Red oak, red maple, glulam, timber bridge

Introduction

A demonstration bridge project has been underway in Pennsylvania for the past several years. The goals of this effort are to design, construct, and monitor hardwood timber highway bridges throughout the state, thus demonstrating the suitability of hardwoods for structural components in highway bridges. Two of the demonstration bridges are hardwood glued-laminated (glulam) bridges, one of northern red oak and one of red maple. An additional eleven hardwood glulam bridges are currently being designed for forestry roads in Pennsylvania by the Department of Conservation and Natural Resources. The objective of the remainder of this paper is to summarize the design of and field performance of the northern red oak and the red maple hardwood glulam demonstration highway bridges. The red oak bridge opened for traffic in November 1991; the red maple bridge opened to traffic in August, 1995.

The projects were cooperative efforts of several organizations under the leadership of a Penn State University Research Team from the Department of Agricultural and Biological Engineering and the Wood Products Program of the School of Forest Resources. The Penn State Research Team was responsible for all quality control matters and specifications related to wood procurement processing, grading, and fabrication. Gwin Dobson and Forman, Inc. of State College, Pennsylvania, designed the substructures and

superstructures and supervised construction; Unadilla Laminated Products, Inc. of Sidney, New York, fabricated the glued laminated structural members and provided fastener hardware for the red oak project; Rigidply Rafters, Inc. of Richland, Pennsylvania fabricated the red maple glulam members and provided fastener hardware and details for the red maple project. Koppers, Inc. of Muncy, Pennsylvania, treated the glued laminated members for both projects. Kamtro Construction of Osceola Mills, Pennsylvania, constructed the red oak bridge; Redrock Construction of Mifflintown, Pennsylvania constructed the red maple bridge. The northern red oak bridge owner is Ferguson Township in Centre County, Pennsylvania; the owner of the red maple bridge is East Pennsboro Township in Cumberland County, Pennsylvania.

Bridge Descriptions

Red Oak Bridge

The northern red oak glued laminated girder and glued laminated deck bridge was designed to replace a 44-year old reinforced concrete tee beam bridge with a 107 kN (12 ton) rating on Township Road T-330 in Ferguson Township in Centre County, Pennsylvania. The bridge superstructure was erected onto the existing stone abutments. The bridge skew, at 45 degrees, was severe. The bridge was designed and constructed prior to publication of BLC-560 Series, Standards for Hardwood Glulam Timber Bridge Design (PennDOT, 1994) and prior to the revision of AITC 119 (AITC, 1996).

The design specifications for the bridge were:

- Loads - HS25 or ML80 live load
- Deflections - Live load deflection less than span/500
- Materials - All superstructure, railings, and parapets to be glued laminated northern red oak
- Clear span between centerline of abutments -10.69 m (35 ft. 0 1/2 in.)
- Overall deck width - 8.54 m (28 ft.)

All structural components were designed in accordance with the 1986 ed. of the National Design Specification for Wood Construction (NFPA, 1986), the 1988 ed. of the Supplement to the National Design Specification (NFPA, 1988), the AASHTO Standard Specifications for Highway Bridges, (AASHTO 1989), and PennDOT Design Manual Part 4 (PennDOT 1990). All the girders were specified as Combination A lay-ups (Fig. la) with the following unadjusted structural properties: F_{bx} = 15.4 MPa (2240 psi); F_v = 1.5 Mpa (230 psi); E = 11.0 GPa (1.6 x 10^6 psi). The girders were braced laterally by endwall diaphragms, midspan diaphragms and by the glulam deck which was fastened to the girders every 0.30 m (12 in.) on center. The glued laminated deck panels were specified as Combination A (without special outer laminations) northern red oak with F_b = 15.4 Mpa (2240 psi), F_v = 1.5 psi (230 psi), and E = 11.0 GPa (1.6 x 10^6 psi).

The bridge superstructure has nine 203 mm by 743 mm (8 in. by 29-1/4 in.) girders spaced 965 mm (38 in.) on center (Fig. 2). All girders were fabricated with 38 mm (1.5 in.) laminations. The 152 mm (6 in.) thick deck consists of 788 mm (31 in.) and 1220 mm (48 in.) wide by 8.54 m (28 ft.) long panels. All panels were spaced approximately 6.5 mm (1/4 in.) apart to accommodate anticipated in-service moisture expansion because the panels were fabricated at 12±2% moisture content and are expected to equilibrate over the stream at about 19% moisture content. The 152 mm (6 in.) deck was designed as a non-interconnected deck (AASHTO, 1989; Ritter, 1990). However, one-half of the bridge was constructed with 32 mm (1 1/4 in.) diameter dowels to observe performance differences, if any, between the asphalt paving over the interconnected panels and the non-interconnected panels. The endwall diaphragms were 152 mm (6 in.) wide by 743 mm (29 1/4 in.) deep and extended the fill 12.08 m (39.6 ft.) skew length. Midspan diaphragms, 150 by 743 mm (3 in. by 29 1/4 in.), were installed perpendicular to the span between each pair of girders for lateral stability. The girders were attached to the abutment with 19 mm (3/4 in.) anchor bolts (all bridge hardware was double dipped galvanized). The bearing design allowed vertical adjustment for proper leveling of the top surfaces of the nine beams. The deck panels were fastened to the girders with 19 mm by 229 mm (3/4 in x 9 in.) galvanized lag bolts. The heads were recessed into the deck. The diaphragms were connected to the girders with three 19 mm by 229 mm (3/4 in. x 9 in.) galvanized lag bolts at each girder. (This detail has been changed in the BLC-560 Standard Plans (PennDOT, 1994) to 2-19 mm (3/4 in,) diameter threaded rods which extend through the diaphragm and two adjacent beams.)

Oakum was installed between deck panels. Before paving, a waterproof geotextile membrane was installed over the deck.

The railings and parapets design consists of 254 mm by 305 mm (10 in. x 12 in.) glued laminated posts spaced 1.83 m (6 ft.) on center, two 152 mm by 203 mm (6 in. x 8 in.) glued laminated rails, and 254 mm x 305 mm (10 in. x 12 in.) glued laminated curbs. The rail system is fastened with galvanized bolts and drift pins. All glulam members were treated with creosote to a retention level of 192.2 kg/m^3 (12 pcf) with a minimum depth of penetration of 6 mm (0.25).

Red Maple Bridge

The red maple glulam longitudinal girder and transverse glulam deck bridge was designed to replace

Figure 1--Glulam girder layups for (a) Combination A northern red oak and (b) red maple bridges.

Figure 2--Sketch of superstructure for the northern red oak glulam bridge.

an old single lane concrete bridge with an 89 kN (10 ton) rating over Possum Hollow Creek in East Pennsboro Township in Cumberland County, Pennsylvania. The entire project included realignment of the roadway and construction of new concrete abutments. The bridge skew was minimal (approximately 15 degrees). The design specifications for the bridge were:

- Loads-HS25 or ML80
- Deflections - Live load deflection less than span/500
- Materials - All superstructure, railings and parapets to be glued-laminated red maple
- Clear span between centerline of abutments-8.20 m (26 ft. 10 3/4 in.)
- Overall deck width - 10.88 m (35 ft. 8 in.)
- Curb-to curb width - 9.76 m (32 ft.)

All structural components were designed in accordance with the BLC-560 Series, Standards for Hardwood Glulam Timber Bridge Design (PennDOT, 1994) and PennDOT Design Manual Part 4 (PennDOT, 1990) All girders were specified as 24f-1.8E combination layups (Figure lb) with the following unadjusted structural properties: F_{bx} = 16.5 MPa (2400 psi); F_v = 1.3 MPa (205 psi); and E = 12.4 GPa (1.8 x 10^6 psi). The girders were braced laterally by endwall diaphragms, midspan diaphragms and by the glulam deck which was lag bolt connected to the girders every 0.30 m (12 in.) on center. The glulam deck panels were specified with uniform grade No. 2 visually stress graded red maple laminations with F_{bx} = 12.4 MPa (1800 psi), F_v = 1.3 MPa (205 psi) and E = 12.4 GPa (1.8 x 10^6 psi).

The bridge superstructure has 15-130 mm by 838 mm (5.125 in. by 33 in.) girders spaced 660 mm (26 in.) on center (Figure 3). All girders were fabricated with 38 mm (1.5 in.) laminations. The 108 mm (4.25 in) thick deck panels were 1.22 m (48 in.) wide by 10.88 m (35.67 ft.) long. The panels were abutted with no spacing between adjacent panels. The deck panels were interconnected with 32 mm (1 1/4 in.) diameter dowels spaced 230 mm (9 in.) on center. The endwalls were 80 mm (3.125 in.) wide by 838 mm (33 in.) deep and extended the full width of the bridge. Midspan glulam diaphragms, 80 mm (3.125 in.) thick by 530 mm (20.875 in.) wide by 750 mm (750 in.) deep were installed perpendicular to the span between each pair of girders. The girders were attached to the abutment with 19 mm (3/4 in.) diameter anchor bolts (All bridge hardware was double dipped galvanized.). The deck panels were fastened to the girders with 19 mm by 229 mm (0.75 in. by 9 in.) galvanized lag bolts. The bolt heads were recessed into the deck. Endwall diaphragms were lag bolt connected to the girders; the midspan diaphragms were connected to the girders with two 19 mm (3/4 in.) diameter threaded rods which extend through the diaphragm and the two adjacent beams. A waterproof membrane was installed between the asphalt base and wearing courses. The railings and parapets are similar to those for the red oak bridge with the exception of the preservative treatment specification. The red maple glulam bridge railings were treated with a CCA/oil emulsion system (Blankenhorn, et al., 1996) at the Koppers Industries plant located in Montgomery, Alabama. This oil emulsion/waterborne system was developed by Hickson Corp. and Koppers Industries, Inc. and it consists of injecting an oil and wax combination into the outer 25.4 mm (1.0 in.) following CCA treatment and drying of the treated wood. The target retention of the CCA was 9.6 kg/m^3 (0.6 pcf). Assay retention analysis indicated that actual retention of the CCA was 12.8 kg/m^3 (0.8 pcf) and the oil/wax retention was 19.2 kg/m^3 (1.2 pcf).

The creosote treatment of the red maple glulam bridge members used a treatment cycle that was similar to the red oak treatment cycle (Blankenhorn et al., 1996). The target retention for the red maple glulam was 192.2 kg/m^3 (12 pcf). Actual assay retention was 169.1 kg/m^3 (16.8 pcf). The creosote penetrated the red maple glulam to a depth of 63.5 mm (2.5 in.) or more with some areas being in excess of 76.2 mm (3 in.).

Red Oak Bridge Performance

Description of Monitoring

The red oak bridge was monitored to evaluate its structural performance, the dimensional stability of the deck panels, the durability of the glulam components, and the performance of the asphalt wearing surface. Live load tests were performed in August, 1991. Deflection profiles of the nine red oak girders were measured when loaded with nominal 334 kN (75 kip) triaxle trucks. The 334 kN (75 kip) live load was first applied to one lane, then to the other lane and finally simultaneously to both lanes. The lane loads were applied to produce maximum deflection of the girders (Figures 4). Dead load creep deflections of the centerline of the girders were measured monthly for three years after the live load test. Also, the dimensional changes of the deck panel widths were measured prior to creosote treatment, immediately after creosote treatment, and monthly for three years. Glulam components were periodically surveyed to document any delamination of glulam bridge components. Finally, the reflexive cracking of the asphalt wearing surface was monitored.

Structural Evaluation

The centerline live load deflection of red oak girders 1 through 9 are plotted in Figure 5. Live load deflection profiles for beams 5 and 8 are plotted in Figures 6 and

Figure 3--Sketch of the superstructure for the red maple glulam bridge.

Figure 4--ML-8O vehicle location for maximum deflection of the red oak glulam bridge.

Figure 5--Centerline live load deflections for the red oak bridge.

Figure 6--Live load deflection profile for beam 5 of the red oak bridge.

Figure 7. Live load deflection profile for beam 8 of the red oak bridge.

Figure 8--Creep deflection of girders 1 to 9 of the red oak glulam bridge.

7. Dead load creep deflections for beams 1 through 9 are plotted in Figure 8.

The centerline live load deflection for load cases I and II are nearly symmetric with respective lane deflections agreeing to within 1 mm for all beams. The measured centerline live load deflection for load case III was nearly symmetric about the bridge centerline. The deflections of beams 4 and 6 differed by 1 mm (Approximately 8% of total deflection) and beams 3 and 7 differed by 1 mm (Approximately 9% of total deflection). The maximum measured centerline live load deflection for load case III (Beam 5) was 13 mm (0.52 in.). The sum of the respective beam deflections for load cases I and II agreed with the measured deflections for load case III to within 1 to 2 mm. (0.08 to 0.16 in.) The deflection profiles of beams 5 and 8 (Figures 6 and 7) show the maximum live load deflection occurring at midspan for each of load cases I, II and III. The symmetry of the live load deflections and the agreement of the superimposed case I and II deflections with load case III deflections are clear indicators of satisfactory live load distribution across the bridge by the glulam deck, the lag bolt connections and the glulam diaphragms.

The predicted centerline live load deflection of beam 5, assuming no composite behavior between the deck and girder, girder E-value of 11.0 GPA (1.6 x 10^6 psi), and an HS25 or ML80 load, was 22 mm (0.85 in.). The observed maximum live load deflection for beam 5 was 14 mm (0.55 in.). Lower actual vs. predicted deflection is probably due to: 1) The conservative design value of E [Shaffer, et al. (1991) reported E-values of 13.1 GPa (1.90 x 10^6 psi) for northern red oak beams] used in the calculations; 2) Neglect of composite action between the deck and girders, and 3) The average E-value determined by static loading of each board used in the bridge girders being 15.5 GPa (2.2 x 10^6 psi). Predicted live load deflection using an

E-value of 15.5 GPa (2.2 x 10^6 psi) equals 16 mm (0.62 in.). Witmer (1996) has reported that composite action between lag bolt connected transverse hardwood glulam decks and beams increase girder stiffness by 8 to 12%. Applying this increase to the northern red oak girder stiffness yields a predicted live load deflection of 14.3 mm (0.56 in.) which compares favorably with the observed deflection.

Dead load deflections for the nine beams for 36 months duration are plotted in Figure 8. The long term dead load deflections vary considerably between beams. The bottom face of some beams experienced downward movement, whereas some experienced upward movement. Maximum upward movement of 10 mm (0.39 in.) was measured for beam 3; maximum downward movement of 8.5 mm (0.33 in.) was measured for beam 8. Dead load deflections of most of the beams remained relatively constant (±2 mm) for approximately 6 to 9 months, then experienced a shift of several mm. The deflection of the interior beams then stabilized. Over the last 24 months of the monitoring, the dead load deflections of the interior (2 through 8) beams did not increase nor decrease by more than 1 mm or 2 mm (0.08 in. or 0.16 in.). The trend suggests that one cycle of annual weather change is necessary to condition, or to seat, the superstructure. The dead load deflection of the exterior beams (1 and 9) fluctuated by nearly 7 mm (0.27 in.) over the last 24 months of monitoring. These beams were exposed to radiation and convective heat loads, whereas the interior beams were not. There are no apparent trends between beam location and direction of movement. One possible explanation of the upward movement of four beams is differential moisture content of the beam from top to bottom. That is, the lower portion of the cross section, being more exposed to the drying action of ambient air, was at a lower moisture content than the upper portion of the beam. Such a situation would have the effect of inducing a slight reverse curvature of the beam.

Dimensional Stability

The design of the red oak and red maple glued-laminated timber bridge included an intentional panel separation between the deck panels. The size of the separation was based on the average transverse swelling (radial plus tangential divided by two) and the width of the deck panels. The separation would allow the deck panels to expand as they increased in moisture content from an average of 12% during fabrication to approximately 20%. after installation of the bridge. The design separation for the red oak glued-laminated timber bridge was 12.7 mm (0.5 inch) and for the red maple bridge was 9.5 mm (0.38 inch). During creosote treatment the deck panels adsorbed moisture and expanded.

Table 1 lists the fabricated and creosote-treated red oak glued-laminated deck panel widths. The bridge design allowed for a maximum panel separation gap of 12.7 mm (0.5 inch) between panels for a total deck expansion of 203.2 mm (8.0 inches). The total width of the red oak glued-laminated deck panels after creosote treatment increased by 219.7 mm (8.65 inches). However, the deck was installed with a gap between each panel even though the deck had expanded past the design amount.

The size of the separation after installation between each panel is listed in Table 2. The separation between each panel was monitored from October 1991 to October 1994. The final size in October 1994 of the separation is also listed in Table 2. The total separation between the panels decreased by 27.70 mm (1.17 inches) after being in service for three years. The cumulative expansion of all the panels after creosote treatment and in service for three years was 247.40 mm (9.74 inches) compared to the cumulative design panel separation of 203.2 mm (8.0 inches).

After the red oak glued-laminated deck panels had been in place for three years, a seasonal variation in panel width began to appear. The panels were at their maximum width in July/August and their minimum width from October to March. The moisture adsorption and desorption resulted in a total annual change in the red oak deck panel width of 14.97 mm (0.59 inches).

Delamination

No delamination of the deck panels nor the girders has occurred since the bridge was installed in October, 1991. Some delamination (e.g., glueline checking) of the curbs has occurred at locations near the roadway surface. The delamination can probably be minimized by adhering to the hardwood lamination procedures specified by Manbeck et al. (1996). These specifications were not fully identified at the time of the red oak bridge construction in 1991. Also, some delamination of the top ends of the guiderail posts has occurred. The guiderail posts were nominal 254 x 305 mm (10 by 12 in.) red oak glulam members. Since they were so large and since nominal 254 mm (10 in.) laminations were not available, the posts were cross laminated to form a square lamination pattern. The top of the posts were sealed with an asphaltic compound. However, this treatment did not adequately protect the post ends.

Wearing Surface

Reflexive cracks began to appear approximately six months after the bridge was completed. After one year, there were 14 cracks in the deck asphalt. All cracks were directly over a deck panel joint. The cracks

ranged in length from 406 mm (16 in.) to 6.07 m (20 ft.). Six cracks were under 2.54 m (100 in.) long, four were between 2.54 and 5.10 m (100 and 200 in.) long, and four were between 5.10 m and 6.07 m (20 ft.) long. Crack widths ranged from less than 1 to 3 mm (0.04 to 0.12 in.) wide. Reflexive cracks have grown over time. In May, 1996, 3 mm (0.125 in) to 6 mm (0.25 in.) wide reflexive cracks extended the full width of the bridge wherever two deck panels were connected. Also a narrow longitudinal crack has developed along the roadway centerline of the bridge over approximately 50% of the bridge length. The reflexive cracks have been sealed once, but have reopened. The presence of icicles and salt stains on the underside of the deck indicate that, in addition to reflexive cracking of the asphalt, the waterproof membrane has failed. The waterproof membrane was installed immediately on top of the creosoted glulam deck panels. The probable causes for the amount of reflexive cracking are: (1) Inadequate mating of the deck panels to the beams before installation of the lag bolts; (2) Installation of the waterproof membrane directly over the deck panels; and (3) The 6 to 8 mm (0.25 to 0.38 in.) gap between the adjacent glulam deck panels. All three factors are easily corrected in the design or construction phases of a project.

Red Maple Bridge Performance

Table 3 lists the non-treated red maple deck panel width and the width (parallel to the skew of the deck) after creosote treatment followed by a low temperature/vacuum steam cycle described by Blankenhorn et al (1996). The bridge design allowed for a maximum gap of 9.5 mm (0.38 in.) for a total deck expansion of 57.2 mm (2.25 in.). The total width of the deck along the skew after creosote treatment increased by 51.2 mm (2.01 in.) from the non-treated width. Consequently, the edges of the deck panels were butted together during installation.

As of May, 1996 there were no reflexive cracks in the asphalt wearing surface nor any signs of delamination of any of the glulam components of the bridge superstructure. Live load testing of the bridge has been delayed and will be conducted in June, 1996. Results of the load testing will be included in the oral presentation.

Ongoing Work

The red maple bridge will be live load tested in June, 1996. At that time differential displacements between adjacent deck panels will also be measured. Also, core samples of the asphaltic wearing surface will be taken by researchers from Virginia Polytechnic Institute to determine the condition of the waterproof membrane installed between asphalt base and wearing courses.

Summary

The results of a monitoring program for a red oak glulam highway bridge have demonstrated that it is

Table 1.--Dimensional Changes in Red Oak Glued-Laminated Timber Bridge Deck Panels Before and After Creosote Treatment.

Panel	Width Prior to Creosote Treatment	Width After Creosote Treatment
1	111.76 cm (44.00 inches)	112.09 cm (44.13 inches)
2	121.92 cm (48.00 inches)	123.82 cm (48.75 inches)
3	121.92 cm (48.00 inches)	122.89 cm (48.38 inches)
4	121.92 cm (48.00 inches)	123.19 cm (48.50 inches)
5	121.92 cm (48.00 inches)	123.19 cm (48.50 inches)
6	121.92 cm (48.00 inches)	123.82 cm (48.75 inches)
7	121.92 cm (48.00 inches)	123.82 cm (48.75 inches)
8	78.74 cm (31.00 inches)	80.01 cm (31.50 inches)
9	78.74 cm (31.00 inches)	80.01 cm (31.50 inches)
10	121.92 cm (48.00 inches)	123.52 cm (48.63 inches)
11	121.92 cm (48.00 inches)	122.89 cm (48.38 inches)
12	121.92 cm (48.00 inches)	123.19 cm (48.50 inches)
13	121.92 cm (48.00 inches)	123.19 cm (48.50 inches)
14	121.92 cm (48.00 inches)	123.19 cm (48.50 inches)
15	121.92 cm (48.00 inches)	123.19 cm (48.50 inches)
16	121.92 cm (48.00 inches)	123.52 cm (48.63 inches)
17	111.76 cm (44.00 inches)	112.40 cm (44.25 inches)
TOTAL WIDTH	1965.96 cm (774.00 inches)	1987.93 cm (782.65 inches)

Table 2--Dimensional Stability of the Panel Separations Between Red Oak Glued-Laminated Timber Bridge Deck Panels.

Panels[1]	Initial Average Panel Separation October 1991	Final Average Panel Separation October 1994	Average Panel Separation Seasonal Variation	
			July 1994	October 1993
1-2	7.87 mm (0.31 inches)	6.60 mm (0.26 inches)	7.62 mm (0.30 inches)	6.35 mm (0.25 inches)
2-3	12.30 mm (0.50 inches)	11.94 mm (0.47 inches)	11.94 mm (0.47 inches)	10.41 mm (0.41 inches)
3-4	10.16 mm (0.40 inches)	8.13 mm (0.32 inches)	9.14 mm (0.36 inches)	8.13 mm (0.32 inches)
4-5	10.67 mm (0.42 inches)	8.64 mm (0.34 inches)	10.16 mm (0.40 inches)	8.64 mm (0.34 inches)
5-6	13.46 mm (0.53 inches)	9.40 mm (0.37 inches)	10.92 mm (0.43 inches)	9.40 mm (0.37 inches)
6-7	11.94 mm (0.47 inches)	9.65 mm (0.38 inches)	10.67 mm (0.42 inches)	9.65 mm (0.38 inches)
7-8	7.62 mm (0.30 inches)	5.33 mm (0.21 inches)	6.35 mm (0.25 inches)	5.33 mm (0.21 inches)
8-9	9.4 mm (0.37 inches)	8.89 mm (0.35 inches)	8.64 mm (0.34 inches)	8.89 mm (0.35 inches)
9-10	7.11 mm (0.28 inches)	5.08 mm (0.20 inches)	5.59 mm (0.22 inches)	4.83 mm (0.19 inches)
10-11	10.16 mm (0.40 inches)	7.37 mm (0.29 inches)	9.14 mm (0.36 inches)	7.37 mm (0.29 inches)
11-12	5.84 mm (0.23 inches)	3.56 mm (0.14 inches)	4.57 mm (0.18 inches)	3.56 mm (0.14 inches)
12-13	8.38 mm (0.33 inches)	4.83 mm (0.19 inches)	5.84 mm (0.23 inches)	4.83 mm (0.19 inches)
13-14	5.08 mm (0.20 inches)	3.81 mm (0.15 inches)	4.32 mm (0.17 inches)	3.81 mm (0.15 inches)
14-15	4.57 mm (0.18 inches)	4.06 mm (0.16 inches)	4.57 mm (0.18 inches)	4.06 mm (0.16 inches)
15-16	5.33 mm (0.21 inches)	3.30 mm (0.13 inches)	4.06 mm (0.16 inches)	3.30 mm (0.13 inches)
TOTAL SEPARATION	130.29 mm (5.13 inches)	100.59 mm (3.96 inches)	113.53 mm (4.47 inches)	98.56 mm (3.88 inches)

[1] **Panels 16 and 17 were butted against each other during installation.**

structurally satisfactory and that the observed and predicted live load deflections are in close agreement. Significant reflexive cracking of the asphaltic deck occurred in the red oak bridge. However, these cracks can probably be minimized by modifying design specifications related to intimate abutting of adjacent deck panels, location of waterproof membranes, and by properly mating deck panels to beams prior to installation of lag bolt connectors. Dead load deflection measurements over a three year period indicate that, after approximately one year, there is little additional(less than 2 mm) dead load deflection in the bridge. The red maple monitoring is incomplete. Live load testing is scheduled for the summer of 1996. There is no evidence of reflexive cracking of the deck or delamination of superstructure components. This suggests that elimination of the gaps between panels, mating decks to beams before installing lag bolts and relocation of the waterproof membrane are effective in minimizing reflexive cracking of the deck wearing surface.

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Table 3--Dimensional Changes in Red Maple Glue-Laminated Timber Bridge Deck Panels Before and After Creosote Treatment.

Panel	Width Prior to Creosote Treatment	Width After Creosote Treatment
A	118.75 cm (46.75 inches)	119.38 cm (47.00 inches)
B	118.90 cm (46.81 inches)	119.54 cm (47.06 inches)
C	118.90 cm (46.81 inches)	119.71 cm (47.13 inches)
D	118.75 cm (46.75 inches)	119.71 cm (47.13 inches)
E	118.90 cm (46.81 inches)	119.54 cm (47.06 inches)
F	118.90 cm (46.81 inches)	119.71 cm (47.13 inches)
G	118.75 cm (46.75 inches)	119.38 cm (47.00 inches)
TOTAL WIDTH	831.85 cm (327.49 inches)	836.97 cm (329.51 inches)

Evaluation of Stress-Laminated T-Beam Bridges Constructed of Laminated Veneer Lumber

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Abstract
Growing interest in wood bridges during the past decade has led to the use of new wood products and innovative designs for bridges. One new wood bridge system that is becoming increasingly popular involves solid, laminated veneer lumber T-beams that are stress laminated together. As part of the development of these bridges, an evaluation program to monitor field performance of six bridges was implemented. This paper describes results of field performance related to bar force, moisture content, and condition assessment, with presentation of load test and analytical evaluations for two of the six bridges.

Keywords: Bridges, Wood, Laminated veneer lumber, Field performance, Stress laminated.

Introduction
The objective of the Timber Bridge Initiative (TBI), passed by the U.S. Congress in 1988, was to further develop and extend the use of wood as a bridge material (USDA 1995). As part of this objective, emphasis has been placed on the use of new engineered wood products and innovative bridge designs. One engineered wood product that has recently been adapted for bridge applications is laminated veneer lumber (LVL). LVL, which is a subcategory of new wood products called structural composite lumber, is made from sheets of rotary-peeled wood veneer that are glued together with waterproof adhesive to form structural members (Figure 1). The thickness of the veneer laminations is commonly 2.5 to 6.4 mm (0.10 to 0.25 in.). In contrast to plywood, LVL laminations are oriented with the grain direction parallel, rather than having some laminations at right angles.

Several characteristics of LVL make it desirable for structural applications. Because it is a manufactured product, LVL can be produced in a variety of sizes and shapes. The laminating process disperses the natural strength-reducing characteristics of wood, which

Figure 1 – Typical LVL beams.

reduces product variability and provides improved design strength and stiffness compared to sawn lumber. Because of this, design values of up to 20.2 MPa (2,900 lb/in^2) in bending and 13,800 MPa (2,000,000 lb/in^2) for modulus of elasticity (MOE) are possible. LVL also provides improved treatability with wood preservatives because of small lathe checks in the veneers that facilitate preservative penetration. The result is generally full preservative penetration of structural components (Tschernitz et al. 1974). Most LVL is produced from either Douglas-fir or Southern Pine veneers, but hemlock, yellow-poplar, oak, and spruce have also been used.

LVL is not a new material. It originated in the 1940s when research was being conducted for high strength aircraft structures (Forest Products Laboratory 1987) and has been produced commercially for more than 25 years. The primary use for LVL has historically been in residential and commercial building applications, such as beams and headers, chords of trusses, and the flange component of prefabricated wood I joists. The first known LVL bridge constructed in the United States was built in 1977 of "press-lam," a type of LVL developed at the USDA Forest Service, Forest Products Laboratory (FPL). This prototype structure, jointly sponsored by the Forest Service and the Virginia State Highway Department, consists of a 79-mm- (3.13-in.-) thick LVL deck supported on 114- by 508-mm (4.5- by 20-in.) LVL beams spaced 762 mm (30 in.) on-center (Youngquist et al. 1979). One of the next known uses of LVL in bridges was in 1988 when a 22.25-m (73-ft) span experimental stress-laminated T-beam bridge was constructed in West Virginia using LVL for the web components. For this structure, flanges constructed of nominal 38- by 229-mm (1.5- by 9-in.) oak lumber laminations were stress laminated between 152- by 1,145-mm (6- by 45-in.) LVL webs spaced 915 mm (36 in.) on-center (Dickson and GangaRao 1989).

The first effort to develop and market a bridge system constructed entirely of LVL was initiated by Trus Joist MacMillan in the late 1980s. These bridges consisted of a series of fully laminated T-beams that were stress laminated together through the flange to form the bridge width (Figure 2). By 1993, approximately 20 bridges of this type were constructed in the western United States, with clear spans of 7.3 to 15.2 m (24 to 50 ft) and widths of 3.0 to 11.0 m (10 to 36 ft). One obstacle to acceptance of the bridges was the lack of LVL design values in the *Standard Specifications for Highway Bridges* published by the American Association of State Highway and Transportation Officials (AASHTO). This was subsequently resolved, and design values for LVL bridges were included in the 1995 interim specifications (AASHTO 1992). As a result the system is becoming more popular as an alternative for new construction and bridge replacement. The bridges are also currently being constructed of another type of structural composite lumber known as parallel strand lumber (PSL) (Meyer 1995).

During the development process for stress-laminated LVL T-beam bridges, a field evaluation program was considered necessary to fully evaluate bridge performance and optimize design methodology. As a result, FPL was contacted to assist in field monitoring, load testing, and analytical evaluations, and a bridge monitoring program was initiated in 1991 for six bridges. The objective of the monitoring was to evaluate performance characteristics and improve design procedures for LVL T-beam bridges through field evaluation and analytical modeling. The scope of this

Figure 2 – Typical stress-laminated LVL T-beam showing (top) the end section of a solid LVL T-beam and (bottom) a completed bridge.

paper is limited to results of field performance related to bar force, moisture content, and condition assessment, with presentation of load test and analytical evaluations for two bridges.

Description of Bridges

Of the numerous LVL T-beam bridges constructed, six bridges located in the northwestern U.S. were selected as representative examples for field evaluation purposes. As shown in Figure 3, the bridges ranged in length from 7.92 to 13.41 m (26 to 44 ft) with widths of 4.88 to 11.43 m (16 to 37.5 ft). The size of the individual LVL T-beams varied for each bridge depending on specific design requirements and ranged from 610 to 635 mm (24 to 25 in.) wide and 356 to 711 mm (14 to 28 in.) deep, with a flange thickness of 152 mm (6 in.) and a web width of 203 mm (8 in.). Four bridges included LVL box beams along the edges of the bridge to improve dimensional stability.

All bridges were stress laminated with ASTM A 722 high strength steel bars spaced 762 mm (30 in.) on-center (ASTM 1988). The design bar force of 133 kN (30,000 lb) provided an interlaminar compression of approximately 1.15 MPa (167 lb/in^2) between the flanges. The LVL for all bridges was pressure treated with pentachlorophenol in heavy oil in accordance with American Wood Preservers' Association (AWPA) Standard C 14 (AWPA 1990). With the exception of the Kenally Creek bridge, which was provided with lumber running planks, all bridges were constructed with an asphalt wearing surface. A detailed description of the South Canal and Mill Creek bridges (discussed later in this paper) follows.

South Canal Bridge

The South Canal Bridge is located in Owyhee County, Idaho, and is a single-lane bridge with a span of 7.71 m (25.3 ft) center-to-center of bearings and an out-to-out width of 4.88 m (16 ft) (Figure 4). The bridge consists of 8 LVL T-beams, 610 mm (24 in.) wide and 356 mm (14 in.) deep.

Mill Creek Bridge

The Mill Creek Bridge is located in Stevens County, Washington. It is a two-lane bridge with a span of 8.93 m (29.3 ft) center-to-center of bearings and an out-to-out width of 7.32 m (24 ft) (Figure 5). The T-beams are 610 mm (24 in.) wide and 406 mm (16 in.) deep. The exterior box beams are 610 mm (24 in.) wide and 406 mm (16 in.) deep, with a top flange thickness of 152 mm (6 in.) and a bottom flange thickness and web width of 114 mm (4.5 in.).

Evaluation Methodology

Evaluation of the LVL T-beam bridges utilized procedures and equipment previously developed by FPL (Ritter et al. 1991). A description of the methodology for moisture content and bar force measurement, load test behavior, analytical evaluation, and condition assessment follows.

Moisture Content

Moisture content measurements were taken on the underside of the deck at web and flange locations using an electrical-resistance moisture meter with 76-mm (3-in.) probe pins in accordance with ASTM D4444-84 procedures (ASTM 1990). Measurements were obtained by driving the pins to depths of 25 to 51 mm (1 to 2 in.), recording the moisture content value, and adjusting the values for LVL temperature and species based on factors obtained from Trus Joist MacMillan. Moisture content measurements of the Petty Creek bridge were taken approximately monthly for 2.5 years. Moisture content measurements for the other bridges were taken prior to installation and at the time of load testing.

Bar Force

Bar force for the Petty Creek bridge was measured on a monthly basis using two load cells developed by FPL. Load cell strain measurements were obtained with a portable strain indicator and converted to units of bar tensile force by applying a laboratory conversion factor to the strain indicator reading. At the conclusion of the monitoring period, the load cells were removed, checked for zero balance shift, and recalibrated to determine time-related changes in the initial load cell calibration. Bar force for the other bridges was measured with a hydraulic jack at the time of construction and load testing.

Load Test Behavior

Static load testing consisted of positioning loaded test vehicles on the bridge deck and measuring the resulting deflections at a series of transverse locations at midspan. Measurement of bridge deflections were taken prior to testing (unloaded), for each load position (loaded), and at the conclusion of testing (unloaded). Deflection measurement from an unloaded to loaded condition was obtained by placing a calibrated rule at the bottom center of each web and reading values with a surveyor's level to the nearest 0.8 mm (0.03 in.).

The load test vehicles at both bridges were fully loaded three-axle dump trucks (Figure 6). At the South Canal bridge, a single truck with a gross vehicle weight (GVW) of 304 kN (68,500 lb) was used.

Bridge name	Length (L)	Width (W)	Depth	Skew (deg)	Edge box beams
Mill Creek	9.1 m (29.8 ft)	7.3 m (24 ft)	406 mm (16 in.)	0	Yes
Petty Creek	11.6 m (38.2 ft)	8.5 m (28 ft)	508 mm (20 in.)	32	Yes
Kenally Creek	9.9 m (32.4 ft)	5.2 m (17 ft)	457 mm (18 in.)	0	No
Franklin Road	13.4 m (44 ft)	11.0 m (36 ft)	711 mm (28 in.)	0	Yes
Wardwell	8.5 m (28 ft)	11.4 m (37.5 ft)	406 mm (16 in.)	0	Yes
South Canal	7.9 m (26 ft)	4.9 m (16 ft)	356 mm (14 in.)	0	No

Figure 3–Typical configuration details for the LVL T-beam bridges.

Figure 4–South Canal bridge, Owyhee County, Idaho.

Figure 5-Mill Creek bridge, Stevens County, Washington.

Figure 6 – Load test truck configurations and axle loads. The transverse vehicle track width, measured center-to-center of the rear wheel lines, was 1.82 m (6 ft).

At Mill Creek, two trucks were used: truck 20 with a GVW of 232 kN (52,200 lb) and truck 13 with a GVW of 227 kN (51,100 lb). The vehicles were positioned longitudinally on the bridges so that the two rear axles were centered at midspan and the front axles were off the bridge span. Transversely, different vehicles positions were used for single-lane and double-lane bridges. At South Canal, the three load positions included load position 1 with the vehicle centered on the bridge width and load positions 2 and 3 with the truck wheel line over the longitudinal bridge centerline (Figure 7). For the Mill Creek bridge, six load positions were used with the vehicles 0.61 m (2 ft) from centerline for load positions 1 through 3 and 1.22 m (4 ft) from centerline for load positions 4 through 6 (Figure 8).

Analytical Evaluation

The load test behavior of each bridge was evaluated analytically to further understand bridge behavior and form a basis for development of design load distribution criteria. Previous research showed that stress-laminated bridge decks could be accurately modeled as orthotropic plates (Oliva et al. 1990). Given the relatively compact section of the T-beams, an orthotropic plate analysis similar to that developed at FPL for stress-laminated decks was investigated. This was done so that the T-beam design criteria could parallel that for stress-laminated decks (currently under devdopment at FPL). To complete the orthotropic plate analysis, the plate thickness was taken as the T-beam flange thickness. To reflect the increase in longitudinal stiffness as a result of the T-beam webs, a transformed section analysis was

Figure 7 –Load test vehicle positions for the South Canal bridge (looking west). For all load positions, the two rear axles were centered over the bridge midspan with the front axle off the span.

completed to determine the equivalent longitudinal MOE of the plate. Properties in the transverse direction for MOE and shear modulus were based on the level of interlaminar compression in the bridge in accordance with the following equations.

$$E_{TS} = 168F_P + 108.6 \quad (E_{TS} = 168F_P + 15{,}749) \qquad (1)$$

$$G_{TS} = 234F_P + 261 \quad (G_{TS} = 234F_P + 37{,}897) \qquad (2)$$

where

E_{TS} = transverse MOE in MPa (lb/in^2)
G_{TS} = transverse shear modulus in MPa (lb/in^2)
F_P = interlaminar compression in MPa (lb/in^2)

An analytical evaluation was completed for each bridge based on load vehicles and conditions, including the level of interlaminar compression at the time of testing. Analysis was also completed for the same load test conditions and transverse vehicle positions using AASHTO HS 20-44 truck loading (AASHTO 1992).

Figure 8 –Load test vehicle positions for the Mill Creek bridge (looking east). For all load positions, the truck rear axles were centered over the bridge midspan with the front axles off the span.

Condition Assessment

The general condition of each of the LVL T-beam bridges was assessed at the time of load testing. The assessments involved visual inspections, measurements, and photographic documentation of the bridge condition. Items of specific interest included the bridge geometry, condition of the LVL components, and the condition of the stressing bars and anchorage systems.

Results and Discussion

Results of the performance of the LVL T-beam bridges follow. Results for moisture content and bar force are based primarily on the Petty Creek bridge, which was continuously monitored for 2.5 years. Load test results and analysis are for the South Canal and Mill Creek bridges.

Moisture Content

All T-beam bridges were installed at a moisture content of 9%-12%. Measurements after installation indicated that the moisture content of the LVL responds relatively quickly to changes in environmental conditions. This is because the lathe checks in the veneers, which facilitate preservative penetration, also allow rapid moisture movement. For the Petty Creek bridge, the average moisture content increased from approximately 12% at the time of construction to approximately 26% at the conclusion of the 2.5-year monitoring program. During this time, variations in moisture content of 5% to 10% were noted on an annual basis as a result of seasonal climatic changes. The 12% increase in moisture content over the monitoring period is due in part to the sheltered location of the bridge, which hinders the drying of the deck, and the last and greatest moisture content readings were taken following the rainy period of the year.

Bar Force

The average bar force for the Petty Creek bridge is shown in Figure 9. At the time of load cell installation in December 1990, the bar force was approximately 112.1 kN (25,200 lb) or 0.97 MPa (140 lb/in^2) interlaminar compression. After increasing slightly, the bar force declined to approximately 90.3 kN (20,300 lb) or 0.78 MPa (113 lb/in^2) interlaminar compression in July 1991. For the remaining 2 years of the monitoring period, bar force remained relatively stable with fluctuations following seasonal climatic changes, which affected the moisture content and dimensional stability of the LVL. At the end of the monitoring period, the average bar force was 82.7 kN (18,600 lb), which is approximately 0.71 MPa (103 lb/in^2) interlaminar compression.

Figure 9–Average trend in bar force for the Petty Creek bridge.

For all the LVL T-beam bridges, the design interlaminar compression of 1.15 MPa (167 lb/in^2) is substantially greater than the 0.69 MPa (100 lb/in^2) commonly used for stress-laminated sawn lumber decks. At this high level, the bar force initially declines due to stress relaxation in the LVL, then stabilizes or increases slightly as the LVL absorbs moisture and expands. Based on the high initial interlaminar compression and initial swelling of the LVL, it is expected that future bar tensioning will not be required for the LVL T-beam bridges.

Load Test Behavior

Load test transverse deflections for the South Canal and Mill Creek bridges are presented in Figures 10 and 11, respectively. The interlaminar compression at the time of the testing was 0.94 MPa (137 lb/in^2) for the South Canal bridge and 1.19 MPa (172 lb/in^2) for the Mill Creek bridge. For both bridges, the deflection plots are similar to the orthotropic plate behavior of stress-laminated deck bridges constructed of sawn lumber (Ritter et al. 1995). At the South Canal bridge, the maximum deflection for load position 1 measured 19 mm (0.75 in.) and occurred under the north wheel line. For load positions 2 and 3, the deflection profiles are approximately symmetrical with maximum deflections of 20 mm (0.78 in.), occurring adjacent to the wheel line at the bridge centerline, and adjacent to the north wheel line, respectively.

At the Mill Creek bridge, the maximum measured deflections for load positions 1 and 2 measured 14 mm (0.56 in.) and occurred at the interior T beam adjacent to the outside wheel line. With both vehicles on the bridge for load position 3, the maximum mea-

sured load test deflection of 17 mm (0.69 in.) was adjacent to the interior wheel line in the south lane. For load positions 4 and 5, the maximum deflections measured 15 mm (0.59 in.) and 14 mm (0.56 in.), respectively, and occurred adjacent to the outside wheel line. The maximum deflection for load position 6 was in the same relative position and measured 15 mm (0.59 in.).

Assuming linear elastic behavior, the sum of the Mill Creek bridge deflections for load positions 1 and 2 and load positions 4 and 5 should equal the deflections of load position 3 and load position 6, respectively. Using superposition, comparative plots of these deflections are shown in Figure 12. As shown, the plots are very similar with only minor differences that are generally within the accuracy of the measurements.

Analytical Evaluation

Comparisons of the measured load test results to the analytical deflections are shown in Figures 13 and 14 for the South Canal and Mill Creek bridges, respectively. As shown, the analytical deflections are very similar to those measured. The apparent differences generally result from analytical deflections greater than those measured with maximum differences of 2 to 3 mm (0.08 to 0.12 in.). Given the accuracy of the load test measurements, the plots indicate that the analytical model accurately represents bridge behavior.

Using the same load test analytical parameters and transverse vehicle positions, the maximum deflection for AASHTO HS 20-44 truck loading for the South Canal bridge was 16 mm (0.63 in.) and occurred for load positions 2 and 3 at the same location as the test vehicle maximum deflection. For the Mill Creek bridge, the maximum HS 20-44 deflection was 26 mm (1.02 in.) and occurred in the two T-beams nearest the bridge centerline for load position 3. Represented as a fraction of the bridge span measured center-to-center of bearings, these deflections are approximately L/482 and L/344 for the South Canal and Mill Creek bridges, respectively.

Condition Assessment

Condition assessments of the LVL T-beam bridges indicated that performance was good, and no significant deficiencies were noted. Inspection of the LVL components showed no signs of deterioration or delamination, and there was no evidence of wood preservative loss or preservative or solvent accumulations on the wood surface. At the Kenally Creek bridge, which was unpaved and did not include the

Figure 10–Load test deflection measurements for the South Canal bridge, with the maximum measured deflection noted for each load position (looking west). Truck wheel line positions are noted with arrows.

LVL box beams along the edges of the bridge, a slight dimensional distortion was noted in the outside T-beams and the web was raised slightly above the bearing. This was attributed to moisture content changes in the LVL and was not evident in any of the other bridges. For all bridges, the exposed steel stressing bars and hardware showed no visible signs of corrosion or other distress, and there was no indication of the bar anchorage crushing into LVL.

Figure 11 –Load test deflection measurements for the Mill Creek bridge, with the maximum measured deflection noted for each load position (looking east). Truck wheel line positions are noted with arrows.

Concluding Remarks

Field evaluation of six stress-laminated LVL T-beam bridges indicates that structural and serviceability performance is good, and the bridges should provide many years of acceptable service. Based on monitoring results, the following conclusions can be made.

- It is feasible and practical to construct stress-laminated bridges using LVL T-beams.
- The moisture content of the T-beams at installation is typically 9%-12%. After installation, the LVL responds relatively quickly to changes in environmental conditions. The moisture content of one bridge in the monitoring program increased from 12% to 26% during 2.5 years.
- Loss of bar force has not been a problem with stress-laminated LVL T-beams. Swelling of the LVL as the material gains moisture tends to maintain a relatively high bar force, despite the effects of wood stress relaxation.

Figure 12–Load position comparisons for the Mill Creek bridge, showing load positions 1 and 2 compared with load position 3 and load positions 4 and 5 compared with load position 6 (looking east).

- Response of the bridges to static truck loading is linear elastic and similar to that for stress-laminated decks constructed of sawn lumber.

- The static load behavior of stress-laminated LVL T-beam bridges can be accurately modeled using a transformed section and orthotropic plate analysis.

Figure 13–Comparisons of the South Canal bridge measured deflections to the analytical deflections based on orthotropic plate analysis (looking west).

Figure 14—Comparisons of the Mill Creek bridge measured deflections to the analytical deflections based on orthotropic plate analysis (looking east).

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Eastern Cottonwood Stress-Laminated Timber Bridges
Enhancing Rural America with Underutilized Species

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Abstract
In 1988, the U. S. Congress passed legislation known as the Timber Bridge Initiative (TBI). The purpose of the legislation was to establish a national program to provide effective and efficient utilization of wood as a structural material for highway bridges. As part of the TBI, a demonstration bridge program was developed to enhance rural America by encouraging innovation through the use of new or previously underutilized wood products. In south-central Iowa, two counties have constructed stress-laminated timber bridges using eastern cottonwood, a local, underutilized species. This paper presents a summary of the performance of three stress-laminated eastern cottonwood bridges constructed in 1994.

Keywords: Bridge, eastern cottonwood, field performance, stress-laminated, timber, wood

Introduction
Approximately 41% of the 578,000 highway bridges in the United States are currently in need of repair or replacement (USDA 1995). Because many of these bridges are short-span crossings, they are ideally suited for wood construction. To address the problem of the deteriorating infrastructure, the U. S. Congress passed legislation known as the Timber Bridge Initiative (TBI) in 1988. The objective of the TBI was to establish a national program to encourage the effective and efficient use of wood as a structural material for highway bridges.

The USDA Forest Service was assigned responsibility for the development and implementation of a National timber bridge program under the TBI. Three emphasis areas were identified: technology transfer, demonstration bridges, and research. The Forest Service Timber Bridge Information Resource Center (TBIRC) in Morgantown, West Virginia, maintains the technology transfer program and administers the demonstration bridge program. The demonstration bridge program provides matching funds on a competitive basis to local governments for the construction of timber bridges that illustrate the use of new or previously underutilized wood products, bridge designs, or design applications.

Responsibility for the research portion of the TBI was assigned to the USDA Forest Service, Forest Products Laboratory (FPL), a national wood utilization research laboratory. As part of the research program, FPL assists local governments in evaluating the field performance of demonstration timber bridges. This enables FPL to collect, analyze, and distribute information regarding field performance and provides a basis for validating and/or revising design criteria, thereby improving efficiency and economy in timber bridge design, fabrication, and construction.

Background

As a result of the TBI, numerous timber bridges have been constructed from wood species not previously used in structural applications. One of these species is eastern cottonwood. Traditionally, eastern cottonwood is used for lightweight containers, interior furniture parts, plywood core stock, and pulp for paper production. Because it is an abundant, readily available species and is the fastest growing tree in North America, eastern cottonwood has the potential to be an economic alternative to traditional structural species (Kennedy 1985). However, the strength and stiffhess of cottonwood are considerably less than the commonly used structural species.

In 1992, the first known stress-laminated deck utilizing eastern cottonwood was constructed over Cooper Creek in Centerville, Appanoose County, Iowa. Although not a demonstration bridge under the TBI, the decision to construct this type of bridge was prompted by information obtained through the TBIRC. Because few stress-laminated timber bridges had been built in the United States at that time, and little information was available regarding design criteria and construction specifications, FPL was contacted for technical assistance. Through meetings with state, local, and FPL representatives, it was determined a stress-laminated bridge deck constructed with eastern cottonwood lumber laminations was feasible. Upon developing a cooperative agreement with several agencies, FPL developed and executed a 28-month bridge performance monitoring program (Ritter and others 1995b).

Based on the success of the Cooper Creek bridge, three more eastern cottonwood stress-laminated bridges were constructed in southern Iowa during 1994. Since the use of eastern cottonwood in stress-laminated decks was still a relatively new concept, FPL was contacted to implement performance evaluation programs for each of the three bridges. As a result, FPL initiated a field monitoring and evaluation program for these bridges.

Objective

The objective of the FPL monitoring program is to evaluate the performance of the stress-laminated cottonwood bridges for a minimum of 2 years, beginning shortly after bridge installation. The performance evaluation includes data collection and analysis related to the bridge behavior under static truck loading and general structural performance. The results will be evaluated with those of the Cooper Creek bridge to establish recommendations for the design and construction of future eastern cottonwood stress-laminated bridges. They will also be reviewed with results from field evaluations of stress-laminated bridges constructed using other species in order to formulate and improve design criteria for stress-laminated timber bridge decks.

Table 1 – SI conversion factors.

English unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile	1.609	kilometer (km)
pound (lb)	0.14	Newton (N)
lb/in^2 (stress)	6,894	Pascal (Pa)

Description of Bridges

In 1994, three eastern cottonwood stress-laminated bridges were constructed in southern Iowa: the Hibbsville and Dean bridges in Appanoose County and the Decatur bridge in Decatur County. Numerous similarities exist between the bridges. Each is a simple-span, stress-laminated deck, approximately 24 ft long and was constructed with eastern cottonwood lumber pressure treated with creosote (Figure 1). (See Table 1 for metric conversion factors.) Because available lamination material did not extend the full length of the bridge, butt joints were placed transverse to the bridge span in every fourth lamination. Longitudinally, butt joints in adjacent laminations were separated by 4 ft (Figure 2), Each bridge is located on a low-volume county road and is unpaved (Figure 3).

The stressing system for each of the three bridges consists of six 1-in.-diameter high strength steel bars, spaced 48 in. on-center. The bars were designed to be tensioned to provide a uniform compressive stress of 100 lb/in^2 between the lumber laminations. The bar anchorages are discrete plate systems that are similar in design, although actual plate dimensions vary. Figure 4 illustrates the anchorage configuration for the Dean bridge. A unique feature of the anchorage configuration for the Dean and Decatur bridges is an untreated white oak plate between the exterior lamination and the steel bearing plate. The anchorage system for the Hibbsville bridge uses a steel plate in place of the white oak plate.

Figure 1 – Typical configuration of the eastern cottonwood stress-laminated bridges. Curb is shown for illustration only and does not represent actual systems used.

Figure 2 - Typical butt joint configuration. Butt joints were placed transverse to the bridge span in every fourth lamination. Longitudinally, butt joints in adjacent laminations were separated by 4 ft.

Although the general configuration and many characteristics of the Hibbsville, Dean, and Decatur bridges are similar, each has several distinct features. A brief summary of each bridge follows.

Figure 3 – Completed Hibbsville (top), Dean (middle), and Decatur (bottom) bridges.

Hibbsville

The Hibbsville bridge is located on a dead-end gravel roadway approximately 2 miles southwest of Numa, Iowa (Figure 5). It is a single-lane deck, 14-in. deep and approximately 17-ft wide. The design bar force required to achieve 100 lb/in^2 interlaminar compression is 67,200 lb. The rail system is composed of a steel w-beam bolted to steel angle posts. Vehicles ride directly on the deck surface because no wearing surface was applied.

Dean

The Dean bridge is on a double-lane, gravel roadway approximately 5 miles southwest of Moulton, Iowa (Figure 5). It is a double-lane bridge, approximately 23-ft wide, with a deck thickness of 15.5-in. A design bar force of 74,400 lb is required for 100 lb/in^2 interlaminar compression. The bridge includes an 18-in.-high timber curb and a 3-in.-thick gravel wearing surface.

Decatur

The Decatur bridge is located on a field entrance roadway approximately 2 miles southwest of Davis City, Iowa (Figure 5). It is a double-lane bridge, 14-in. deep, and approximately 2l-ft wide. As with the Hibbsville bridge, the design bar force of 67,200 lb is required to achieve 100 lb/in^2 interlaminar compressive stress. The bridge includes a 17-in.-high timber curb system and a gravel wearing surface.

Research Methods and Results

As previously mentioned, FPL was contacted by local government officials to develop and implement a performance evaluation program for the three eastern cottonwood stress-laminated bridges. The program required monitoring several performance indicators, including moisture content, bar force, static load behavior, and general structural condition. The research methods utilized procedures and equipment previously developed (Ritter and others 1991; Wacker and Ritter 1992). Preliminary results for the first 22 months of the 24-month monitoring for each bridge follow. In most cases, results from the Decatur bridge monitoring are presented but are also representative of the performance of the Hibbsville and Dean bridges.

Figure 4 – Details of the Dean bridge discrete plate bar anchorage configuration.

Moisture Content

Global changes in the moisture content of stress-laminated decks can significantly affect the performance of the structure. If moisture is lost, the deck can shrink, resulting in a decrease in stressing bar force. Conversely, if moisture is gained, swelling of the timber can occur and cause an increase in stressing bar force. Changes in moisture content can also affect the deck stiffness, creep, transverse stress relaxation, and anchorage system performance. The effect of global moisture content changes in a stress-laminated timber bridge depends largely on the moisture content of the laminations at the time of construction.

Moisture content of the eastern cottonwood bridges was measured using an electrical-resistance moisture meter with 3-in. insulated probe pins in accordance with ASTM D4444-84 (ASTM 1990). Measurements were obtained by driving the pins into the underside of the deck at depths of 1 to 3 in., recording the moisture content values, and adjusting the values for temperature and wood species (Forintek 1984).

For each bridge, the deck lamination moisture content was measured at the first load test, which occurred shortly after bridge installation. At that time, the average lamination moisture content for the Hibbsville, Dean, and Decatur bridges was approximately 25%, 24%, and 23%, respectively. These values substantially exceeded the equilibrium moisture content, which is expected to average 18% to 20% (McCutcheon and others 1986). Since bridge installation, the average lamination moisture content of each bridge has remained relatively stable with minor fluctuations of 3% to 5% in the measurement zone as a result of seasonal climatic changes.

As the lamination moisture content gradually decreases toward equilibrium, the deck width will decrease, resulting in bar force loss. Ideally, the moisture content at installation should be 16% or less. This would result in the global moisture content increasing toward equilibrium, thereby causing the deck to swell, which tends to increase bar force and offset bar force loss caused by stress relaxation.

At the conclusion of the monitoring period, the average lamination moisture content for each deck will be measured and compared with earlier measurements to determine if the global moisture content of the decks is moving toward an equilibrium level.

Figure 5 – Location maps for the Hibbsville, Dean, and Decatur bridges.

Bar Force

Stress-laminated bridges perform properly when adequate compression is maintained between the lumber laminations to prevent vertical slip. At the time of construction, stressing bars are normally tensioned to a design force that results in an interlaminar compression level of 100 lb/in^2. It is assumed that 50% to 60% of the compressive stress will be lost over time due to stress relaxation in the laminations. This loss is considered acceptable because slip between the laminations does not occur until the interlaminar compression level is 20 to 24 lb/in^2 (Ritter and others 1995a).

Interlaminar deck compression is evaluated by monitoring force in the stressing bars. To monitor bar force, load cells developed at FPL were installed on two stressing bars of each bridge (Wacker and Ritter 1992). Load cell measurements for the Hibbsville and Decatur bridges were obtained on a monthly basis by county personnel, using a portable strain indicator. For the Dean bridge, load cell measurements were obtained on an hourly basis through a remote data acquisition system. For each bridge, load cell strain readings were converted to units of bar tensile force by applying a laboratory calibration factor to the strain indicator reading.

The average trend in bar force for the Decatur bridge is illustrated in Figure 6 and is typical of the bar force trend for the Hibbsville and Dean bridges. At the start of the monitoring period, the bars were tensioned to approximately 53,900 lb, or 80% of the design force. As indicated in Figure 6, the average force dropped to approximately 37,400 lb, or 56% of the design force, within the first month of the monitoring period. At the end of the first month, the rate of loss greatly decreased.

Figure 6 – Average trend in bar force of the Decatur bridge.

Several factors contribute to changes in bar force, including wood stress relaxation, moisture content, and bar anchorage performance. General observations can be made regarding the impact of each factor, although the exact effect of one variable cannot be accurately defined because of the complex interaction of the factors.

Stress Relaxation – When deck laminations are subject to long-term loads applied by the stressing bars, the wood slowly compresses across the bridge width resulting in bar force loss. This occurrence is known as stress relaxation and is greatest when the deck is initially stressed, then gradually decreases. The large initial bar force loss and subsequent slower force reduction illustrated in Figure 6 is indicative of stress relaxation. Because bar force loss caused by stress relaxation increases as the moisture content of the wood increases, bar force loss in the three cottonwood bridges was likely affected by the relatively high moisture content of the laminations.

Moisture Content – At this point in the monitoring period, the moisture content of the three bridges has not significantly decreased. Therefore, the current bar force loss cannot be attributed to a decrease in moisture content. This phenomena will be examined further when the moisture content is measured at the conclusion of monitoring. Long-term loss caused by the moisture content decreasing towards equilibrium is anticipated.

Bar Anchorage – The performance of the bar anchorage directly affects bar force. Anchorage systems are intended to distribute bar force into the deck without causing wood crushing in the exterior laminations. If crushing occurs, bar force loss can be great.

The discrete plate anchorage system employed on two of the cottonwood bridges is unique because a white oak plate was placed under the steel bearing plate. Normally fill-length dense hardwood exterior laminations are used in stress-laminated bridges, because they have stronger compression perpendicular to grain characteristics and are less likely to experience crushing beneath the anchorage plates. Cottonwood is relatively weak in compression perpendicular to grain; therefore, the white oak plates were added to the anchorage system in hopes that they would perform the same function as the full-length dense hardwood laminations. Crushing of the exterior laminations has not occurred, indicating that the white oak plates are effective in distributing the load. The bar force losses illustrated in Figure 6 cannot be attributed to anchorage performance because crushing has not been observed.

Figure 7 – Vehicle configurations and axle loads. The single axle (right) represents the front of the vehicle. The track width, measured center-to center of the rear tires, was 6 ft.

Load Test Behavior

Static load testing of the bridges is important to the overall bridge monitoring program because information obtained from the tests is used to refine and improve design procedures and evaluate effects of design variables on bridge performance. To date, one load test has been performed on each bridge. It is anticipated that the second group of tests will be conducted by the end of 1996. In each of the tests, fully-loaded trucks were positioned on the bridge deck and resulting deflections were measured at a series of locations along the bridge midspan. Measurements were taken prior to testing (unloaded), for the load positions (loaded), and at the conclusion of testing (unloaded) by suspending calibrated rules from the deck underside and reading values to the nearest 0.04 in. with a surveyor's level. The accuracy of the measurements is estimated to be +/- 0.02 in. A summary of each load test follows.

Hibbsville – The Hibbsville bridge load test was conducted May 11, 1994. A single test vehicle, Truck A, with a gross vehicle weight of 49,800 lb (Figure 7),

Figure 8 – Transverse load positions (looking east). For all load cases, the two rear axles were centered about the bridge midspan with the front axles off the span. Drawings are for illustration only and are not to scale.

was positioned longitudinally with the two rear axles centered about the bridge midspan. Transversely, three load positions were used (Figure 8).

Dean – The Dean bridge load test was also conducted May 11, 1994, and used the same test vehicle (Figure 7) and longitudinal vehicle position used for the Hibbsville bridge load test. Transversely, two load positions were used (Figure 8).

Decatur – The load test was conducted August 3, 1994. Two test vehicles were employed: Truck B with a gross vehicle weight of 49,840 lb and Truck C with a gross vehicle weight of 50,680 lb (Figure 7). As with the previous load tests, the trucks were positioned longitudinally with the two rear axles centered about the bridge midspan. Transversely, three load positions were used (Figure 8).

Results of the Decatur bridge static load test are presented in Figure 9. For each load position, transverse deflections are shown at the bridge midspan as viewed from the west, looking east. No permanent residual deformation was measured at the conclusion of testing and movement at the abutments was not detected. For load positions 1 and 2, the maximum measured deflection of 0.33 and 0.37 in., respectively, occurred beneath the outside wheel line. The absolute maximum measured deflection of 0.45 in. occurred at the bridge centerline for load position 3, with both vehicles on the bridge.

Assuming linear elastic behavior, uniform material properties, proper vehicle placement, and accurate deflection measurements, the summation of the deflections resulting from two individual truck loads applied separately should equal the deflection resulting from both trucks applied simultaneously. This is illustrated in Figure 10, where the sum of load positions 1 and 2 is compared to load position 3. The plots are virtually identical with only minor variations within the accuracy of the measurement methods. The results presented in Figure 10 indicate that the bridge behavior under the applied loads is within the linear elastic range. Results from the load tests performed on the Hibbsville and Dean bridges also indicate linear elastic behavior.

Condition Assessment

The general condition of each eastern cottonwood stress-laminated bridge was assessed at the beginning of the monitoring period and involved visual inspections, measurements, and photographic documentation. Items of specific interest included the bridge geometry and the condition of the timber deck, stressing bars, and anchorage system. A second assessment

Figure 9 – Transverse deflection for the Decatur bridge load test, measured at the bridge centerspan (looking east). Bridge cross-sections and vehicle positions are shown to aid interpretation and are not to scale.

will be made at the conclusion of the monitoring period and compared to the first in order to further evaluate bridge behavior and performance. Specific observations pertaining to the individual bridges follow.

Hibbsville – The first condition assessment of the Hibbsville bridge indicated that the wood components are performing well with no signs of deterioration. Inspection of the anchorage system revealed no signs of crushing of the discrete plate anchorage into the outside laminations and no measurable distortion of the bearing plate. The exposed steel stressing bars and nuts showed no signs of corrosion; however, because the steel bearing and anchor plates were not galvanized, they are experiencing corrosion.

Dean – Results of the Dean bridge condition assessment were similar to those of the Hibbsville bridge. The wood components and exposed steel stressing bars and nuts showed no signs of deterioration. The anchorage system was performing properly with no noticeable signs of crushing into the exterior laminations; however, the steel bearing and anchor plates exhibited corrosion because they were not galvanized. Inspection of the bridge geometry revealed that the width was distorted. The south edge of the deck was essentially straight, and the north edge of the deck was approximately 8.5 in. narrower in width at the midspan than at the abutments. It is suspected that this is largely attributable to the layout of the cottonwood laminations, which varied in width as much as 0.5 in.

Figure 10 – Comparison of the sum of the deflections from load position 1 and 2 to the measured deflections of load position 3 (looking east). Transverse deflections were measured at the bridge centerspan.

Decatur – Evaluation of the individual wood components and the anchorage system during the condition assessment of the Decatur bridge indicated acceptable performance. The steel components exhibited no visible signs of corrosion. Along the west abutment, it was noted that both comers curled upward approximately 1.5 in. away from the abutment (Figure 11). At both edges, the displacement of the deck away from the deck gradually reduced until at 18 in. from each deck edge, the deck made contact with the abutment. This will be reexamined at the second condition assessment.

Figure 11 – Deck distortion at the west abutment of the Decatur bridge.

Concluding Remarks

To date, these three eastern cottonwood stress-laminated bridge decks in Iowa are performing well and are expected to provide many years of acceptable service. Based on the monitoring conducted thus far, the following observations and recommendations are given.

- It is feasible and practical to construct stress-laminated decks using eastern cottonwood, a species not typically used in structural applications.

- The moisture content of the Hibbsville, Dean, and Decatur bridges at installation was approximately 25%, 24%, and 23%, respectively. The moisture content of the bridges has remained relatively stable, with minor fluctuations of 3% to 5% in the measurement zone as a result of seasonal climatic changes. This somewhat high moisture content has not adversely affected the structural integrity of the bridges, although it has likely contributed to a high level of stress relaxation. A lamination moisture content less than or equal to 19% at installation is recommended.

- Currently the bridges are either unsurfaced or have a gravel wearing surface. The gravel and debris tracked onto the bridge trap moisture and do not allow the deck to dry. The addition of an asphalt wearing surface and an underlying asphalt impregnated geotextile fabric would help keep the wood decks dry.

- Each bridge experienced rapid bar force loss within the first month following stressing of the deck. The average bar force in the Decatur bridge decreased from 80% to 56% of the design force during the first month of monitoring, at which time the rate of loss greatly decreased and stabilized. The majority of the bar force losses in the bridges is attributed to stress relaxation in the cottonwood laminations.

- Load tests indicate that the bridges are performing within the linear elastic range. Results of a second load test on each bridge will be compared to the first load test to verify bridge behavior and performance characteristics.

- The anchorage system of each bridge is performing well, although the bearing and anchor plates are exhibiting corrosion because they were not galvanized. The exposed steel stressing bars and nuts show no visible signs of corrosion. All steel components should be galvanized to prevent deterioration. Crushing in the exterior laminations is not evident.

- In two of the bridges, white oak bearing plates were used. At this time, the plates are effectively distributing the bar force into the deck.

- The timber components of each bridge exhibit no signs of deterioration.

- The irregular width of the Dean bridge, is likely caused by unequal lamination widths and subsequent lamination layout.

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Dynamic Evaluation of Timber Bridges

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Abstract

The dynamic response of both glued-laminated timber (glulam) girder bridges and stress-laminated (stress-lam) deck bridges was determined from field test results using heavily loaded trucks. Deflections were measured for various vehicle speeds and transverse positions at the bridge midspan and were recorded using a high speed data acquisition system. A dynamic amplification factor (DAF) was computed from these data. These field tests are part of a larger research study that will also include analytical research. Only some of the experimental data are described in this paper. The overall objective of the larger study is to determine the dynamic behavior of timber bridges so that reliable design specifications can be developed.

Key Words: Timber, Bridge, Dynamic, Glued laminated, Girder, Stress laminated, Deck

Introduction

Wood has been used as a bridge material in the United States for hundreds of years. Despite the exclusive use of wood bridges during much of the 19th century, the 20th century brought a significant decline in the percentage of wood bridges relative to those of other materials. At the present time, approximately 8% of the bridges listed in the National Bridge Inventory are wood (FHWA 1995). Recently, there has been a renewed interest in wood as a bridge material, and several national programs have been implemented to further develop wood bridge systems. As a result of the Timber Bridge Initiative and the Intermodal Surface Transportation Efficiency Act, passed by Congress in 1988 and 1991, respectively, funding has been made available for timber bridge research (Duwadi and Ritter 1994).

A portion of the timber bridge research is aimed at refining and developing design criteria for wood bridge systems. This project, to investigate the dynamic characteristics of wood bridges, is part of that program and involves a cooperative research study between Iowa State University; the USDA Forest Service, Forest Products Laboratory; and the Federal Highway Administration. The first phase of the project addressed the dynamic performance of stress-laminated timber bridge decks (Ritter et al. 1995). The second phase of the project assessed the dynamic characteristics of glulam timber girder bridges (Wipf et al. 1996). The third phase of the project will assess the dynamic characteristics of longitudinal glulam timber deck bridges.

General Research Program

Field tests for this program were designed to observe bridge deflections and vertical accelerations and test vehicle vertical accelerations under both static and

Figure 1 -- Layout of a typical glulam girder bridge

dynamic loading. Vertical deflections were measured for several vehicle velocities for two different bridge entrance conditions: the in situ condition and that due to an artificial bump at the entrance, which was used to simulate a potential rough bridge entrance condition that might occur in the field. Dynamic deflection data were compared to static deflections to quantify a dynamic amplification factor (DAF) for each test.

Description of Bridges

Glulam timber girder bridges typically consist of a series of longitudinal glulam beams that support transverse glulam deck panels (Fig. 1). The girders are available in standard nominal widths, ranging from 10.1 to 40.6 cm (4 to 16 in.) with girder depth limited only by transportation and pressure treating restrictions. Deck panels are usually 12.7 to 17.1 cm (5 to 6 3/4 in.) thick, 1.2 m (4 ft) wide, and are continuous across the bridge width. Lateral support and alignment of the girders are provided by transverse bracing at the bearings and at intermediate locations along the span. Glulam girder bridges are feasible for spans ranging from 6 to 42 m (20 to 140 ft), although most are in the span range of 7 to 24 m (25 to 80 ft).

Stress-laminated timber bridge decks (see Fig. 2) consist of a series of wood laminations that are placed edgewise between supports and stressed together with high strength

Figure 2 -- Layout of typical stress-lam timber deck bridge

steel bars (Ritter 1990). The bar force, which typically ranges from 111.2 to 355.8 kn (25,000 to 80,000 lb), squeezes the laminations together so that the stressed deck acts as a solid wood plate. The concept of stress laminating was originally developed in Ontario, Canada, in 1976 as a means of rehabilitating existing nail-laminated lumber decks that delaminated as a result of cyclic loading and wood moisture content variations (Taylor and Csagoly 1979; Taylor et al. 1983). In the 1980s, the concept was adapted for the construction of new bridges and numerous structures were successfully built or rehabilitated in Ontario using the stress-laminating concept. The first stress-laminated bridges in the United States were built in the late 1980s. Since that time several hundred stress-laminated timber bridges have been constructed, primarily on low volume roads.

The bridges tested in this program involve both glulam girder bridges and stress-laminated deck bridges. Table 1 summarizes information about the 12 bridges presented in this paper.

Table 1 -- Bridge description (1 m = 3.28 ft)

Bridge	Type	Span C-C bearings (m)	Width (m)	Bridge and bridge approach pavement type
Trout	Stress-lam deck	13.99	7.83	Asphalt
Little Salmon	Stress-lam deck	7.62	4.72	Unpaved
Lampeter	Stress-lam deck	6.77	9.02	Asphalt
Capitola	Stress-lam deck	7.13	10.85	Asphalt
Olean	Stress-lam deck	7.68	9.36	Asphalt
Schuylkill	Stress-lam deck	10.64	7.92	Chip Seal
Wadesboro	Stress-lam deck	6.92	11.19	Asphalt
Teal	Stress-lam deck	9.60	7.25	Asphalt
Chambers County	Glulam girder	16.18	8.23	Asphalt
Mud Creek	Glulam girder	12.74	6.80	Chip Seal
Wittson - Span 1	Glulam girder	15.70	4.63	Asphalt
Wittson - Span 3	Glulam girder	31.09	4.63	Asphalt

Instrumentation

The dynamic response of each bridge was recorded during the passage of the three axle trucks traveling at constant velocity. Deflections were measured at midspan and quarterspan of each bridge span using Celesco potentiometer transducers (DCPT). Accelerometers were also mounted at several locations on the bridge at midspan and quarterspan. Details of the complete instrumentation can be found in Ritter et al. (1995).

Acceleration data were also collected on the vehicle simultaneously with the bridge DCPT data. The accelerometers were mounted on the vehicle frame over the rear axles and on the rear tandem axle.

Test Procedure

The dynamic load behavior of the bridge was evaluated for several vehicle velocities for in situ and artificially rough approach conditions at the bridge entrance. For the two-lane bridges, two different transverse vehicle positions were used: (1) eccentric, with the left wheel line (driver side) 0.61 m (2 ft) to the right of centerline and (2) concentric, with the axle of the truck centered on the bridge (i.e., straddling the centerline). For each bridge, the test vehicles used were tandem axle dump trucks with steel leaf rear suspensions. The range of gross vehicle weights was 244.6 to 275.8 kn (55 to 62 kips).

To obtain a basis by which the dynamic load effects could be compared, crawl tests were performed for each loading position. During these crawl tests, the vehicle velocity was approximately 8 kmh (5 mph). Deflections at higher velocities were then obtained with velocities ranging from 16 kmh (10 mph) to safe upper limit speeds based on bridge alignments.

Data Processing

A plot of bridge deflection versus vehicle position along the bridge length (using the vehicle front axle as a reference) was made for each DCPT location at midspan. A typical example of the dynamic versus crawl deflections is given in Figure 3. The maximum deflection obtained for crawl speed is δ_{stat}. The maximum dynamic deflection is δ_{dyn}.

A dynamic amplification factor (DAF) was computed for each bridge. Each DCPT location was scanned to find the maximum absolute crawl deflection, and this data point was then used as the reference point for calculation of the DAF. As per recommendations by Bahkt and Pinjarkar (1989), this approach yields the most useful design information. It should be noted that the data point with the highest crawl deflection typically had the highest dynamic response. The DAF was computed as

$$DAF = 1 + \left(\frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \right) \qquad (1)$$

where

DAF = dynamic amplification factor,

δ_{dyn} = maximum deflection under the vehicle traveling at designated speed, and

δ_{stat} = maximum deflection under the vehicle traveling at crawl speed.

Figure 3 -- Typical bridge dynamic deflection data for bridge tests

Plots of the dynamic amplification factor are shown in Figure 4. In this paper, only the DAF data associated with the in situ conditions are presented (i.e., no artificial rough entrance conditions are given). A summary of the maximum DAF and the experimental fundamental frequency for each bridge is shown in Table 2.

Discussion of Results

From observation of the DAF data presented here and more detailed analysis not presented in this paper, it is clear that DAF data from a finite number of experimental tests provides only limited information about the complex dynamic behavior of the vehicle/bridge system (Ritter et al. 1995, Wipf et al. 1996). The DAF data do provide some limited information about the possible amplification of

deflections that can be expected. The highest DAF recorded was 1.60 for the Little Salmon bridge.

Table 2 -- Summary of the experimental field data

Bridge	Maximum DAF	Experimental fundamental frequencies (Hz)
Trout	1.23[a]	3.9
Little Salmon	1.60[b]	8.6
Lampeter	1.10	10.6
Capitola	1.04	9.0
Olean	1.18	10.7
Schuylkill	1.08	5.9
Wadesboro	1.10	9.6
Teal	1.18	7.4
Chambers County	1.12	6.4
Mud Creek	1.38[a]	8.9
Wittson - Span 1	1.15	5.9
Wittson - Span 3	1.09	2.8

[a] in situ conditions included rough entrance.
[b] in situ condition included both rough approach and rough entrance.

However, it should be noted that the approach conditions for this bridge, which was an unpaved gravel road, were extreme, and much worse than the others, and definitely contributed to the high DAF value. Mud Creek bridge had the next highest DAF of 1.38, but it also had a rough in situ bridge entrance condition. The next highest DAF was 1.23 for Trout bridge, which also had a rough in situ bridge entrance condition. In general, the DAF is magnified when the vehicle's initial conditions (i.e., bounce and pitch motion) are magnified by either the approach conditions of the roadway or the entrance conditions at the bridge. For bridges with smooth entrance conditions, the highest DAF was 1.18 for both Olean and Teal stress-lam bridges. For the glulam stringer bridges, the highest DAF for smooth entrance conditions was 1.15 for span 1 of the Wittson bridge.

In summary, although DAF data are not exclusive indicators of dynamic behavior, the experimental values represent actual field performance under the test conditions. As this study progresses and more data are available, further analysis and recommendations will be forthcoming.

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Figure 4 -- Plots of in situ DAF

Figure 4 -- Plots in situ DAF (continued)

Timber Bridge Hardwood Glulam Deck Connector Evaluations under Static and Repetitive Loads

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Abstract

Timber bridge deck connectors are crucial to predicted bridge behavior and long-term serviceability yet only limited information documents the performance characteristics for these important bridge connections. New data are presented on the load resistance performance of two different fastener installations often utilized for glulam timber deck-to-girder connection in bridge superstructure construction. Fastener installations investigated are the lag bolt connector and deck clip system. Results on lag bolt and deck clip performance are based on tests with hardwood glulam deck-to-girder specimen assemblies. Hardwood glulam timber are a recent innovation to highway bridge construction with no detailed information to describe their connection behavior. Specimen assemblies were tested to evaluate lateral resistance (shear) both in primary and secondary load orientations and withdrawal resistance (axial) properties. Test efforts for lateral resistance with the lag bolt connections included assembly evaluations of load resistance under monotonic conditions and after repetitive shear displacement up to one million (10^6) cycles. Test measurements were made to determine load-deformation response for characterization of initial slope or elastic stiffness (Kl), post-yield slope (K2) and determination of five percent offset load (yield strength) for both connector systems.

Keywords: Connections, Hardwood, Glulam, Lag Bolt, Deck Clip, Deck-to-Girder, Lateral Resistance, Withdrawal, Load-Slip, Static, Repetitive Loading Fatigue.

Introduction and Background

Timber bridge deck connectors must resist vehicular and braking loads and provide partial composite action for a superstructure system. Accordingly, these connections are a vital link in the structural behavior of a timber bridge and its long-term structural serviceability. One typical superstructure for highway bridge construction is the longitudinal stringer with transverse deck panels of glue-laminated (glulam) material (Ritter, 1989). This type superstructure is one of the hardwood-based glulam timber bridge designs in BLC-560 developed for the Pennsylvania Department of Transportation (PennDOT). Manbeck et al. (1994) provides detailed discussion on development of these standard bridge design and construction plans. Document BLC-560 plan series (PennDOT, 1994) cover 18-90 ft clear span highway structures with options for either red maple, yellow poplar or red oak glulam timber bridge construction. Currently, the BLC-560 (1994 edition) relies exclusively on galvanized A307 steel 3/4-in. diameter lag bolt connectors for deck-to-girder bridge installation.

Various activities are underway to modify timber bridge specifications to a metric version (BLC-560M) and update the plan series from ASD (Allowable Stress Design) to LRFD (Load Resistance Factor Design) basis (Manbeck et al., 1996). The update brings PennDOT bridge designs into compliance with the American Association of State Highway Transportation Officials (AASHTO) LRFD specification (AASHTO, 1994). BLC-560M will also be expanded to provide hardwood glulam deck designs with construction details for application to new or retrofit steel girder bridge construction. Activities include investigations on composite behavior for improvements on bridge girder design efficiency. One important effort has been finite element modeling to define interlayer partial composite behavior of the BLC-560 girder with transverse deck system (Witmer, 1996). Critical to this activity are the load-slip properties of bridge deck-to-girder connections and other load transfer mechanisms. Research with softwood bridge designs (Gutkowski et al., 1978) indicate that composite behavior is highly dependent on connection stiffness. Hardwood glulam in highway bridge construction represents a recent innovation. Review of literature fails to reveal detailed information on the performance properties of deck connectors installed with hardwood glulam. With this void in information PSU (Penn State University) researched have instituted an intensive test program to evaluate BLC-560 bridge connector performance.

Connector performance evaluations include standard lag bolts and alternative deck clip installations. Deck clip installations are under consideration as a BLC-560M bridge construction detail. Test program focus also includes other BLC-560 load transfer mechanisms: steel dowels employed for adjacent deck-to-deck connection and hardwood glulam diaphragm versus steel cross frame bracing for stringer-to-stringer connection. Further tests examine connectors for installation of hardwood glulam deck panels on steel girder bridge systems. The intensive scope of this test program has been the subject of two recent studies (Thomforde, 1995 and Witmer, 1996). Today's presentation focuses on lag bolt and deck clip connection performance evaluations (Thomforde, 1995). Connector performance testing includes primary and secondary lateral (shear) resistance and direct withdrawal (axial) resistance for simulated BLC-560 deck-to-girder assemblies. Other bridge connector research findings will be presented at the upcoming IWEC (International Wood Engineering Conference: New Orleans, LA).

Primary lateral resistance refers to connector shear testing in the longitudinal bridge direction and secondary implies a load orientation perpendicular to primary test orientation. Connection data analyses were performed to obtain load-slip properties for elastic stiffness (K1), post-yield stiffness (K2), and yield strength. Evaluation tests for both connectors were conducted under monotonic (single force application) static load conditions. Additional test measurements summarize lag bolted deck-to-girder load-slip behavior after exposure to repetitive shear displacement to examine connection fatigue. One special study consideration was the performance of lag bolt deck correctors installed with glulam girders fabricated with unglued edge-to-edge lumber laminations. Red maple glulam beams with unglued combination 2x4/2x6 lumber laminations were previous investigated (Janowiak, et al., 1995) for inclusion into BLC-560M bridge applications. Edge-to-edge gaps are possible for this type glulam timber and the affect of these dislocations may prove consequential to connector performance.

Experimental

Test Evaluation Objectives

Performance characteristics of assembled deck-to-girder specimens were tested to fullfill several study goals or objectives (Thomforde, 1995):

•To determine yield strength (5% offset load) and stiffness (K1) differences in withdrawal loading between test assemblies having solid lumber lamination glulam girders and combination unglued edge-to-edge multiple piece lumber lamination glulam girders.

•To determine yield strength (5% offset load) and stiffness (Kl) differences between glulam deck-to-girder assemblies connected with lag bolt versus deck clip resistance under monotonic load conditions for both primary and secondary test orientations.

•To determine the lag screw lateral resistance yield strength (5% offset load) and stiffness (Kl) after exposure to cyclic loading of deck-to-girder connections with both solid lamination glulam girders and combination unglued edge-to-edge lamination glulam girders for both primary and secondary test orientations.

•To determine the stiffness and lateral resistance of lag bolted red maple glulam deck-to-girder connections after exposure to cyclic shear displacements.

Fulfilling these study objectives were of primary concern but additional analyses were made to characterize post-yield stiffness (K2). This paper concentrates on the elastic stiffness connection data.

Experimental Deck-to-Girder Evaluations

Experimentation to evaluate deck-to-girder load resistance characteristics did not include all possible test subsets of BLC-560 glulam timber or wood species (Red Maple, Yellow Poplar and Red Oak) with respect to deck connector type (lag bolt versus deck clip installation), glulam type (combination 2x4/2x6 versus solid lamination). Figure 1 depicts the overall experimental design for lateral resistance and withdrawal resistance connector performance evaluations under static and cyclic loading test modes. With red maple glulam (Figure 1) two A307 3/4-in. lag bolt connection test series were included for experimentation (e.g. 9-in. versus 12-in. Lag Bolt). Different deck-to-girder assemblies included a 3.125-in. deck (9-in. lag bolt connection) and a thicker 5.125-in. deck (12-in. lag bolt connection). Both installations correspond to typical BLC-560 decks. Alternative deck-to-girder connectors (WEYCO DECK CLIP) for evaluation are a cast aluminum 90 degree comer bracket with offset toothed cleat for deck connection.

Deck-to-Girder Specimen Preparation

Deck-to-girder connection specimens were derived from glued-laminated timbers that met recognized manufactured standards (ANSI, 1992) with fabrication from No. 2 & better grade lumber. Thomforde, 1995 provides explanation on glulam timber with deck-to-girder connection test specimen preparation. Specimen assemblies were devised so that installation hole placement for all test connections compiled with minimum end and edge requirements for full NDS (National Design Specifications) connector design load (NFPA, 1991). Pilot holes for the threaded lag screw installation into the glulam girder section were drilled 5/8-in., 9/16-in., and l/2-in. diameter for red oak, red maple, and yellow poplar, respectively. These diameters meet NDS recommendations for pilot hole sizing. Clearance holes for the unthreaded lag bolt shanks through the deck were drilled 3/4-in. diameter. Pilot and clearance hole drilling were conducted with wood boring bits to minimize wood fiber tear out.

Assemblage of deck-to-girder specimens followed a detailed installation procedure including pilot hole swabbing with creosote to ease connector penetration and a specific torque schedule to mate deck-to-girder assemblies. Torque schedules were developed from a separate deck-to-girder test series to evaluate applied torque versus resultant compressive perpendicular-to-grain load level to establish torque limits with respect to individual glulam material. (Thomforde, 1995). Lag bolt installations were achieved through a devised gear box drive mechanism equipped with torque sensor. Installation procedures with torque limit were followed to control variation between connector specimens. Figures 2-5 show assembled test specimens with respect to deck thicknesses (3.125-in. versus 5.125-in.), lag bolt versus deck clip installation, and also the solid lamination compared to combination 2x4/2x6 lamination glulam girders used for lateral resistance and direct withdrawal evaluations.

Figure 1. Flow chart of the deck-to-girder test connection experimental design.

Deck-to-Girder Test Protocol

Assembled deck-to-girder connections lateral resistance specimens with unique test size geometry excluded following standard ASTM D1761 (ASTM, 1994) test protocol. Specially constructed test apparatus jigs were employed to handle evaluation of the large sized glulam specimens. A single shear loading fixture was developed for the monotonic test evaluations (Figure 6). Apparatus development was based on a similar device (Pellicane, et al, 1984) used to study load slip of nail-jointed connections. This apparatus was attached to an Ametek (60,000 lb capacity) test machine for force application. Load induced displacements were monitored with paired system of linear potentiometers and real time data acquisition with tests terminated after 0.20-in. connection slip. The two-tenths in. end point for testing was well-beyond elastic limit response for the deck-to-girder connections. All monotonic test series were conducted at a 0.010 in/min loading rate (ASTM D1761) and connection specimen assembly immediately prior to test evaluation. Evaluations for both primary and secondary load orientations test series include thirty independent specimen observations. Cyclic loading test evaluations were limited to only five test replications.

Deck-to-girder specimens for exposure to repetitive shear displacements were evaluated in a cyclic test apparatus. The apparatus includes three principal components bottom stationary support frame for restraint of the girder, top slide displacement frame to supply shear action displacement to the deck portion of connection specimen, and motor-operated slider drive mechanism (Figure 7). Machine frame design provided for simultaneous evaluation of five deck-to-girder specimens. Slider drive mechanism with an equipped eccentric cam system permitted control over the magnitude of lateral displacement. To conduct the cyclic test series the cam offset was calibrated based on monotonic data to achieve an approximate NDS connection design load. Load resistance measurements were taken to monitor joint degradation (connection fatigue) as a function of number of cyclic shear displacements (1, 10, l00, 1000, 10,000, 100,000, 300,000, 600,000 and 10^6 cycles). Cyclic loading rate was 6.26 Hertz. This cycling speed closely replicates the natural frequency for a glulam timber bridge (Abendroth, 1989). After exposure to the 10^6 displacement cycles the lateral resistance specimens were tested under monotonic load conditions to identify residual connection performance.

Withdrawal resistance tests to characterize deck-to-girder load-slip under single force application were conducted with the Ametek universal test machine. Two linear potentiometers were again used to measure joint deformation at 1/2 second intervals. Testing force was applied through a spreader bar to the girder main member at equidistant position along the member length on opposite sides of the supported deck. Test evaluations (0.010 in/min loading rate) were conducted to a maximum 0.20-in joint deformation limit as opposed to ultimate load. Some deck-clip joint connections failed catastrophically prior to this 0.20-in deformation limit. After test evaluation withdrawal specimens were examined for connector or wood (compression perpendicular-to-grain) failure and the lateral resistance test series inspected to assess connection yield mode (Johansen, 1949). Lag bolt connections under axial loading were strongly influenced by wood fiber compaction under the lag connector washer. For lateral resistance, dismantled lag bolted 5.125-in. deck-to-girder specimens exhibited predominately yield mode IV failure (e.g. two plastic hinge points per shear plane). Thinner 3.125-in. deck with the 9-in. lag bolt connections tended to be more evenly split between yield modes IV and III, (e.g. bearing-dominated wood fiber yield failure).

EXPERIMENTAL RESULTS

Load-Deformation Data Analysis

European yield model (EYM) is now standard practice for lateral resistance connection design with the NDS (1991 ed). Yield strength is typically taken as a 5% offset load value by offsetting initial slope of the load-deformation curve by a deformation equal to 5% of the bolt diameter (Wilkinson, 1993). Load-deformation response of deck-to-girder connections provided limited linearity, especially for deck clip evaluations. Researchers who have conducted similar evaluations can appreciate the often erratic behavior of connection tests. Instead of the standard 5% offset method, a linear regression approach was utilized to determine 5% offset load for valuation of yield strength. Comparison of regression to EYM 5% offset with analysis of random load response curves showed reasonable agreement between methods. On average primary lateral resistance values agreed within three and two percent for yield load and yield displacement, respectively. Secondary loading orientation showed slightly less agreement with 4% and 12% differences between methods. Test results reported are load at yield, deflection at yield, and elastic slope (stiffness, K1) to qualify connection behavior at the 5% offset value.

Lateral Resistance Results - Monotonic Loading

Tables 1 and 2 collectively provide summary for all the various monotonic test series (Figure 1) for hardwood glulam deck-to-girder connection behavior at estimated EYM 5% offset.

Results (Table 1 and 2) show deck clips consistently provided greater stiffness but not necessarily higher load at yield resistance. Lag bolt connections tend to provide higher load at yield at least in the primary test orientation. Average 5% offset loads for the 9-in. and 12-in. lag bolt connector test series were compared to design performance limit as predicted by yield theory equations (NDS, 1991). Comparisons indicate that experimentally derived yield loads after safety and normal duration adjustment matched well the theoretical NDS design limits within most instances a plus or minus 10% difference. The largest difference (-20.l%) occurred for the combination 2x4/2x6 girder series with secondary test orientation.

Statistical Comparison of Lateral Resistance Results

Testing subsets of the experimental design were paired into four studies cases for statistical comparisons between solid lumber to combination 2x4/2x6 girder type and connector type with respect to the three different glulam wood species. Comparisons (t-tests at $\alpha = 0.05$) to evaluate performance differences are presented (Table 3 and 4) for these study cases with respect to primary and secondary orientation lateral resistance results. Tables 3 and 4 indicate that most test subsets compared (Study Case 1-4) for 5% offset load and elastic stiffness are statistically different at $\alpha = 0.05$ significance level.

Lateral Resistance Results - Cyclic Exposure

Force resistance with induced displacement under cyclic loading for lag bolt and deck clip test series are presented in Table 5.

Values shown (Table 5) represent averaged force resistance measurements normalized to 1.6 NDS connection design load (the 1.6 factor is for adjustment to 10-minute load duration). Normalized values indicate that force resistance of the deck-to-girder connections tended to exceed the predicted NDS design load for four of the six test series.

CM9Z (combination 2x4/2x6 girder with 9-in. lag bolt connection) test series on averge exhibited the greatest departure from predicted NDS lateral resistance. This departure may related to greater edge distance of the installed lag bolt connector compared with narrower solid lamination test girder assemblies. As a trend it is observed that connection resistance declined below NDS design load after exposure betwen 10^3 to 10^5 number of cycles. Table 6 presents the data on cyclic exposure specimens subsequently tested under monotonic load conditions to 0.20-in. deformation limit to evaluate residual yield strength and stiffness (K1) values.

Withdrawal Resistance Results

Descriptive statistics for withdrawal resistance data for the deck-to-girder test series are presented in Table 7. Withdrawal results (Table 7) are interesting specific to the unexpectedly high stiffness for combination 2x4/2x6 girder with 9-in. lag connection (CM9W) yet & lower yield load compared with (M9W) solid girder performance. Some uncertainty existed over this mean test observation. Re-examination of the CM9W data files indicate data reduction analyses may have not completely excluded initial nonlinear settling deformations. Also it was questioned whether a significant size effect (8-in. 2x4/2x6 combination width versus 5.125-in. solid girder width) contributed to observed withdrawal performance. An additional ten CM9W 5.125-in. specimens have since been tested with resultant load-slip properties load at 'yield' (5362 lb), deflection at yield (0.042 in.), and elastic slope K1 (152719 lb/in). These values more closely match the observed trend for the other test series. It is believed that the larger 2x4/2x6 girder contributed significantly to influence initial test results. With these findings new statistical are appropriate before comparisons are made on withdrawal performance.

Summary

To summarize, the experimental results for monotonic loading indicate that both lag bolt and WEYCO deck clip type device with hardwood glulam provide lateral resistance that met with theoretical NDS design limits. The deck clip connection appears to be a viable option for update of BLC-560M. Overall deck clip installations provided higher average joint connection stiffness (K1) for both primary and secondary test orientation. However, primary test orientation data show lag bolt connectors provide better performance in terms of higher 5% offset (load at yield). Observed differences in connector performance data (K1 and load at 'yield') were found in most instances to be statistical significant (α at 0.05). Of special interest the 2x4/2x6 red maple combination girder proved to provide lateral resistance consistent (no real difference $\rho = 0.16$) at least in terms of yield strength with that for the red maple solid lamination type glulam girder. Experimental results under cyclic shear displacement conditions showed lateral resistance decline below NDS design load after exposure between 10^3 and 10^5 cycles. Witmer, 1996 provides detailed analysis to describe this connection degradation or fatigue behavior for hardwood glulam deck-to-girder connections.

Table 1. Summary of 5% offset load-slip properties with respect to primary (z) test orientation.

Test Series	Load at 'Yield' (lbs) Mean	Std. Dev.	COV (%)	Deflection at 'Yield' (in) Mean	Std. Dev.	COV (%)	Elastic Slope (K1) (lbs/in) Mean	Std. Dev.	COV (%)
CM9Z[1]	4,739	447	9.4	0.183	0.061	33.8	27,667	6,739	24.4
M9Z[2]	4,572	453	9.9	0.140	0.036	25.7	34,122	8,103	23.7
M12Z[3]	5,884	744	12.6	0.211	0.073	34.8	32,146	11,815	36.3
P12Z[4]	3,674	388	10.6	0.184	0.045	24.8	22,101	6,022	27.2
R12Z[5]	5,431	418	7.7	0.131	0.036	27.5	46,075	10,746	23.3
M5Z[6]	2,564	921	35.9	0.019	0.022	116	217,270	101,150	46.6
P5Z[7]	2,413	592	24.5	0.021	0.020	97.4	166,203	67,269	40.5
R5Z[8]	2,715	759	28.8	0.021	0.020	94.6	208,358	120,264	57.7

[1]Red maple 3.125" deck with 9-in. lag screw connected to combination (2x4/2x6 lumber) 8-in. wide girder.
[2]Red maple 3.125" deck with 9-in. lag screw connected to solid lamination 5.125-in. wide girder.
[3]Red maple 5.125" deck with 12-in. lag screw connected to solid lamination 5.125-in. wide girder.
[4]Yellow poplar 5.125" deck with 12-in. lag screw connected to solid lamination 5.125-in. wide girder.
[5]Red oak 5.125" deck with 12-in. lag screw connected to solid lamination 5.125-in. wide girder.
[6]Red maple 5.125" deck with WEYCO deck clip (5-in. attaching lag screws) connected to solid lamination 5.125-in. wide girder.
[7]Yellow poplar 5.125" deck with WEYCO deck clip (5-in. attaching lag screws) connected to solid lamination 5.125-in. wide girder.
[8]Red oak 5.125" deck with WEYCO deck clip (5-in. attaching lag screws) connected to solid lamination 5.125-in. wide girder.

Table 2. Summary of 5% offset load-slip properties with respect to secondary (y) test orientation.

Test Series	Load at 'Yield' (lbs) Mean	Std. Dev.	COV (%)	Deflection at 'Yield' (in) Mean	Std. Dev.	COV (%)	Elastic Slope (K1) (lbs/in) Mean	Std. Dev.	COV (%)
CM9Y[1]	4,314	1,199	27.8	0.167	0.084	53.7	29,124	9,512	32.8
M9Y[2]	4,992	779	15.8	0.222	0.073	32.9	24,250	5,676	23.4
M12Y[3]	5,936	645	10.9	0.279	0.104	37.3	23,954	7,863	32.8
P12Y[4]	4,086	313	7.7	0.218	0.041	19.2	20,130	4,817	23.9
R12Y[5]	5,511	507	9.2	0.209	0.052	25.0	41,981	10,648	25.4
M5Y[6]	8,732	1,963	22.5	0.132	0.095	72.2	91,695	46,963	51.2
P5Y[7]	5,326	2,168	40.7	0.062	0.051	85.2	124,400	61,475	49.4
R5Y[8]	5,485	2,125	38.7	0.034	0.016	46.7	151,610	70,879	46.7

[1,2,3,4,5,6,7and 8] See Table 1 for Footnotes

Table 3. Statistical comparisons for four study cases with primary load test orientation results.

Study Case	Test Subsets Compared	Probability (ρ-value) 5% offset load	Probability (ρ-value) elastic stiffness, K1
1	M9Z vs. CM9Z[1]	0.16	0.0018
2	R12Z vs. R5Z[2]	0.0000	0.0000
3	P12Z vs. P5Z[3]	0.0000	0.0000
4	M12Z vs. M5Z[4]	0.0000	0.0000

[1,2,3 and 4] See Table 1 for Footnotes, Z denotes Primary Test Load Orientation

Table 4. Statistical comparisons for four study cases with secondary load test orientation results.

Study Case	Test Subsets Compared	Probability (ρ-value) 5% offset load	Probability (ρ-value) elastic stiffness, K1
1	M9Y vs. CM9Y[1]	0.021	0.0021
2	R12Y vs. R5Y[2]	0.95	0.0000
3	P12Y vs. P5Y[3]	0.031	0.0000
4	M12Y vs. M5Y[4]	0.0000	0.0000

[1,2,3 and 4] See Table 1 for Footnotes, Y denotes Secondary Test Load Orientation

Table 5. Force resistance under cyclic loading conditions normalized to NDS design loads.

Test Series	Log (Cycles) 10^0	10^1	10^2	10^3	10^4	10^5	$3x10^5$	$6x10^5$	10^6
M9Z[1]	1.26	1.18	1.18	0.90	0.88	0.70	0.71	0.69	0.69
M12Z[2]	1.18	1.09	1.01	1.03	0.96	0.80	0.58	0.79	0.69
CM9M[3]	1.65	1.57	1.18	1.09	1.08	0.98	0.88	0.92	0.81
M9Y[1]	0.89	0.86	0.86	0.81	0.68	0.50	0.29	0.34	0.24
M12Y[2]	1.14	1.07	1.02	0.76	0.72	0.52	0.48	0.45	0.43
CM9Y[3]	0.89	0.85	0.77	0.84	0.63	0.42	0.29	0.25	0.27

[1,2and 3] See Table 1 for Footnotes, Z and Y denote primary versus secondary test orientations

Table 6. Summary of residual 5% offset load-slip properties of cyclic test specimens

Test Series	Load at 'Yield' (lbs) Mean	Std. Dev.	COV (%)	Deflection at 'Yield' (in) Mean	Std. Dev.	COV (%)	Elastic Slope, K1 (lbs/in) Mean	Std. Dev.	COV (%)
M9Z[1]	3963	771	19.4	0.075	0.018	23.8	60839	20271	33.3
CM9Z[2]	3728	689	18.5	0.072	0.020	27.8	57620	19268	33.4
M12Z[3]	5359	1645	30.7	0.096	0.051	53.0	67444	33421	49.6
M9Y[1]	4235	946	22.3	0.103	0.031	30.4	44303	9162	20.7
CM9Y[2]	4012	273	6.8	0.168	0.092	54.5	28954	11867	41.0
M12Y[3]	3559	609	17.1	0.130	0.077	59.2	37987	19806	52.1

[1,2 and 3] See Table 1 for Footnotes, Z and Y denote primary versus secondary test orientations

Table 7. Summary of withdrawal 5% offset load-slip properties under monotonic test conditions.

Test Series	Load at 'Yield' (lbs) Mean	Std. Dev.	COV (%)	Deflection at 'Yield' (in) Mean	Std. Dev.	COV (%)	Elastic Slope (K1) (lbs/in) Mean	Std. Dev.	COV (%)
CM9W[1]	3,925	1,030	26.2	0.007	0.004	58.7	641,681	239,689	37.4
M9W[2]	5,300	893	16.9	0.025	0.015	58.7	184,986	73,558	39.8
M12W[3]	6,260	1,017	16.2	0.016	0.009	52.4	259,736	108,776	41.9
P12W[4]	4,074	788	19.4	0.017	0.008	47.7	230,218	60,447	26.3
R12W[5]	8,615	1,031	12.0	0.017	0.005	27.4	445,923	106,862	24.0
M5W[6]	3,535	1,104	31.2	0.013	0.006	44.9	309,877	103,438	33.4
P5W[7]	3,387	526	15.5	0.017	0.005	33.3	223,436	83,949	37.6
R5W[8]	3,646	1,105	30.3	0.018	0.013	70.9	291,238	183,639	63.3

[1,2,3,4,5,6,7, and 8] See Table 1 for Footnotes, W denotes direct withdrawal testing

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Figure 2. Lag bolt (9-in.) connection for 3.125-in. thick deck with solid lamination girder (5.125-in. width).

Figure 3. Lag bolt (12-in.) connection for 3.125-in. thick deck with combination 2x4/2x6 lamination girder (8-in. width).

Figure 4. Lag bolt (12-in.) connection for 5.125-in. thick deck with solid lamination girder (5.125-in. width).

Figure 6. Lateral resistance test apparatus with open view to illustrate clamped deck clip test assembly.

Figure 5. Deck clip (5-in. attaching lag bolts) connection for 5.125-in. thick deck with solid lamination girder (5.125-in. width).

Figure 7. Cyclic loading frame machine to conduct repetitive shear displacement on assembled deck-to-girder specimens.

A BUILT-UP TIMBER GIRDER FOR BRIDGE CONSTRUCTION IN DEVELOPING COUNTRIES

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Abstract

Just as it is in developed countries, most developing countries utilize concrete and steel in their various forms of reinforced concrete, prestressed concrete and steel, for the construction of bridges. Now, most of these countries do not produce cement and steel locally and have to rely on their importation. As a result, a large proportion of their hard earned foreign currency is utilized for the importation and construction of these bridges. Paradoxically, many of these countries abound in timber as a natural resource which could be utilized for the purpose. The authors have identified the bridge girder as the main constraint in the use of wood for medium span bridges and have developed a built-up timber girder to solve the problem. The paper gives the highlights of the girder for the project, discusses why the girder is appropriate to developing countries with timber as a natural resource and outlines important factors that need to be considered in the design and construction of such a girder.

Keywords: Built-up, timber girder, bridge construction, developing countries, web laminations and flanges.

Introduction

In Ghana, like in many developing countries, cement and steel in their various forms of reinforced concrete, prestressed concrete and steel members, are utilized for bridge construction, even though these materials are to a large extent imported into the country. The cost to the nation for these importations is very great, when one considers the nation's foreign exchange earnings.

Ghana, however, abounds in various species of timber with a broad spectrum of properties, many of which are excellent for bridge construction. It was therefore necessary to consider timber as a substitute for the imported materials, for bridge construction.

In examining why wood had not been considered before this time, reasons identified are:

i. wood easily decays in the tropics in its natural state due to fungal and insect attack. People are prejudiced against the use of it for permanent bridge construction because of this defect.
ii. sawn timber is limited in its length and height and therefore limits bridge spans that wood may be utilized for.
iii. many wooden bridges built previously were built by a rule of thumb and were ugly and served as a short term solution to crossings.
iv. wood is combustible and this makes it vulnerable to bush fire outbreaks during the dry season.

To overcome these disabilities of wood for bridge construction:

i. use is made of naturally durable timber which is further chemically treated using copper chromium arsenate.
ii. a new mechanically built-up girder capable of being designed for a long span bridge has been developed by the authors.
iii. the bridge is designed to be aesthetically pleasing.
iv. by creating firebelts around the bridge using stonework the bridge may be preserved against bush fires.

Highlights of the Girder

The girder consists of two side flanges at the upper and lower ends. In some circumstances, top and bottom flanges may be included (Figure 1). These flanges are mechanically held to web laminations through the use of bolts. The web laminations consist of two layers with the laminations of one layer being diagonal to the other layer's laminations. Theoretically, a bolt is placed at every center of intersection of two diagonal laminations. Timber plates are placed at the extreme ends of the girder to act as:

i. bearing stiffeners.
ii. to connect web laminations which terminate at girder ends.

The advantages of using such a girder for a developing country are many.

i. Long span girders may be easily built using the system of connection as shown in Figure 2. Note that, in a long span bridge only flanges need to be lapped.
ii. It is possible to fabricate such a girder in a workshop under strict supervision, and later transport it to the bridge site to be assembled in position.
iii. Only simple tools are required and hence avoids the use of expensive machines.
iv. It is labor intensive; which is advantageous to a country with a high unemployed labor force.
v. It does not require very highly skilled artisans. Any artisan with experience in handling electric saws, planes and drills, is capable of handling the project.
vi. Bolts utilized for fabrication are easily manufactured from concrete steel reinforcing bars. The durability of the bolt maybe improved through the process of "blueing", in which the bolts are heated to about 400°F and suddenly quenched in residual engine oil.
vii. It avoids the use of steel gusset plates which may be expensive in these countries. This is an added advantage over a timber lattice girder or truss girder.

Figure 1 — Mechanically built-up timber girder for the project

Figure 2 – Moment connection between two girder parts

Geometrical Arrangement of Web Laminations

Geometrical Relationships

For proper fabrication of the girder, it is imperative to arrange the layer laminations in such a way that, the centerline of a lamination in one layer, crosses the centerline of another lamination in the second layer, along a given line within flanges as shown in Figure 3. This line may be referred to as the bolt line.

With this arrangement, if *d is* the distance between the upper and lower bolt lines, is the angle of inclination of web laminations with the axis of the girder and *w* is the width of each web lamination, then, there is a relationship between *d,* and *w,* given as

$$\frac{n}{\cos\mu} = \frac{d}{(w/2)} \qquad (1)$$

where *n* is an integral number, giving the number of web laminations lying within the two laminations discussed above. This is easily verified from the diagram and implies that once any three of the above variables are chosen, the fourth variable is fixed.

In design, one may assume *d* and *w;* then *n/cos* is known. It may also be desirable to have close to a given angle say 30°. The value of *n* may then be computed which would most likely be a non-integer. An integer number above or below the value obtained for *n* may then be assumed and reevaluated, the value of which will be close to 30°.

Common Point of Origin to Draw Web Laminations

Referring to Figure 4, if O is the geometrical center of the girder, and the centerline of lamination 1 is drawn through O, then the position of lamination 2 centerline is fixed. It easy to see that if *n* is an even integer, then one other lamination, (3), parallel to (2), will pass through O, since *n/2* is an integer. Hence, to start drawing laminations, one may start from O.

However, if *n* is an odd integer, then *n/2* is a non-integer and a common point O from which the other lamination may be drawn moves from O along lamination 1 to a point O such that the vertical distance of O' from O, h_{vert}, is given by the relationship (Figure 4).

$$h_{vert} = \frac{w}{4\cos\mu} \qquad (2)$$

Adherence to these geometrical equations will produce a mirror image arrangement of web laminations for the two layers.

Figure 3 – Geometrical relationships in girder

Figure 4 – Determination of common point where two laminations in different layers can start from

Girder Structural Behavior

Theoretical Analysis

A theoretical solution may be obtained by considering every bolt position in the beam as a joint and the material between bolts as a beam member. In this way finite element solution may be applied to the beam to obtain strains and beam stresses in the beam.

Such an analysis has two defects. These are:

i. the process is wieldy and does not lend itself easily to office design.

ii. due to the number of bolts, friction forces arise between members and produces results different from what the above may predict.

There is therefore the need to derive equations for the structure based on the laboratory observations on the beam behavior. The following assumptions are made with respect to the beam behavior.

i. The flanges are wide so that there is strain variation across its width.

ii. The web lamination is narrow and strain across its width may be assumed as uniform (i.e., tensile or compressive).

iii Due to the friction between laminations in the two layers, it is assumed that a mechanical joint occurs at each point of intersection of any two diagonal laminations.

iv. Plane sections in the girder before bending, remain plane after bending.

Based upon these assumptions, and the displacement geometry in Figure 5, the general strain-displacement equation for the web lamination may be derived as

$$\varepsilon_w = \frac{\partial u}{\partial x}\cos^2\mu \pm \frac{\partial v}{\partial x}\sin\mu\cos\mu \quad (3)$$

where $_w$ is the strain in web and u, v are horizontal and vertical displacements. This displacement equation was first derived by Ghaboussi in reference one.

Stresses Due to Moment – Using equation 3 and assumption {iv} above, the effective moment of inertia of the section may be derived as

Figure 5 – Displacement geometry in girder

$$I_e = mI_w\cos^4\mu + I_f \quad (4)$$

where I_w is the moment of inertia of the web assuming it is continuous. I_f is the moment of inertia of the flange and m is the ratio of the moduli of elasticity i.e.,

$$m = \frac{E_w}{E_f} \quad (5)$$

The resulting stresses in the flange $_{f,m}$ and the web $_{w,m}$ due to moment M are given as:

$$\sigma_{f,m} = \frac{My}{I_e} \quad (6)$$

$$\sigma_{w,m} = \frac{My}{I_e} m \cos^2 \mu \quad (7)$$

where y is the vertical distance from the neutral axis

Stresses Due to Shear – Just as in other beams, the shear is related to the moment as

$$Q = \frac{\partial M}{\partial x} \quad (8)$$

Using equation 8, it may be shown that

$$\sigma_{w,q} = \pm \frac{2Q}{bwI_e \sin 2\mu} [(A\bar{y})_f + m \cos^4 \mu (A\bar{y})_w] \quad (9)$$

where the same timber is used for flange and web, m= 1 and equation (9) reduces to

$$\sigma_{w,q} = \pm \frac{2Q}{bwI_e \sin 2\mu} [(A\bar{y})_f + \cos^4 \mu (A\bar{y})_w] \quad (10)$$

where $(A\bar{y})_w$ and $(A\bar{y})_f$ are the static moment of area of web and flange respectively about the neutral axis in the girder cross-section.

Similarly, the shear stress in the flange is given as

$$\tau_{f,q} = \frac{Q(A\bar{y})}{I_e \; b_f} \quad (11)$$

Defections – The total deflection of the girder consists of:

i. flexural deflection.
ii. shear deflection.
iii. creep deflection.
iv. slip deflection.

Flexural Deflection — This may be computed using the usual beam deflection formulae for various load conditions and types of support. The flexural rigidity to use is the $E_f I_e$ where E_f *is* the young's modulus of flange timber and I_e is the effective moment of inertia of the girder cross-section.

Shear Deflection — Unlike steel and concrete girders in which their depths tend to be shallow, practical dimensions of the girders are such that they tend to be deep. This means shear deflections cannot be neglected.

To compute the shear deflections, the usual formulae for beams subject to various loading and support conditions may be used. The effective shear rigidity $(GA)_e$ may be computed using the formula

$$(GA)_e = \frac{E_f \; I_e^2}{m \sum_{k=1}^{4} f_k} \quad (12)$$

where m is as given in equation 5. For the same timber for troth the web and the flange, m = 1, thus

$$(GA)_e = \frac{E_f \; I_e^2}{\sum_{k=1}^{4} f_k} \quad (13)$$

where f_k is a function of beam cross-section geometry only. The function f_k is given in references two and three.

Creep Deflection — Creep deflection in any material is a function of stress level and time. Timber tends to creep and research is needed to determine the ultimate amount of creep deflection to be expected for the timber girder. From preliminary survey, doubling the flexural deflection for dead loads seems to give adequate results for the creep deflection.

Slip Deflection — An upper limit for slip deflection may be obtained by knowing the maximum tolerance permitted for the size of bolt hole and computing the product

$$\delta = \sum U \partial l \quad (14)$$

where U *is* the resulting force in the member with the bolt due to a unit load in the direction in which deflection is required and ∂l is the displacement at each bolt, i.e., the tolerance.

Conclusions

1. It has been shown that for a developing country which abounds with wood, and has to import cement and steel, the utilization of wood for bridges is cost effective.
2. The wooden girder chosen for the job, has been developed by the authors and has the advantages enumerated above, over other timber girders for a developing country.
3. By making use of the geometrical relationship developed in the text, the girder can be correctly set out.
4. The use of the structural equations as stated above, enables the structural proportioning for moment, shear and deflection to be effected.
5. Tests conducted at the Building & Road Research Institute, Ghana, reference three and Virginia Polytechnic Institute and State University, Blacksburg, Virginia, have shown that within the elastic range, equations outlined in this paper are adequate in predicting the behavior of the girder under load.

6. A prototype bridge designed using these equations is shown in Figure 6.

Figure 6 — A demonstration bridge showing girder and articulated truck on bridge

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ii. the structures laboratory of the Civil Engineering Dept., Virginia Tech.; where the girder models were tested.
iii. the Brookes Center of the Dept. of Wood Science and Forest Products; where the models were built.

PORTABLE SURFACES FOR CROSSING UNSTABLE ROADBEDS

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Abstract

Low-volume roadbeds surfaced with aggregate and native soil are a vital part of many U.S. Department of Agriculture Forest Service road networks. A common concern of low-volume road designers is the development of ruts through short, unstable sections. Ruts reduce vehicle access, affect local streams and hydrology, increase maintenance, and accelerate the loss of surfacing material as a result of erosion. The Forest Service has investigated portable surfaces as an alternative to continuous maintenance, use of crushed aggregate, or reconstruction. This report discusses field evaluation to quantify the reduction of rut depth for two types of wood portable surfaces. Wood pallets and wood mats were installed on native soil timber harvest roads in northcentral Florida. On average, the surfaces reduced rut depth by 127 mm (5 in.).

Keywords: Road, portable, surface, harvesting, rut

Introduction

Most low-volume roads maintained by the Forest Service consist of roadbeds surfaced with aggregate or native soil and designed according to established engineering design principles. Often, the roads are designed to provide short-term access for routine maintenance. A concern of designers and managers is the short sections of unstable surface or subsurface material, which typically are due to high moisture content. This unstable material leads to the development of ruts (Fig. 1). Continued traffic deepens the ruts until they are bladed or crushed aggregate surfacing material is added to the unstable section.

Limiting the development of ruts is important for several reasons. As ruts develop, surface or intercepted subsurface water is diverted from the designed road drainage, which may alter local hydrology, especially in very flat topographical areas. Water can run in the ruts, loosening and transporting soil particles, which erodes the roadbed within the ruts. Subsequent erosion may destroy sections of the road. Also, water with suspended soil particles may eventually drain into local streams, potentially increasing the stream turbidity. Deep ruts can damage or incapacitate vehicles.

The Forest Service recognized the need for portable surfaces to inhibit the development of ruts. The surfaces were visualized as a means of crossing short sections cf unstable soil as an alternative to continuous maintenance, the use of crushed aggregate, or reconstruction. The San Dimas Technology and Development Center performed a market search and published a report describing 12 portable surfaces (Mason 1990). Six of these surfaces were chosen for field evaluation based on the following criteria: portability, recyclability, purchase price, and availability. These surfaces were evaluated on the Osceola Ranger District in northcentral Florida under normal low-volume forest road use. All were qualitatively judged to be successful at visibly reducing rut depth and increasing the duration of vehicle access.

Figure 1—Rut development on low volume road in Florida.

Figure 2—Schematic of prefabricated wood pallet (1 ft = 0.3048 m).

Field evaluations were continued to quantify the reduction in rut depth (Hislop 1996). In the work reported here, we discuss two wood portable surfaces that were evaluated on existing native soil roadbeds in northcentral Florida. The evaluations were performed in cooperation with the Rayonier Corporation and the Osceola National Forest. The majority of the vehicles driven over the sections during the field evaluation period were five-axle loaded and unloaded log trucks with a gross vehicle weight of 11,200 N (80,000 lb).

Description of and Experience With Portable Surfaces

Some important considerations common to portable surfaces are as follow:

- surface weight, when considering installation and removal equipment
- blading of existing ruts, for reducing bending stress
- use of geotextile, for confining soil beneath the portable surface and for separating the soil and portable surface (separation facilitates removal of the surface by reducing tension-related stress)
- coverage of entire section, to prevent ruts at surface ends
- dividing and marking of section prior to installation, to avoid unnecessary moving of portable surface

Wood Pallets

Private companies are producing a much sturdier variation of wood pallets typically used for shipping. One type of new pallet is made of 76.3- by 203.2-mm (3- by 8-in.) timber planks, nailed together in three plies. The pallets range from 2.4 by 3.6 m to 2.4 by 4.8 m (8 by 12 ft to 8 by 16 ft). They are reversible, and broken planks can be replaced easily, prolonging the life of the pallet. Each end has an overlap area for connection between pallets to make roads and platforms (Fig. 2).

One disadvantage is pallet width. Pallets are connected along the 2.4-m (8-ft) edge, which is too narrow for log truck roads. The pallet can be cut into 1.2- by 4.3-m (4- by 14-ft) half-pallets. Each half-pallet is placed in a wheel path, providing the necessary road width. Half-pallets weigh less (154 N (1,100 lb)) than full pallets and are less cumbersome during installation. Another disadvantage of wood pallets is that forklifts are the easiest means for installation and removal but they are not typical equipment on forest harvesting road construction sites. A front-end loader or backhoe and lifting chains are common construction equipment, but the planks are too close together to use chains. An option is a thin choker cable that can be run between the planks and hooked to the lifting chains.

Wood Mats

Wood mats are similar to dragline mats typically used by tracked vehicles. The mats are constructed of wood posts with nominal dimensions of 101.6 by 101.6 mm (4 by 4 in.) and 152.4 by 152.4 mm (6 by 6 in.), at least 3 m (10 ft) long. Holes are drilled through each post 0.6 m (2 ft) from each end. The posts are connected together by threading 4.8-mm (3/16-in.) galvanized steel cable through each set of holes over the full length of the mat. Loops are made at the end of each cable using cable clamps. The loops extend beyond tbe last post at each end of the mat and are used to pick up the mat during installation and removal (Fig. 3).

The construction of the mat is the most time-consuming aspect of the operation. Three people need up to 3 hours to cut, drill, and cable together a 6.1-m- (20-ft-) long, 3-m- (10-ft-) wide mat. One set of drilling location marks should be made on the ground for all

Figure 3—Wood mat installation.

the posts to prevent errors in marking each post. Time can be saved if several drills are available. A welding torch or some other means of controlling cable-end fraying increases threading speed. Care must be taken in using the cable clamps; incorrect use can lead to slippage. It is important to make sure that all cable loops are placed under the mats after installation. Otherwise, they catch on the underside of vehicles. The 101.6-mm- (4-in.-) thick mats are considered the most critical in terms of bending strength and have been proven to work well. These mats are lower in cost and lighter in weight than the 152.4- (6-in.) mats.

Geotextile

There is much literature on the use of geotextiles, and many types of geotextiles are available. The main type used for field evaluations is a nonwoven, needle-punched filter fabric. This geotextile is inexpensive and was readily available where the field evaluations were performed. In general, the geotextile should meet the following requirements: (1) be capable of retaining soil underneath while allowing water to pass through, (2) have a rough surface to limit movement of the portable surfaces, and (3) be removable after use.

Installation of a geotextile is recommended to limit penetration of portable surfaces into the soil. Moreover, the time required to remove a portable surface is often reduced by adding a geotextile. Although the geotextile may be in good condition after use, additional weight from soil and water may make the fabric too heavy to be removed, Some concerns have been raised about leaving the geotextile in place after use. Ideally, to form a visually acceptable solution and limit continued access, the geotextile should be removed. If the geotextile is cut in short lengths, it may be removable and reusable although the length at which a geotextile is no longer effective is unknown.

Table 1—Stabilization alternatives.

Surface type	Weight (N (lb))	Cost[a] ($/m^2)
Wood half-pallet	176.4 (1,260)	46.92
Wood mat		
101.6 mm (4 in.) post	288.4 (2,060)	25.93
152.4 mm (6 in.) post	393.4 (2,810)	34.57
Limestone aggregate	3,920 (28,000)	12.35

[a]1 ft^2 = 9.29 × 10^{-2} m^2.

Comparison of Portable Surfaces

Table 1 summarizes information on different types of portable surfaces as well as crushed limestone aggregate, which is typically used to stabilize roadbeds in northcentral Florida. The cost data include labor to construct the surface and installation and removal costs. Geotextile is included in the cost of each portable surface. Costs are based on the surface necessary to cross an unstable roadbed section 9.1 m (30 ft) in length on a straight section of a single-lane road. Such a surface would require two 2.4-by 4.8-m (8- by 16-ft) pallets cut in half, with one 1.2-m (4-ft) half-pallet placed in each wheel path; two 3- by 4.6-m (10- by 15-ft) wood mats; or crushed limestone aggregate covering 3 by 9.1 m (10 by 30 ft), 203.2 mm (8 in.) deep. Information on service life is too limited to determine lifecycle costs. Howewr, unlike portable surfaces, crushed limeestone aggregate cannot be removed and reused. Also, several applications of the aggregate may be necessary, depending on soil conditions and amount of traffic.

Site Description and Data Collection Methods

The previous study by the San Dimas Technology and Development Center concluded that portable surfaces visibly reduce rut depth. The objective of our field evaluation was to measure this reduction in rut depth. The evaluation consisted of determining soil characteristics and collecting rut depth data for control and portable surface sections. The general site preparation consisted of the following:

- locating short, straight road sections where continuous maintenance was being performed,
- blading existing ruts in the chosen control and portable surface sections,
- placing the geotextile and portable surfaces on the roadbed,
- giving access to traffic, and
- installing stringline stakes and initiating daily data collection until the end of traffic use or field evaluation.

Figure 4—Portable surface section.

Figure 5—Control section.

Figure 6—Schematic of site layout and cross-section locations.

Site Description

The road section stabilized by the portable surfaces was near a culvert installation where road drainage tended to collect. Figure 4 shows the 2.4-m- (8-ft)- long wood mats made of 101.6-mm (4-in.) posts as ramps on either side of the 4.3-m- (14-ft-) long mats made of 152.4- (6-in.) posts. The wood pallets butted up to the mats. Geotextile was placed under the portable surface. The control section was located on the same road approximately 54.9-m (180-ft) from the portable surfaces (Fig. 5). The control had similar moisture content and shading characteristics as that of the portable surface. The log truck traffic volume was approximately 30 round trips/day.

Soil Characterization and Rut Depth

The soil under the control and portable surfaces was characterized to determine similarities within each site. Provided the sections were similar, differences in rut depth would be attributable to the portable surfaces. Figure 6 shows the portable surface layout at the test site. The circles show stake locations where cross-section profiles were measured for rut depth. Soil characterization data point locations are not shown because (1) they varied each day as a result of disturbance from the previous day's tests and (2) we assumed that each entire section was similar. Also, the data were gathered each day at one location for the control and at three locations for the portable surfaces. Rut depth data were collected along the same cross-section each day.

Soil samples were taken at a depth of approximately 50.8 mm (2 in.) to determine moisture content as the main indicator of soil instability. The samples were placed in resealable plastic bags. Moisture content was determined using a moisture content machine, which runs on the same principles as does the oven drying technique. Cone penetrometer data were gathered to characterize soil strength. Figure 7 shows the Irregular Cone Index penetrometer. The 25.4-mm (1-in.) marks along the staff are the points of gage reading. The penetrometer was manually driven into the soil to a depth of 152.4 mm (6 in.) or until the maximum load was reached on the gage. The cone penetrometer readings were initially taken once at each data point location each day. Later, readings were verified by taking two to three readings for each data point location.

Rut depth was determined from road cross-section profiles using a stringline method. Stakes were permanently placed along the road edges. The stakes were marked at 0.3 m (1 ft) above the soil surface for placement of the stringline (Fig. 8). Road cross-sections were drawn by measuring transversely along the stringline to the highest and lowest ground elevations outlining the ruts. Elevations were measured from the stringline. The points representing the highest and lowest elevations were determined visually.

Figure 7—Cone penetrometer.

Figure 8—Stringline method used to measure rut depth.

Analysis of Results

Soil strength and moisture content of the control and portable surfaces were analyzed to verify that the control was representative and a viable baseline for the site. Cone penetrometer readings were converted to soil strength (California bearing ratio) using conversion factors supplied with the equipment. The other variables—traffic, climate, construction, and maintenance—were the same for the control and surface sections. The soil was determined to be a silty sand.

Practical constraints limited data collection. Because the portable surfaces could not be moved during use, all soil characterization data, except for the data for the final day, were gathered from the side of the surfaces. For the control section, all soil characterization data, except the data for the first day, were gathered from the wheel paths. Therefore, only the first day's data from outside the wheel paths and the final day's data from inside the wheel paths were compared for soil strength. Because of traffic constrains, no data were taken prior to roadbed use by traffic. Traffic was stopped in order to blade the sections and place the portable surfaces and was then allowed to continue while initial measurements were taken. Thus, we thought it reasonable to assume that both the control and portable surface sections had no initial ruts and that the first day's data represented the entire section.

Soil Characteristics

The California bearing ratio (CBR) is an index of soil strength in regard to shear failure under load. Because the change in CBR is not linear, a doubling of the CBR value does not equate to a doubling of soil strength (Barksdale 1991). The relationship of CBR to soil strength is a logarithmic curve. Thus, for smaller CBR values, changes in soil strength are large for incremental changes in CBR; strength rapidly decreases at higher CBR values. The following comparisons of the measured CBR are based on a soil strength chart supplied with the cone penetrometer equipment.

Figure 9 shows the CBR values for the first day. These data are assumed to be representative of soil conditions prior to placement of the portable surfaes and use by traffic. The portable surface data are averages, Moisture content was 2 percent greater in the portable surface section.

From a depth of 0 to 50.8 mm (2 in.), the CBR values of the portable surface section were slightly greater than that of the control but less then 3.2. At these CBR values, a shear failure through the 50.8-mm (2-in.) depth would be expected within two passes of a log truck. The CBR of the surface section was less than that of the control by up to 0.5 at depths of 76.2 to 101.6 mm (3 to 4 in.). Because the values are so close, little difference in shear failure would be expected to a depth of 101.6 mm (4 in.). At 127 mm (5 in.), the CBR value was 1.3 greater in the control section, which would result in a difference in rut depth for a small number of vehicle passes.

Figure 9—Comparison of California bearing ratio (CBR) to rut depth prior to traffic.

Figure 10—Comparison of CBR to rut depth at end of evaluation.

Figure 10 shows the CBR values for the final day. There was no difference in moisture content between the sections. Cone penetrometer data were averaged trom four values, two taken within each wheel path. The CBR of the portable surface section was greater by up to 0.8 at a depth of 76.2 mm (3 in.) and 127 mm (5 in.). Because the CBR values are so close, little difference in rut depth would be expected. At a depth of 101.6 mm (4 in.), the CBR of the portable surface section was greater by 3, which would result in a difference in rut depth for a small number of vehicle passes.

Based on these data, the control section is representative of the portable surface section. Given the total number of log truck passes and the higher moisture content in the surface section during the test period, the rut depth in the control section is expected to be a conservative estimate of the rut depth expected in the surface section. The following rut depth analysis determines the reduction in rut depth caused by using portable surfaces.

Figure 11—Determination of rut depth.

Rut Depth

Rut depth was interpreted as the difference between the low point within a wheel path and the average of the high points on either side of a wheel path (Fig. 11). Control and portable surface sections were compared for rut depth differences within the same wheel path.

Interpretation of rut depth for the portable surface sections proved difficult. The wood mat surfaces were uneven as a result of tracked dirt and natural warp of the wood. With the wood pallets, the difficulty in interpretation was due to plank breakage. It was difficult to determine if the changes in depth were due to broken planks or compaction of the pallet into the road surface. Measurements taken along the edges of the portable surfaces at the completion of the field evaluation indicated that the portable surfaces were embedded in the roadbed surface approximately 38.1 mm (1-1/2 in.). Using this embedment value as the baseline, the rut depths were determined for each day. Figures 12 to 13 show the differences in rut depth for each wheel path. In the north wheel path, the difference varied from 134.6 to 154.9 mm (5.3 to 6.1 in.); in the south wheel path, from 66 to 101.6 mm (2.6 to 4 in.).

Conclusions

This report describes a portion of a field evaluation of portable surfaces used to cross short sections of unstable roadbed in northcentral Florida. The evaluation quantified the effectiveness of the portable surfaces in reducing rut depth from log truck traffic. Moisture content and cone penetrometer data were gathered to determine soil characteristics of portable surface and control sections. Cross-section profiles were measured to compare rut depths in portable surface and control sections.

Comparison of data taken on the first and final days verified that the control section conservatively represented the surface section. Moisture content averaged slightly higher in the portable surface

Figure 12—Rut depth in north wheel path over time.

Figure 13—Rut depth in south wheel path over time.

sections. In general, differences in CBR values were minimal, and for the total number of vehicle passes, rut depths would be similar.

The cross-section profiles indicated that the portable surfaces reduced rut depth by an average of 127.0 mm (5.0 in.). The main cause of ruts appears to be localized bearing capacity failure. The top 25.4- to 50.8-mm (l- to 2-in.) of roadbed soil become saturated and loosened by traffic and move laterally from under the loads. The portable surfaces distribute the load over a larger area and the geotextile retains the soil. This inhibits the lateral movement of the soil, which reduces rut depth.

Portable surfaces are a temporary, reusable alternative for crossing unstable low-volume roadbeds surfaced with aggregate or native soil, under specific conditions. For the field evaluation reported here, those conditions were short sections prone to continual rut development in silty sand soil roadbeds that provide access for log trucks. Portable surfaces are inexpensive, readily available products for road designers to consider as an alternative to reconstruction, frequent road maintenance, or use of nonreusable crushed aggregate. The most effective portable surface depends on the equipment and funding available, and the initial and allowable final site conditions.

For manual installation, wood mats can be assembled and disassembled on site. For limited funding, wood mats made of 101.6- by 101.6-mm (4- by 4-in.) posts would be least costly. Wood mats should be used for initial sites that are not flat, such as rolling dips. For final site constraints of few or no ruts, wood mats or wood pallets leave no noticeable ruts. Wood mats are recommended as the best overall portable surface for crossing short sections of unstable roadbed soil.

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Railing Systems for Longitudinal Timber Deck Bridges

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Abstract

Bridge railing systems in the United States have historically been designed based on static load criteria given in the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges*. In the past decade, full-scale vehicle crash testing has been recognized as a more appropriate and reliable method of evaluating bridge railing acceptability. In 1989, AASHTO published the *Guide Specifications for Bridge Railings* which gives recommendations and procedures to evaluate railings by full-scale vehicle crash testing. In 1993, the National Cooperative Highway Research Program (NCHRP) published *Report 350, Recommended Procedures for the Safety Performance Evaluation of Highway Features*, which provides criteria for evaluating longitudinal barriers. Based on these specifications, a cooperative research program was initiated between the University of Nebraska-Lincoln and the Forest Products Laboratory, and later the Federal Highway Administration, to develop and crash test several bridge railings for longitudinal wood decks. This paper describes the successful development and testing of nine resulting railing systems in accordance with the AASHTO Performance Level 1 and 2 (PL-1 and PL-2) requirements, and the Test Level 1 and 4 (TL-1 and TL-4) requirements of *NCHRP Report 350.*

Keywords: Timber Bridges, Bridge Rail, Crash Testing, Roadside Safety, and Longitudinal Barrier.

Introduction

The primary purpose of a bridge railing is to safely contain errant vehicles crossing a bridge. To meet this objective, railings must be designed to withstand the force of an impacting vehicle. In designing railing systems for highway bridges, engineers have traditionally assumed that vehicle impact forces can be approximated by equivalent static loads that are applied to railing elements. Although rail loads are actually dynamic, the equivalent static load method has been used for many years as a simplified approach to standardized railing design. Until recently, the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (1) required that rail posts be designed to resist an outward transverse static load of 44.5 kN (10,000 lb). A portion of this load was also applied to posts in the inward transverse, longitudinal, and vertical directions and to the rail elements. These requirements were identical for all bridges regardless of bridge geometry or traffic conditions. Thus, a railing for a single-lane bridge located on a low-volume road was required to meet the same loading requirements as a railing for a bridge located on a major highway.

Despite the widespread use of design requirements based primarily on static load criteria, the need for more appropriate full-scale vehicle crash test criteria has long been recognized. The frost U.S. guidelines for full-scale vehicle crash testing were published in 1962 (2). This initial l-page document provided basic guidelines for the test vehicle mass, approach speed, and impact angle and served to provide a degree of uniformity to the traffic barrier research in progress at the time. Through subsequent use of this document, the need for more comprehensive guidelines became apparent, and several reports were published during the 1970s through the National Cooperative Highway Research Program (NCHRP). In 1981, NCHRP published *Report 230, Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances* (3). This comprehensive report provided recommendations relative to crash testing and evaluation of longitudinal barriers and served as the basis for future bridge rail crash testing requirements.

Although crash test criteria have been available for many years, the requirement to implement crash testing as a means of evaluating bridge railings in the United States has been jurisdiction dependent. Some states implemented extensive bridge rail crash testing programs, while others continued to exclusively use static load design. The first recognition of full-scale crash testing in a national bridge specification came in 1989 when AASHTO published *Guide Specifications for Bridge Railings* (4). This specification presents recommendations for the development, testing, and use of crash-tested bridge railings and refers extensively to *NCHRP 230* for crash testing procedures and requirements.

A primary concept of the *AASHTO Guide Specifications* is that bridge railing performance needs differ greatly from site to site, and railing designs and costs should match site needs. Thus, recommended requirements for rail testing are based on three performance levels: Performance Level 1 (PL-1), Performance Level 2 (PL-2), and Performance Level 3 (PL-3). The PL-1 requirements represent the "weakest" system, and the PL-3 the "strongest" system. The relationship between the railing performance level and requirements for a specific bridge depend on a number of factors, such as the type of roadway, design speed, average daily traffic, and percentage of trucks in the traffic mix.

The recently published *NCHRP Report 350, Recommended Procedure for the Safety Performance Evaluation of Highway Features* (5), provides for six test levels for evaluating longitudinal barriers; Test Level 1 (TL-1) through Test Level 6 (TL-6). Although this document does not include objective criteria for relating a Test Level to a specific roadway type, the lower test levels are generally intended for use on lower service level roadways and certain types of work zones while the higher test levels are intended for use on higher service level roadways.

In 1994, AASHTO published the *LRFD Bridge Design Specifications* (6) as an update to the *Standard Specifications for Highway Bridges* (1) and the *Guide Specifications for Bridge Railings* (4). For crash testing bridge railings, three performance levels were provided similar to those provided in the *Guide Specifications for Bridge Railings* (4). Guidelines for crash testing bridge railings followed procedures provided in both the *AASHTO Guide Specifications* and *NCHRP Report 350.* Yield line and inelastic analysis and design procedures, as originally developed by Hirsch (7), were also provided for bridge railings as a replacement to the 44.5 kN (10,000 lb) equivalent static load design procedures.

Emphasis on the use of crash-tested rails for new Federally funded projects has significantly increased the role of fill-scale crash testing as a means of evaluating railing performance. Recently, the Federal Highway Administration (FHWA) officially adopted *NCHRP 350* as a replacement for *NCHRP 230* and has strongly suggested that AASHTO also adopt the test level definitions contained in *NCHRP 350,* thus making crash-tested railings mandatory for most bridges. Most highways on which wood bridges are installed will require railings that meet either the AASHTO PL-1 or PL-2 requirements, or the NCHRP 350 TL-1 through TL-4 requirements. A railing that meets either PL-3, TL-5, or TL-6 requirements currently has a very limited application for wood bridges because of the high traffic volumes and speeds associated with these levels.

As of August 1990, 25 bridge rails had been successfully crash tested in accordance with the requirements of the *AASHTO Guide Specifications* and approved for use on Federal-aid projects by the FHWA (8). Of these crash-tested railings, 24 are for concrete bridge decks and one is for a wood deck. For wood bridges to be viable and competitive with other bridges in the future, a range of crash-tested bridge railings for different wood bridge types was required. Based on this need, National emphasis was placed on developing a limited number of crash-tested railings for wood bridges.

Background

To meet the need for crashworthy railings for wood bridges, the USDA Forest Service, Forest Products Laboratory (FPL), in cooperation with the Midwest Roadside Safety Facility (MwRSF) of the University of Nebraska-Lincoln, the FHWA, and the wood products industry initiated a program to develop crash-tested bridge rails for longitudinal wood decks. The program objectives were to develop a total of nine crashworthy rails: three to meet AASHTO PL-1, one to meet AASHTO PL-2, three to meet NCHRP 350 TL-1, one to meet NCHRP 350 TL-4, and one intended for very low performance conditions. The scope of the project was limited to railings for longitudinal wood decks, 252 mm (10 in.) or greater in thickness, and constructed of glued-laminated (glulam) timber, spike-laminated lumber, or stress-laminated lumber. In each system, the lumber laminations are placed edgewise and oriented with the lumber length parallel to the direction of traffic. A brief description of each longitudinal deck bridge type is provided in *Timber Bridges: Design, Construction, Inspection, and Maintenance* (9).

Longitudinal glulam timber decks are constructed of panels that consist of individual lumber laminations glued together with waterproof structural adhesives. The panels are 1.07 to 1.38 m (3.5 to 4.5 ft) wide and effectively function as a large, solid block of wood. To form the bridge deck, panels are placed side by side and are interconnected by transverse distributor beams bolted to the deck underside at intervals of 2.4 m (8 ft) or less. These distributor beams are designed to transfer vertical loads between adjacent panels. They are not designed to resist lateral loads.

Spike-laminated decks are constructed of sawn lumber laminations 102 mm (4 in.) in nominal thickness. The individual laminations are interconnected with spikes that are typically 8 or 9.5 mm (0.3125 or 0.375 in.) in diameter and 356 to 406 mm (14 to 16 in.) long. The decks are commonly manufactured in panels that are 1.5 to 2.1 m (5 to 7 ft) wide and interconnected with transverse distributor beams in a manner similar to longitudinal glulam timber decks.

Stress-laminated decks are constructed of sawn lumber laminations that are typically 51 to 102 mm (2 to 4 in.) in nominal thickness. The laminations are stressed together with high strength steel bars that are placed through holes drilled through the center of the wide faces of the laminations. When tensioned, the bars create compression between the laminations, and the entire deck effectively acts as a solid, orthotropic wood plate.

Test Requirements and Evaluation Criteria

The test requirements and evaluation criteria for this project followed procedures defined in the *AASHTO Guide Specifications* (including applicable references to *NCHRP 230)* and the *NCHRP 350* criteria. These procedures establish a uniform methodology for testing and evaluating railings so that the safety performance of different railing designs, tested and evaluated by different agencies, can be compared. It is impractical and impossible to test all railings for all possible vehicle and impact conditions. Therefore, the procedures specify a limited number of tests using severe vehicle impact conditions and a set of evaluation criteria against which test results may be evaluated.

Test Requirements

Vehicle impact requirements for rail crash testing depend on the railing performance level/test level and are specified as requirements for vehicle type and weight, impact speed, and impact angle relative to the longitudinal rail axis. Testing for PL-1 and TL-1 requires two vehicle impact tests while testing for PL-2 and TL-4 requires three vehicle impact tests. A summary of the requirements for PL-1, PL-2, TL-1, and TL-4 are shown in Table 1. In some cases, all tests for a given level may not be required if a railing with similar geometry and strength was previously tested and found to be satisfactory.

In addition to vehicle impact requirements, the *AASHTO Guide Specifications* and the *NCHRP 350* criteria also specify requirements for data acquisition and construction of the bridge railing. Requirements for data acquisition are referenced to *NCHRP 230* and *NCHRP 350* and include specific data collection parameters and techniques that must be completed before, during, and after the crash test. Construction requirements specify that the bridge rail be designed, constructed, erected, and tested in a manner representative of actual installations. To properly assess the performance of most bridge rails, they must also be evaluated as a system in combination with the bridge superstructure for which it is intended. This is very important when considering rails for wood bridges because the attachment of the rail to the bridge deck and the ability of the wood superstructure to resist applied loads may often be the controlling parameters.

Evaluation Criteria

Evaluation criteria for full-scale crash testing is based on three appraisal areas: structural adequacy, occupant risk, and vehicle trajectory after the collision. Criteria for structural adequacy are intended to evaluate the ability of the railing to contain, redirect, or permit

Table 1 – Vehicle impact requirements for AASHTO PL-1 and PL-2 and NCHRP 350 TL-4 bridge railings.

	Impact Conditions		
AASHTO Performance Level (4)	Small Car (816 kg)	Pickup Truck (2,449 kg)	Single-Unit Van Truck (8,165 kg)
1	80.5 km/h 20 deg	72.4 km/h 20 deg	
2	96.6 km/h 20 deg	96.6 km/h 20 deg	80.5 km/h 15 deg
	Impact Conditions		
NCHRP 350 Test Level (5)	Small Car (820 kg)	Pickup Truck (2,000 kg)	Single-Unit Van Truck (8,000 kg)
1	50 km/h 20 deg	50 km/h 25 deg	
4	100 km/h 20 deg	100 km/h 25 deg	80 km/h 15 deg

controlled vehicle penetration in a predictable manner. Occupant risk evaluates the degree of hazard to occupants of the impacting vehicle. Vehicle trajectory after the collision is concerned with the path and final position of the impacting vehicle and the probable involvement of the impacting vehicle with other traffic. Note that these criteria address only the safety and dynamic performance of the railing and do not include service criteria such as aesthetics, economics, bridge damage, or post-impact maintenance requirements. The following evacuation criteria are summarized from the *AASHTO Guide Specifications* for PL-1 and PL-2 testing (similar evaluation criteria are provided in *NCHRP 350):*

1. The railing shall contain the vehicle; neither the vehicle nor its cargo shall penetrate or go over the installation. Controlled lateral deflection of the railing is acceptable.

2. Detached elements, fragments, or other debris from the railing shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.

3. Integrity of the passenger compartment must be maintained with no intrusion and essentially no deformation.

4. The vehicle shall remain upright during and after collision.

5. The railing shall smoothly redirect the vehicle. A redirection is deemed smooth if the rear of the vehicle does not yaw more than 5 degrees away from the railing from time of impact until the vehicle separates from the railing.

6. The smoothness of the vehicle-railing interaction is further assessed by the effective coefficient of friction where = 0.0 to 0.25 is good, = 0.26 to 0.35 is fair, and > 0.36 is marginal. Requirements for computing are given in the *AASHTO Guide Specifications.*

7. The impact velocity of a hypothetical front-seat passenger against the vehicle interior, calculated from vehicle accelerations and 610-mm (2-ft) longitudinal and 305-mm (1-ft) lateral displacements, shall be less than 9.15 m/s (30 ft/s) in the longitudinal direction and 7.63 m/s (25 ft/s) in the lateral direction. In addition, the vehicle highest 10-ms average accelerations subsequent to the instant of hypothetical passenger impact should be less than 147 m/s^2 (483 ft/s^2) in the longitudinal and lateral directions.

8. Vehicle exit angle from the barrier shall not be more than 12 degrees. Within 30.5m (100 ft) plus the length of the test vehicle from the point of initial impact with the railing, the railing side of the vehicle shall move no more than 6.1 m (20 ft) from the line of the traffic face of the railing.

Development Phase

Based on a fundamental understanding of the performance characteristics of each deck type, development work was initiated to formulate a methodology for the railing tests. From the standpoint of economics and time, it was considered impractical to develop and test different rail systems for each longitudinal deck type. Rather, a more feasible approach was undertaken to develop several railing systems that could be adapted to each of the three longitudinal deck types without modifications that would result in reduced performance. To accomplish this, it was determined that railing development and testing should utilize the weakest deck type. This conclusion was based on the premise that if successful tests could be completed on the weakest deck, the railing could be adapted for use on stronger decks without adversely affecting performance.

In assessing the potential resistance of each deck type to transverse railing impact forces, consideration was

given to the strength of the wood and mechanical reinforcement. Of primary concern was loading that could introduce tension perpendicular to grain stress in the wood deck.

Of the three deck types, the stress-laminated deck was considered the strongest for transverse railing loads, because the high strength steel bars are continuous across the deck width. Loads developed at vehicle impact can be effectively distributed across the deck by the bars, making the entire deck width effective in resisting the applied loads.

The spike-laminated deck was considered to be of intermediate strength. If rail loads are applied transverse to the panel length, the loads are resisted by the spikes in withdrawal. Because of this, tension perpendicular to grain in the lumber laminations is not a concern; however, the spikes could be pulled from the deck resulting in longitudinal separations between the laminations, and additional reinforcement could be required.

The glulam timber deck was considered to be the weakest in resisting railing loads, because the glulam timber panels act as solid pieces of wood, and loads applied transverse to the panel length are most likely to introduce tension perpendicular to grain and failure in the upper panel section. Mechanical reinforcement was considered necessary for longitudinal glulam timber decks to resist railing loads without damage. Thus, the glulam timber deck was considered the weakest deck for transverse railing loads and was selected for full-scale crash testing. If bridge railings performed acceptably on the glulam timber system, it was rationalized that the railings could be adapted to other longitudinal wood bridge decks with no reduction in railing performance.

The primary emphasis of the railing design process was to develop rails that would meet the requirements of the *AASHTO Guide Specifications* and *NCHRP 350*. Additionally, it was determined that consideration should be given to (1) extent of probable damage to the structure after vehicle impact and the difficulty and cost of required repairs; (2) adaptability of the railing to different wood deck types; (3) rail system cost to the user, including material, fabrication, and construction; (4) ease of railing construction and maintenance; and (5) aesthetics.

The conclusion of the development phase involved the design of several railing systems and preparation of plans and specifications for testing. The selection and design of these final systems were based on a review of other railings that had been successfully crash tested, as well as those that are currently used on wood bridges but had not been crash tested. To the extent possible, feasible designs were evaluated using computer simulation models. Although several proven computer models were used, it was difficult to adapt the programs for wood components because the behavior and properties of the wood systems at ultimate loading were unknown. Data collected during the crash testing were used to refine input parameters and to more accurately predict railing performance in subsequent tests.

Test Methodology

Testing of all bridge rails was conducted at the Midwest Roadside Safety Facility in Lincoln, Nebraska. The site is located at an airport and was formerly a taxiway and parking area for military aircraft. It includes approximately 11 ha (27 acres) of concrete pavement and 1.6 ha (4 acres) of soil surface. To perform the rail testing, a test bridge was constructed that measured approximately 2.4 m (8 ft) wide and 28.6 m (93.75 ft) long, in five simply-supported spans measuring 5.72 m (18.75 ft) each. The deck was constructed of 273-mm (10.75 -in.) thick glulam timber panels, 1.2 m (4 ft) wide. The glulam timber for the deck was Combination No. 2 Douglas Fir given in the *AASHTO Standard Specifications for Highway Bridges* (1) and was treated with pentachlorophenol in heavy oil in accordance with American Wood Preservers' Association (AWPA) Standard C14 (10). Two glulam timber panels were placed side by side to achieve the 2.4-m (8-ft) width, and transverse distributor beams were attached to the deck underside per AASHTO requirements (l). The test bridge was supported by concrete footings that were placed in excavations so that the top of the test bridge was level with the concrete surface at the site.

Vehicle propulsion and guidance were provided by steel cable configurations. For propulsion, a reverse cable tow with a 1:2 mechanical advantage was used. A cable was attached to the front of the vehicle, routed through a series of pulleys, and was connected to a tow vehicle that traveled in a direction opposite to the test vehicle. The unoccupied test vehicle was then pulled by the tow vehicle and released from the tow cable approximately 9.2 m (30 ft) prior to impact. A vehicle guidance system developed by Hinch was used to steer the test vehicle (11). Using this system, the the left front wheel hub is attached to a tensioned steel cable that maintains the vehicle's direction along a designated straight path. Approximately 9.2 m (30

ft) from impact, the guidance connection is sheared off and the vehicle separates from the guidance cable.

Data acquisition parameters and techniques for the crash testing program were based on requirements of the *AASHTO Guide Specifications* and *NCHRP 350* and followed three testing phases: pretest, test, and post-test. In the pretest phase, the as-built bridge rail and vehicle were documented using photography and drawings that indicated the applicable configuration, dimensions, and vehicle weight. During the test phase, data regarding the vehicle impact speed, impact angle, trajectory, and accelerations were collected primarily through the use of high-speed motion picture photography and accelerometers mounted on the vehicle. In the post-test phase, the condition of both the railing, bridge superstructure, and vehicle were documented using photography and standardized damage assessment methods, including the Traffic Accident Data Scale (12) and Vehicle Damage Index (13). Additional instrumentation was placed on some railings to assess vehicle impact forces transmitted to the bridge rail and superstructure (14).

Successfully Tested Railings

As a result of the development and testing program, nine bridge railings were successfully developed and tested for longitudinal wood decks. Three of the railings were tested at PL-1, one was tested at PL-2, three were tested to TL-1, one was tested at TL-4, and one was tested primarily for low-volume forest road applications at impact conditions less than TL-1. Each railing was tested on the glulam timber deck and is adaptable to the spike-laminated and stress-laminated decks. All the PL-1, PL-2, and TL-4 designs employed posts spaced 1.9 m (6.25 ft) on-center and utilized high strength steel bars through a portion of the bridge deck to act as reinforcement in distributing railing loads without damage to the bridge. Two of the three TL-1 designs employed posts spaced 1.9 m (6.25 ft) on-center, while the third TL-1 design was a curb railing configured with scuppers spaced 3.0 m (10 ft) on-center. The railing design for conditions less than TL-1 consisted of a curb railing with scuppers spaced 2.9 m (9.5 ft) on-center. Glulam timber for the rail members was Combination No. 2 Douglas Fir as given in the *AASHTO Standard Specifications for Highway Bridges* (1), treated with pentachlorophenol in heavy oil to AWPA Standard C14 requirements (10). Sawn lumber for posts, curbs, scuppers, and spacer blocks was No. 1 Douglas Fir (1), treated with creosote to AWPA Standard C14 requirements (10).

A detailed discussion of the testing and results for each railing system is beyond the scope of this paper and, for most of the railing systems, is presented in detail in previous publications (14-18). Overall, no significant damage to the test bridge was evident from any of the vehicle impact tests. For the railing systems with glulam timber rails, damage to the railing was primarily gouging and scraping resulting from the vehicle impact. All glulam timber railing remained intact and serviceable after the tests and replacement of the railing was not considered necessary. For the steel thrie beam railings, there was permanent deformation in the rail and post in the vicinity of the impact location. This would necessitate replacement of specific railing and post members, but damage was relatively minor considering the severity of the impact. A brief description of each railing that met all test criteria follows.

PL-1 Railings

The three tested PL-1 railings included a glulam timber rail with curb, a glulam timber rail without curb, and a steel rail. Photographs and drawings of the PL-1 railings are shown in Figures 1 and 2, respectively.

The glulam timber rail with curb consisted of a single glulam timber rail mounted on a sawn lumber post. The post was connected with a single bolt to a lumber curb that was supported by scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts and timber connectors.

The glulam timber rail without curb consisted of a single glulam timber rail mounted on a sawn lumber post. The lower portion of the post was placed in a steel box that was attached to the bridge deck with high strength steel bars.

The steel rail consisted of a 10-gauge steel thrie beam rail mounted to a steel wide flange post. The lower end of the post was bolted to a steel plate that was connected to the bridge deck with high strength steel bars.

PL-2 Railing

The PL-2 railing included a steel rail and steel channel section, as shown in Figure 3. The steel rail was a modified version of that tested at PL-1. Minor changes in the railing geometry and the addition of a steel channel section above the rail were necessary to resist the increased loads at PL-2.

TL-1 Railings

The three tested TL-1 railings included a flexible steel rail, a semi-rigid steel rail, and a curb-type timber rail.

Photographs and drawings of the TL-1 railings are shown in Figures 4 and 5, respectively.

The flexible steel rail consisted of a 12-gauge W-beam rail mounted to a breakaway sawn lumber post. The lower end of the post was placed between two steel angles that were connected to the vertical edge of the bridge deck with lag screws.

The semi-rigid steel rail consisted of a 12-gauge W-beam rail mounted to a steel wide flange post. The post was bolted to a steel plate which was bolted to the bridge deck surface.

The low-height, curb-type timber rail was constructed with a glulam timber rail and supported with scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts and timber connectors.

TL-4 Railing

The TL-4 railing included a glulam timber rail with curb, as shown in Figure 6. The glulam timber raiI with curb consisted of a single glulam timber rail mounted on a sawn lumber post and was a modification of the curb system tested at PL-1. Because of the greater loads at TL-4, rail and post sizes were increased and bolts and timber connectors attaching the curb and scupper to the bridge deck were increased.

Low Performance Railing

The low performance railing developed for impact conditions less than TL-1 included a low-height timber curb rail, as shown in Figure 7. Three geometries were considered for the curb rail — a square shape, a rectangular shape, and a trapezoidal shape. The curb rail was constructed with sawn lumber and supported with scupper blocks. The curb and scupper blocks were connected to the bridge deck with bolts.

Concluding Remarks

This program clearly demonstrates that crashworthy railing systems are feasible for longitudinal wood decks. Even at high-impact conditions required by AASHTO PL-2 and NCHRP 350 TL-4, the railing systems performed well with no significant damage to the bridge superstructure. With the development of crashworthy railing systems, a significant barrier to the use of longitudinal deck wood bridges has been overcome.

Figure 1 — Photographs of bridge railings successfully crash tested to AASHTO PL-1 (photographs taken prior to testing).

Figure 2 – Drawings of bridge railings successfully crash tested to AASHTO PL-1. Drawing units are millimeters

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Figure 3 – Steel thrie beam bridge railing successfully crash tested to AASHTO PL-2 (photograph taken prior to testing). Drawing units are millimeters.

Figure 4 – Photographs of bridge railings successfully crash tested to NCHRP 350 TL-1 (photographs taken prior to testing).

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Figure 5 – Drawings of bridge railings successfully crash tested to NCHRP 350 TL-1. Drawing units are millimeters.

Figure 6 – Glulam timber bridge railing successfully crash tested to NCHRP 350 TL-4 (photograph taken prior to testing). Drawing units are millimeters.

Figure 7 – Low performance bridge railing successfully crash tested with a 3/4-ton pickup truck at 15 mph and 15 degrees (photograph taken prior to testing). Drawing units are millimeters.

Development of Wood Highway Sound Barriers

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Abstract
Prototype designs for wood highway sound barriers meeting the multiple criteria of structural integrity, acoustic effectiveness, durability, and potential for public acceptance are being developed. Existing installations of wood sound barriers were reviewed and measurements conducted in the field to estimate insertion losses. A complete matrix of design options for wood barriers was developed into a set of slides along with several concrete designs, and presented in a controlled test to a group of human subjects for evaluation. The results of this testing showed that the wood barrier designs present an acceptable appearance, both to the driver and to the community behind the barrier. Moreover, the tests indicated a preference for moderate relief treatment, or a variety of design elements. The results of the human subject and acoustic testing have been incorporated into a series of designs for wood sound barriers. A prototype barrier will be built and tested.

Keywords: noise, barriers, acoustics, aesthetics.

Introduction
The use of sound barriers in the United States has gained popularity as real estate developments along highways continue to grow. The total length of sound barriers constructed in this country equaled over 1,486 linear kilometers in 1992 (FHWA, 1994). These barriers are constructed using earth, precast concrete, concrete block, brick, wood, metal, and combinations of these materials at a total cost of over 875 million (in 1992 dollars) (FHWA, 1994). Most of these barriers are constructed of concrete masonry and concrete, followed by earth berms and wood. Approximately 17 percent of barriers were constructed of wood or a combination of earth berm and wood (Cohn and Harris, 1990). Recent trends concerning the percentage of wood barriers constructed with respect to masonry, concrete, and combinations of the above materials show that this percentage is decreasing (Weiss, 1989). Much of the reason for the trend against the use of wood barriers relates to problems concerning durability, aesthetics, and acoustic effectiveness. Due to inadequate design and detailing many wood barriers deteriorate due to exposure, causing them to degrade, not only in appearance, but acoustic effectiveness as well. Panels in the barrier may deform and pull apart, causing sound to leak through the barrier.

A goal of this research is to develop a coordinated approach to the design of wood highway sound barriers. The acoustic effectiveness and public acceptance of barriers will be evaluated first in order

to limit design options for assessment of costs, durability, and structural integrity of wood and concrete barriers. Acoustic testing of existing barriers is used to evaluate the acoustic effectiveness of several design types. Testing of human subjects' impressions of computer edited images is used to evaluate the public acceptance of different design types. These initial testing programs were used to develop guidelines for the design of wood barriers, including guidelines for effective acoustic design and for aesthetic treatments generally acceptable to the public. These guidelines are now being applied to the development of a series of prototype designs of highway sound barriers.

Acoustic Effectiveness

In-situ measurements

The acoustic effectiveness of sound barriers was determined by in-situ testing of existing sound barriers. Because extensive data on insertion losses and transmission losses of different barrier types do not exist, the in-situ testing of the different design types and materials was necessary. The goal of this testing is to determine the insertion losses and transmission losses of different wood and concrete barriers. These data were normalized for geometric differences among the barriers to allow for direct comparison between the different barrier design types. The objective was to determine if wood and concrete barriers are equivalent in terms of acoustic effectiveness.

The wood barrier designs investigated in the in-situ testing are listed in the Guide Specification for Highway Noise Barriers (NFPA, 1985). The three design types listed include timber plank, plywood, and glued-laminated barriers. Timber barriers, also called post & panel barriers, are barriers that employ heavy timber posts along with dimension lumber panels. Plywood barriers are barriers that employ plywood panels, usually supported by dimension lumber. Glued-laminated barriers employ glued-laminated wood members to create a noise barrier.

With the assistance of the Departments of Transportation of Pennsylvania, Maryland, New York, and New Jersey, as well as from representatives from the wood industry, wood barriers were located in these states. Barriers selected for testing were located on relatively flat ground and were accessible from both sides. Ten wood barriers were selected for the in-situ testing four glued-laminated barriers, four timber barriers, and two plywood barriers. The glued-laminated barriers were located in Erie on the east and west sides of I-79, outside Washington D.C. on I-495, and near Troy, New York, on Route 7. Of the timber barriers, two were located on the Hutchinson River Parkway outside New York City and the other two were on the Long Island Expressway in New York. The plywood barriers were located at a truck weigh station across the Pennsylvania / Maryland border on I-83 and outside Baltimore on I-95. Five precast concrete panel barriers were also identified for the in-situ testing. The concrete barriers were located on I-79 in Erie, I-695 outside Baltimore, I-95 outside New York City, Route 24 outside Whippany, New Jersey, and I-78 in New Jersey. The Erie glued-laminated barriers were 15 years old while the rest of the barriers were less than 10 years old.

Insertion Loss Calculations

Insertion losses of the selected barriers were determined. Because measurements prior to installation of the barriers were not available, and equivalent sites without the barrier could not be located, the indirect predicted method given in ANSI S12.8-1987, *Methods for Determination of Insertion Loss of Outdoor Noise Barriers*, was used with the prediction model specified in the FHWA Highway Traffic Noise Prediction Model (FHWA-RD-77-108, 1978).

With the estimated pre-barrier and measured post-barrier sound levels, it was possible to estimate the insertion losses. The pre-barrier sound levels were corrected for sound level, spreading loss and ground effects, and sensitivity differences between microphones to predict levels without the barrier. The post-barrier sound levels were corrected for background noise, and barrier insertion loss was determined by subtracting post-barrier sound level from the pre-barrier sound level. These calculations were done for each of the two or three measurements made at each of the three receiver locations of 3.1, 7.6, and 15.3 m behind the barrier. The insertion losses of each barrier were the calculated mean in each l/3-octave band between 200 Hz and 5 kHz.

Normalization of Estimated Insertion Loss

Variation was observed in the insertion losses within groups of barriers of similar design type Much of the variation was due to differences in height of the barriers, topography, and distance of the barrier from the roadway. The barrier heights and distances from the roadway, for the barriers for which valid sound

Table 1 –Barrier Heights and Distances from Roadway

Barrier Type	Barrier Location	Height (m)	Road to Barrier (m)
Precast Concrete	I-695, Baltimore	6.7	9.0
Precast Concrete	I-95, New York City	4.9	6.6
Precast Concrete	I-78, New Jersey	2.5	10.9
Precast Concrete	Rt. 24, Whippany, NJ	5.3	9.8
Plywood	I-95, Baltimore	3.3	2.4 (6.1)
Plywood	I-83, Maryland Weigh Station	4.3	24.4
Glu-lam	I-495, Washington D.C.	5.1	1.5 (6.1)
Glu-lam	Route 7, Troy, NY	2.5	4.9
Post & Panel	Long Island Expressway, NY	6.7	12.2
Post & Panel	Hutchinson River Parkway, NY	1.9	7.6
Post & Panel	Hutchinson River Parkway, NY	3.3	14.0

measurements were made, are listed in Table 1 for each barrier. The normalization of the insertion losses for the three different receiver locations involved the use of the prediction model presented in FHWA-RD-77-108 (FHWA, 1978). The difference between the insertion loss of a barrier estimated from measurements and the predicted value for a barrier at a given height and distance from the road on a flat site was the normalization factor for a barrier. The estimated insertion losses are normalized by subtracting the normalization factor from the estimated insertion loss. The insertion losses for all the barriers were normalized to a height of 4.3 m, a distance of 9.2 m from the roadway, and a flat site. The height, 4.3 m, was chosen as a typical height for barriers. The distance, 9.2 m, was chosen because it is the mean of the source-to-barrier distances for barriers used in this research project. Normalization of estimated insertion losses allowed direct comparisons of all the test barrier types and locations.

A-weighting of Insertion Losses

To approximate the sound heard by the normal human ear, A-weighting was applied to the normalized insertion losses and transmission losses. The A-weighted sound level, with the units of dBA, is the sound pressure levels in decibels measured with a frequency weighting network corresponding to the A-scale specified by ANSI S1.4-1971 (ANSI, 1971). The A-scale tends to attenuate levels of frequencies below 1000 Hz. The A-weighted normalized insertion and transmission losses are presented in Table 2. The tabulated values are the A-weighted normalized insertion losses for the three receiver locations as well as the A-weighted transmission losses for each barrier.

Results

The A-weighted insertion losses in Table 2 allowed comparisons of the insertion losses for different barrier design types. The same data allowed comparison of each barrier's performance to a target minimum insertion loss of 10 dBA at 3.1, 7.6, and 15.3 meters behind the barrier.

The mean standard error values, as well as the range and coefficients of variation of the standard error values, are presented in Table 3. Tabulated values are for the three insertion loss calculations as well as for the transmission loss calculations. The table further breaks down the standard error by finding the error in calculations affected by background noise and calculations not affected by background noise.

Table 3 –Standard Errors for Insertion Loss Calculations

receiver @[3]	3.1 m	7.6 m	15.3 m
mean (dB)	1	1	1
max. (dB)	2	2	2
min. (dB)	1	1	1
C.O.V. (%)	7	9	10

With the exception of the concrete barrier on I-87, the estimated transmission losses are higher than the estimated insertion losses, indicating that transmission through the barrier is not a dominating factor in the acoustic performance of the barriers.

The precast concrete barriers generally had a fairly high insertion loss (7 to 20 dBA) for the three receiver positions shown with lower losses at 15.3m (7 to 16 dBA) than closer to be barrier at 3.lm (12 to 19 dBA).

Table 2 –A-Weighted Normalized Insertion Losses and Transmission Losses

Type	Location	Height (m)	Road to Barrier (m)	IL @3.1m (dbA)	IL @7.6m (dbA)	IL@ 15.3m (dbA)	TL (dbA)
PC[a]	I-695, MD	6.7	9.0	12	10	7	19
PC	I-95, NY	4.9	6.6	18	17	12	22
PC	I-78, NJ	2.5	10.9	19	16	16	17
PC	SR 24, NJ	5.3	9.8	19	14	14	22
Plywood	I-95, MD	3.3	6.1	17	12	8	15
Plywood	I-83, MD	4.3	24.4	7	6	7	14
Glu-lam	I-495, DC	5.1	6.1	15	11	7	21
Glu-lam	SR 7, NY	2.5	4.9	16	14	10	20
P & P[b]	LIE[c], NY	6.7	12.2	18	11	7	15
P & P	HRP[d], NY	1.9	7.6	21	18	15	15
P & P	HRP, NY	3.3	14.0	12	14	15	15

[a]Precast Concret
[b]Wood Post and Panel
[c]Long Island Expressway
[d]Hutchinson River Parkway

There are significant differences in the estimated insertion losses for the two plywood barriers; the losses for the barrier on I-95 range from 17 to 12 dBA at 3.1m and 7.6m, where the losses for the barrier on I-83 range from 7 to 6 dBA at 3.lm and 7.6m. This is partly due to the differing distance of these two barriers from the roadway the barrier on I-95 was 2.4m from the roadway and the barrier on I-83 was 24.4 meters from the roadway, almost twice the distance for any of the other barriers tested in the program. This, along with clearly observable leaks in the barrier produced the low estimated losses presented in Table 2. However, the I-95 barrier does show losses that fall within the range of the losses for the concrete barriers, except for the transmission loss, which is lower than any of the transmission losses for the concrete barriers. The estimated insertion losses and transmission losses for the glued-laminated barriers are within the range of both the estimated losses for the concrete barriers. The only estimated insertion loss that falls below 10 dBA is for the barrier on I-495 at 15.3m. Because this barrier was very close to the four one-way lanes of I-495, an accurate equivalent source location was difficult to provide, and the sensitivity to the normalization factor is higher for smaller barrier-to-roadway distances. The lower estimated insertion loss for the I-95 barrier at 5.3m may be partly due to the inability to provide an accurate normalization. All of the concrete barriers had much greater roadway to barrier distances (see Table 1). The high value of transmission loss (21 dBA) indicates that the acoustic performance for this glued laminated barrier is similar to the concrete barriers with estimated transmission losses of 19 to 22 dBA.

The post & panel barriers, on the other hand offered the most divergent set of estimated insertion losses. Two reasons can account for the variable results. Background noise masked some of the 1/3 octave band levels for the 1.9 meter high Hutchinson River Parkway barrier, providing incomplete results. At the same time, both of the Hutchinson River Parkway barriers were influenced by high correction factors in the normalization process, since the barrier was low, and the distance from the roadway was large. The attenuation results should, consequently, not be used to make final decisions about post & panel barriers. The only remaining barrier, the one on the Long Island Expressway, had insertion loss values of 18 dBA at 3.1 m, 11 dBA at 7.6 m, and 7 dBA at 15.3 dBA and a transmission loss of 15 dBA.

Public Acceptance

Selection of Design Types

Computer edited images, presented in 35 mm slides, were designed for subjects to make evaluations on the general appearance of the barriers rather than the appearance of specific barrier design types. Thus, the slides presented images that vary in barrier layout and panel orientation rather than finish or detail. Barrier layout considered variations in the plans of the barriers. Variations included flat or linear plan, relief

Table 4–Matrix of Barrier Design Types for Public Acceptance Evaluation

	FLUSH	RELIEF	SHADOWBOX
Vertical - wide strips	F1 / B1	F2 / B2	F3 / B3
Horizontal - narrow strips	F4 / B4	F5 / B5	F6 / B6
Vertical - narrow strips	F7 / B7	F8 / B8	F9 / B9
Horizontal - wide strips	F10 / B10	F11 / B11	F12 / B12
Combination - vertical and horizontal	F13 / B13	F14 / B14	F15 / B15
Concrete	F16 / B16	F17 / B17	F18 / B18

plan, or shadowbox plan. The flat plan had the posts and panels centered on a single line. The relief plan had the posts centered on a single line while the panels alternate being connected to the posts' front arid back. The panels were either connected to the front of one post and to the back of the next or alternated being connected front-to-front and back-to-back of posts. The shadowbox plan was similar to the relief plan except that the relief in the barriers are deeper, requiring separate posts to be installed to achieve the depth of the relief

Panel orientation considered variations in the elevation of the barriers. Variations included wide and narrow strips, horizontal and vertical strips, and combinations of these variations. All these variations in plan and elevation were developed after reviewing designs of existing barriers. With the wide variety of designs, the goal was to include most of the feasible barrier designs in the public acceptance evaluation. Even tbough it did not identify the kind of wood products to use in the design, it did identify the appearance of the barriers more likely to be accepted by the public.

For comparison purposes, slides of concrete barriers in the three layout variations were created and also evaluated by the subjects. These barriers have the standard wide horizontal panels used for precast concrete barriers in the three different plan layouts. Together, all these variations allowed development of a matrix of different barrier designs. This matrix allowed for a wide variety of wood barriers to be compared.

The combination of letters and numbers shown in Table 4 are codes which identified the slides for the rest of this research. The letters F and B refer to Front side (highway side) and Back side (residential side). These letters were then applied to each box in the matrix which had its own unique number. For example, Slide F5 was the slide with the barrier constricted of widehorizontal panels in the relief plan layout and viewed from the front or highway side. Altogether, there were 36 slides with half containing views from the front and half containing views from the back. Figures 1 and 2 illustrate example front and back views of Slides F5 and B5.

Figure 1–Computer-generated front view of a wood sound barrier.

Figure 2–Computer-generated back side view of a wood sound barrier.

Human Subjects Testing

Human subjects evaluated the appearance of highway sound barriers using the 36 computer-edited slides of barriers.

Twenty-four human subjects rated the 36 slides using semantic-differential (SD) rating scales and individual rating scales. Both of these scales are described later. The human subjects were divided into three groups of eight subjects each. Each group rated all 36 slides

using the SD scales and a select group of slides using the individual rating scales. The two scaling techniques were used in separate sessions by each group. This divided the concentration time for each group into more reasonable intervals. Within the two sessions, the slides were divided into two groups, the slides of barriers viewed from the highway side and those viewed from the residential side. The subjects were first presented with slides of barriers viewed from the highway side and were asked to rate the barriers from the perspective of a driver. After a five minute break, the subjects were presented with slides of barriers viewed from the residential side and were asked to rate the barriers from the perspective of a homeowner living next to the barrier. The goal was to determine if differing perspectives would result in different ratings.

The subjects included undergraduate students, graduate students, and staff at the Pennsylvania State University, as well as people employed elsewhere in State College. The undergraduate and graduate students were from a variety of locations throughout the United States and world. The subjects' ages ranged from 20 to 40, with the majority of subjects having their age in the mid to high 20s. All the subjects were read the same set of instructions, were given the same set of forms to read and sign, were presented with same examples of sound barriers, and were presented the same set of slides. Each group had the slides presented in a different sequence.

Semantic Differential Rating Scales. –Subjects first used semantic-differential (SD) scales to evaluate the 36 slides. The SD scales were designed to elicit specific responses to attributes of the various barrier designs. They attempted to identify all the factors involved in a person's opinion about sound barrier designs. The subjects' responses obtained by these scales were statistically examined to determine how many factors influenced the subjects' responses and which attributes of the barriers caused these responses.

Principal components factor analysis and analysis of covariance were used to determine which scales were used in a consistent manner and the statistical significance of the results. However, the results of the SD scales also helped select the slides shown in the individual rating scales. The slides selected were those which drew favorable responses in the SD scales on both the highway and residential sides. For comparison purposes, the concrete barrier which received the most favorable responses, as well as the barrier which received the most unfavorable responses, were included in the individual rating scales.

Individual Rating Scales –The slides selected from the SD scales were displayed again to the human subjects, who were asked to rate these barriers with individual rating scales. These differed from the SD scales since they asked the subject to rate the barrier using whatever criteria they establish. Thus, while SD scales attempted to evaluate specific areas of subjective impression, the individual scales attempted to evaluate the overall impression of the slide.

The individual ratings themselves were quite simple. After selecting the slides by reviewing the results of the SD scales, these slides were all presented to the subjects before any ratings were performed. The subjects were then asked to rate each design on a scale from 1 to 10 with the rating to be done relative to all the other designs shown. Again, this procedure was repeated twice within each group of human subjects, once for the slides viewed from the driver's perspective and once for the slides viewed from the homeowner's perspective.

Analyses of covariance (ANCOVA) determined if variations between the averaged results of the groups and slides were statistically significant. The set of slides used in the individual ratings contained slides that received favorable and unfavorable responses in the SD scales. The ANCOVA determined if statistical significance existed between ratings of slides favored and unfavored in the SD scales. The set contained slides of wood and concrete barrier designs allowing for the ANCOVA analyses to determine if the ratings of concrete barriers showed statistically significant differences from ratings of wood barriers. Thus, in many ways, the individual rating scales served as a check of the findings obtained by the SD scales. Barriers which were favored in the SD scales were evaluated by the individual rating scales. If the responses of the individual rating scales reinforced the findings of the SD scales, then the SD scales were given more validity. If the individual rating scales and SD scales did not have the same trend, it may be inferred that the SD scales did not find the correct responses or factors which are involved in a subject's opinion. Thus, individual rating scales and SD scales were a pair of tools which, when used together, greatly strengthened the survey results.

Analysis of Rating Scales

Principal Component Factor Analysis of Semantic-Differential Rating –The group averages of the SD scales for each slide were subjected to a principal components factor analysis (Flynn, 1979). This analysis statistically determined the subsets of rating scales that were used in similar or consistent ways by the subjects. A group of scales evaluated in a consistent manner had high intercorrelation. Strong differences in subjects' ratings of factors are not required, but rather consistent rankings of alternatives result in high intercorrelation. When the ratings' means are calculated, these factor subsets can be plotted so that alternatives being tested can be directly compared. Their validity can be seen in the graphs if scales that should have a high intercorrelation consistently rank alternatives in a similar manner. Reasons for inconsistent rankings must be determined.

Three factors, which were identified and named after the data analysis, caused 75.7 percent of the variance in the ratings for the driver's perspective and 76.4 percent of the variance in the ratings for the homeowner's perspective. These three factors were evaluated by the same SD scales for both perspectives, indicating that the three factors were the same for the two perspectives. Because the rest of the factors for both views only explained about four percent, or less, of the variances, the three factors mentioned above will be used for the rest of this analysis and the analysis of variance.

The three factors which caused a majority of the variances in the ratings were named for identification purposes. The first factor had the scales such as appropriate/inappropriate, pleasant/foreboding and attractive/unattractive, were the subjects' emotional responses to the barrier designs and were given the name *evaluative.* The second factor had scales such as darkening/lightening, bright/dim, public/private, and rural/urban, were concerned with the subjects' impressions of the effect barrier designs had to the surrounding and were given the name *environmental.* The third factor had the scales fortifying/weakening and safe/unsafe, were concerned with subjects' impressions of the personal state of being caused by the different barrier designs and were given the name *physical.*

The results of the statistical analysis of the SD ratings, reveals that barriers which were more "disliked" had distinctive results in the SD scales while barriers which were "liked" did not have distinctive results. The subjects' individual SD ratings consistently rated certain barriers unfavorably, causing the distinctive results. The subjects' individual SD ratings, however, rated different barriers favorably. The group means of the barriers favored by the majority of subjects are slightly lowered by the unfavorable ratings from a few subjects, causing the results to be less distinctive. One of the tendencies in the results was the dislike of the shadowbox plan Except for slide F3, all the slides of shadowbox designs received negative responses from the subjects.

Another tendency shown in the results was the negative responses towards precast concrete barriers. Except for slide F17, all the other concrete barriers received negative responses. For comparison purposes, the slides F17/B17 were included in the individual rating scales to check this assumption. Even though the concrete barriers received responses such as "bright" and "lightening" these barriers did not receive more favorable evaluative responses. These barriers were rated as more "fortifying" and "urban" than the wood barriers They also received negative evaluative responses, making the color of the barrier a trait which can only help, not cause, a barrier to be acceptable. That the only wood barrier receiving a "lightening" and "bright" response were F7 and B7, this tendency is reinforced. These two slides did not receive as many positive responses as some of the other barriers. Since the results of this study suggest that a lighter finish on a wood barrier was more favorably received than a darker finish, the value of the finish on a wood barrier is a factor meriting further investigation.

Analysis of Variance of Semantic-Differential Rating Scale Results –The results from the semantic-differential scales and the principal components analysis were used to determine critical differences in value. The critical differences were determined in two steps. One, the mean of the scales contributing to a factor were determined. Then, these means were subjected to an analysis of covariance (ANCOVA) with post hoc tests to determine which means have critical differences. These steps are described in this section.

An analysis of covariance was conducted to determine if any mean of a group is significantly different from the other group means and if any mean of a slide is significantly different from the other slide means. Because it is an omnibus test, it did not specify which

group's mean was significantly different from the others nor did it specify which slide's mean was significantly different the others. It only alerted the user that there are significant variations in the results. An F-test was conducted to determine if there was significant variation between the group and slide means at a significance level a = 0.05. A Tukey, HSD (honestly significant difference), post hoc test was used to determine significant differences between individual means (Stoline, 1981).

The variation between slides ratings for each of the factors were analyzed with a Tukey post hoc test. The results reinforce the tendencies identified earlier in the SD ratings. Barriers which were disliked, such as F6, had distinctive ratings while barriers which were liked did not have distinctive ratings. At best, their acceptance was lukewarm. Again, a simple plan layout and panel orientation for barriers was favored while the concrete and shadowbox barriers were disliked. Thus, it was concluded that no significant difference in values was observed between the groups and that the preliminary observations can be investigated further with the individual rating scales.

Analysis of Variance of Individual Rating Scales -The variation between individual rating scale group means was analyzed using analyses of covariance with Tukey post hoc tests. The results of the ANCOVA showed that there was no significant difference between the three human subjects groups but there was a significant difference between the slides. The ANCOVA determined the F-test values for the variations between slides and between subject groups for each slide. Using a significance level of 0.05, the critical values of F for the slides and groups were determined to be 2.15 and 1.74, respectively. While both perspectives had significant values for the variations between slides, neither view had significant values for the variations between groups.

Comparison of Rating Scales Results

With the semantic-differential and individual rating scales completed and analyzed, results were combined to observe tendencies which would aid in developing general design guidelines for wood sound barriers. The preliminary guidelines established by the SD scales were the unfavorable response towards concrete and shadowbox designs and the favorable response towards simple designs. By comparing the preliminary guidelines with the results of the individual rating scales, it was possible to determine if these guidelines were reinforced by the individual rating scales.

The first preliminary guideline was the dislike of the shadowbox design. This design type received negative responses by both rating scales (e.g., the slides receiving the most negative responses was F6/B6). Not only did the subjects dislike its appearance, but they found it more "fortifying" than barriers with the same panel elements organized in the flat and relief plan layouts. Thus, shadowbox designs are not recommended for wood sound barriers from aesthetic viewpoint.

The second preliminary guideline was the negative response towards precast concrete barriers. The subjects gave negative responses concerning the appearance of these barriers and also found them to be more "fortifying" and "urban" than the wood barriers. By including the most, and only, favorably received concrete barrier slide and its view from the other side, F17/Bl7, in the individual rating scales, it was found that the barrier received an unfavorable response on the view of the back side only. It appears that the unfavorable ratings of concrete barriers may result from the backyard view of these barriers

The final preliminary guideline from the SD scales was that the design type should be simple in plan layout and panel orientation. The four barriers receiving favorable responses in the individual rating scales shared this trait. The slides of these barriers were F2/B2, F4/B4, F10/B10, and F1l/B11. Even though there was not one plan or panel layout that these slides favor, the slides F13/B13 and F17, which received ratings close the favored barriers' ratings, offered some more insights into the preferred design traits of barriers. These slides had simple flat or relief barriers as well. Thus, all of these barriers suggested simplicity for the designs receiving the most favorable responses without a plan layout or panel orientation being specified.

Barrier Design Development

Design of Barrier System

On the basis of the testing programs, the original matrix of barriers shown in Table 4 has been modified by ruling out the shaded cells in Table 5, to a more manageable series of eight design options. The shadowbox designs have been ruled out on the basis of their generally poor acceptance in the human subjects testing Vertical orientation of timber plank and glued laminated panels

Table 5-Matrix of Barrier Design Types for Standard Drawing Production

	FLUSH	RELIEF		SHADOWBOX
	X	FLAT	SKEW	
Vertical - wide strips	X			
Horizontal - narrow strips	X	X	X	
Vertical - narrow strips	X			
Horizontal - wide strips	X	X	X	

requires a horizontal purlin for stability, which is incompatible with the relief design options. The remaining options are shown in Table 5 are being incorporated into a series of standard drawings and specifications, using either glued laminated or dimension lumber. Two different types of relief treatments are shown on the matrix and will be presented in the final design: in the flat relief treatment, panels between the posts are parallel to the roadway and attach at both ends to the front or back of the pints in alternating fashion. In the skewed treatment, the panels and posts are rotated slightly with respect to the roadway and the panels all attach to the back of one post and the front of the adjacent post.

The final design matrix consists entirely of design options that have common details, allowing for a systematic approach. The posts, for instance will be the same for any of the designs under equivalent conditions of height, geography, etc., and will only need to be designed once for all eight proposed design options. The intermediate panels will not differ significantly from one configuration to another and will not require an entire set of eight designs. Thus, the end user of the designs, will have available an entire palette of designs that may be used uniformly, or may vary within a single barrier design, depending on the preferences of the designer, the community, and the adjacent property owners.

Prototype Construction and Testing

Among the eight design options within the final scheme, it has been chosen to build the proposed test barrier in the flush configuration using glued laminated posts and tongue and groove timber plank panels. This configuration has been chosen as the simplest to build, and as having met the criteria for public acceptance based on the testing completed to date. However, the same post configuration can be used for most of the other design alternatives in the matrix, including variations in the relief treatment the substitution of glued laminate panels for solid sawn panels, and the use of alternative top treatments. The panel material will be 2x6 T & G Southern Yellow Pine, CCA treated (0.40 pcf retention), and field treated with a water repellent coating. The posts will be glued laminated SYP 16F-V5 6 3/4 X 11, CCA treated (0.60 pcf and field treated with a water repellent coating. The test barrier will be assembled on the test facility at the Pennsylvania Transportation Institute in May 1996.

The testing program for the prototype barrier will include acoustical measurements before and after construction of the barrier, and will include measurements made with horizontal cracks intentionally inserted within the panels and with and without a T-profile top treatment.

The prototype barrier will also be used to improve methods for estimating insertion losses of existing sound barriers. In the in situ study described above, it was necessary to rely on noise measurements made only after the barrier was in place and to estimate noise levels that would exist without the barrier. Measurements were also limited to favorable atmospheric conditions, that is, wind speeds less than 5 km/h. It is clear from this experience that improvement in prediction of noise levels without the barrier based on measurements made with the barrier under a wide range of atmospheric conditions is needed. So, the wood barrier that is under construction will be used for extensive measurements of noise propagation under differing atmospheric conditions, both before and after construction of the barrier. Measurements will also be made at an equivalent site near the barrier under a wide range of atmospheric conditions. These measurement, along with computer models of outdoor sound propagation, will be used to develop methods for the use of in-situ measurements to accurately estimate insertion losses for barriers.

Conclusions

The in-situ testing program of existing wood sound barriers has concluded that properly designed, detailed, and maintained wood sound barriers can achieve similar insertion losses to barriers of other materials, including precast concrete and masonry Moreover, wood sound barriers of any of the general design types studied can be designed and built to achieve a 10 dB or more insertion loss.

The human subjects testing program has concluded that any of the wood sound barrier design types studied (with the possible exception of the shadowbox design) can be configured to be generally acceptable to the public, from the perspective of a driver on the road served by the sound barrier, and the perspective of an adjacent property owner. The study also furnished indications that wood barriers may be preferred in certain environments to barriers of similar configuration built of harder materials such as precast concrete

The construction and testing of a prototype will furnish valuable information on the susceptibility of wood sound barriers to cracking and the influence of these cracks on acoustic performance, and will help in broadening the conditions under which insertion loss measurements can be made on existing sound barriers

The continuing study will result in a development of a prototype system of wood sound barriers offering a variety of configurations and constructible with a variety of wood materials.

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Shear Strength of Wood Beams

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Abstract

Experimental shear strength research conducted cooperatively with the USDA Forest Service, Forest Products Laboratory; Washington State University; and the Federal Highway Administration on solid-sawn beams is summarized in this paper. Douglas Fir, Engelmann Spruce, and Southern Pine specimens were tested in a green condition to determine shear strength in members without checks and splits. Sizes tested ranged from nominal 51 by 102 mm to 102 by 256 mm. Additional tests were conducted on air-dried solid-sawn Douglas Fir and Southern Pine specimens. A three-point loading setup investigated the effect of splits and checks on shear strength and a five-point loading setup investigated drying effect on beam shear. Based on the experimental tests, the following are concluded: (1) shear strength of green solid-sawn without splits varies with size and may be characterized using a shear area or volume parameter (2) air-dried Southern Pine shear strength free of splits is equivalent to that for Southern Pine glued-laminated timber; (3) tests on seasoned Douglas Fir and Southern Pine gave mixed results on the effect of splits and checks; and (4) fracture mechanics predictions of the shear strength of artificially split Southern Pine were conservative.

Keywords: Shear strength, beams, design, size effect, Engelmann Spruce, Douglas Fir, Southern Pine, fracture mechanics, splits, checks.

Introduction

Shear design values for solid-sawn structural members are currently derived from small clear, straight-grain specimens (ASTM 1995a). Under typical conditions, wooden beams and columns sometimes develop splits and checks (Fig. 1). These splits and checks are a result of drying as the member equilibrates to the surrounding moisture condition or from repeated wet/dry moisture cycling that may be encountered in exposed timber bridge stringers. Because of the placement of the member within a structure and the local climate, the occurrence and degree of splitting are varied and unpredictable. Published shear design values (AFPA 1991b) account for this uncertainty by assuming a worst case scenario-a beam that has a lengthwise split at the neutral axis. If the design engineer is confident that a member will not split lengthwise, then the design shear value may be doubled.

This approach may lead to an inefficiently designed beam. To increase design accuracy for shear strength, typical beams, rather than small, clear specimens, must be studied. Structural members may or may not contain splits or checks; therefore, an understanding of the shear strength of both unsplit/unchecked and split/checked beams is critical to the design process.

Figure 1—Deeply checked structural timber beams and columns.

The overall purpose of this research was to improve the shear design criteria as it applies to wooden beams. Specific objectives were to

- Develop a beam shear strength database for different solid-sawn wood species;
- Determine the unsplit, unchecked shear strength for wood beams of different sizes and if strength varies with size;
- Determine if the solid-sawn shear strength for unchecked, unsplit beams is similar to that for glued-laminated timber; and
- Determine the effect of checks and splits on beam shear strength.

These objectives were met through an experimental testing program and analysis of the experimental results.

Background

Two approaches based on different failure criteria have historically been used for studying the shear strength of wood beams: (1) a classical approach based on the strength of an unsplit member and (2) a fracture mechanics approach based on the strength of a split or checked member.

Unsplit Wood Shear Strength

In the past, most shear research focused on the small, clear strength for various species using the standard ASTM shear block test (ASTM 1995a). Alternative shear test procedures have been proposed (Radcliffe and Suddarth 1955), but the shear block test is still the accepted method for determining wood shear strength values. However researchers have questioned the applicability of shear block information to predict the actual strength of wood beams.

Huggins and others (1964) found that beam shear strength and ASTM D143 shear strength were different and that beam shear strength depends on the shear span, defined as the distance from the support to the nearest concentrated load. A series of Canadian studies investigated the effects of member size on shear strength. Several of these studies experimentally investigated shear strength using simply-supported beams (Longworth 1977, Quaile and Keenan 1978). Foschi and Barrett (1976, 1977) approached shear strength with Weibull's weak link theory. They showed that shear strength varies with beam geometry and loading. Their work is the basis for the size effect relationship in the Canadian building code.

For the past 10 years, the Forest Products Laboratory has increased its research focus on beam shear. Soltis and Gerhardt (1988) summarized and reviewed existing literature on shear research. Rammer and Soltis (1994) investigated shear strength with a five-point loading setup for glued-laminated members. Leicester and Breitinger (1992) investigated beam shear test configurations. All this activity focused on determining the unsplit, unchecked beam shear strength. Research currently underway is addressing the effects of splits and checks after seasoning on shear strength.

Shear Failures In Actual Structures

Practitioners and scientists have long been interested in the effect of splits and checks on residual shear strength. A bulk of the early research on the effects of splits and checks in timber members came from the evaluation of stringers removed from railroad bridges. Railroad engineers are concerned with the proper time to replace the member because of strength loss as a result of checking, splitting, and deterioration.

The Santa Fe railroad system investigated the condition of timbers removed from a 35-year-old bridge in Arkansas and two 20-year-old bridges in Oklahoma and Arizona. Strength of these timbers was compared with the strength of four virgin timbers to determine strength loss. At the time of testing, all in-service timbers had moisture content levels less than 12% and the virgin timbers had a moisture content level of approximately 17%. Testing consisted of third-point loading on a span-to-depth ratio between 10 and 11. All members showed signs of checking or splitting and some had signs of deterioration. Of the 25 beams tested, 20 experienced shear failures that were influenced by checks. Maximum shear strength loss was 72% for the timbers from Arizona and approximately 40% to 50% for the timbers from Oklahoma and Arkansas (Santa Fe System 1921).

McAlister (1930) reported on the strength tests of 12 untreated 203- by 305-mm Douglas Fir timbers removed from a bridge with 53 years of service life. His experiments included bending and compression tests on timbers and small, clear specimens. Of the 12 specimens tested in three-point bending, five failed in horizontal shear at a span to depth ratio of 10 and a 14% moisture content. McAlister attributed the shear failures to the degree of checking in the timbers.

Newlin and Heck (1934) tested the residual strength often 203- by 457-mm Douglas Fir stringers removed from a bridge that had been in service for 23 years. Two end-matched specimens were fabricated whenever possible from the original specimens, which were greater than 9.14 m long. One specimen was tested on a 5.79-m span and the other on a 4.57-m span. Seven of the ten 5.79-m specimens failed in horizontal shear, and all the 4.57-m specimens failed in shear. They noted that all shear failures occurred at checks, splits, or bolt holes. Small, clear specimens taken from the material showed no effects of service life, unlike the large timbers.

Wood (1954) investigated the residual strength of two 101-year-old Eastern White Pine floor beams. Each beam was approximately 254 by 254 mm and was tested on a 3.96-m span in three-point loading. One beam failed in bending, and the other failed in horizontal shear along a plane containing deep checks.

In all these cases, the failure mode in the existing timber members tended to be governed by shear if a deep check or split was present. Checking and splitting were a result of drying or cyclic environmental changes over the life of the member.

Experimental Shear Failures

Documented shear failures were noted in experimental programs as early as 1912. Cline and Heim (1912) summarized a large experimental testing program to determine the mechanical properties of 11 wood species. Third-point bending specimens ranged in size from nominal 51 mm by 51 mm by 0.46 m to 203 mm by 406 mm by 4.6 m in a wet and air-dried condition. Shear failures were noticed in both the green and air-dried specimens. As the specimen size decreased, so did the percentage of shear failures. In general, the air-dried members had a higher percentage of shear failures at each size when compared with their green counterparts.

In a study to determine the mechanical properties of Alaskan wood, Markwardt (1931) tested Sitka Spruce and Western Hemlock timbers in a green and air-dried condition. Members were tested in third-point loading at a span-to-depth ratio of 11.25. Of the green material tested, 30% of the Sitka Spruce and 24% of the Western Hemlock timbers failed in shear. For the air-dried specimens, 50% of the Sitka Spruce and 66% of the Western Hemlock timbers failed in shear. Markwardt observed that the Sitka Spruce material was of a higher grade than the Western Hemlock, because the Sitka Spruce members were cut from larger diameter trees with few defects. Western Hemlock members were cut from small-diameter trees that contained heart rot, shakes, and decayed knots and might have contributed to the increased occurrence of shear failures.

To evaluate the effects of checks, Newlin and others (1934) conducted bending tests using a built-up beam made of Sitka Spruce. Additionally, they proposed a theory to explain the effect of checks or splits, which is incorporated into current design standards (AREA 1991, AFPA 1991a), by the following, for a concentrated load

$$V = \frac{P(\ell_c - x)\left(\frac{x}{d}\right)^2}{\ell_c\left[2 + \left(\frac{x}{d}\right)^2\right]} \qquad (1)$$

In this theory, known as the two-beam theory, the length and depth of checks are not considered, only the position x of the load, P from the support, beam depth d and the clear span ℓ_c are relevant. Researchers have since shown that the underlying assumptions of this theory are incorrect (Keenan 1974, Soltis and Gerhardt 1988).

Norris and Erickson (1951) conducted a pilot study on the effect of splits on shear strength. They developed a theory based on the assumption that the stress concentration at the tip of the split is approximated by an unknown function that relies on the split length to beam depth ratio. This function can only be determined empirically from test data. Fifteen tests with two different loading patterns were conducted using Sitka Spruce. Based on these tests, the equation developed by Norris and Erickson to explain the effects of splits is

$$\frac{\tau_c}{\tau_m} = 0.674\frac{d}{a}\sqrt{\frac{a-c}{c}} - 0.053 \qquad (2)$$

where τ_c is the shear stress at the neutral axis; τ_m is the maximum shear stress; a is the position of the concentrated load; d is the beam depth; and c is the length of the split.

Based on a survey of glued-laminated timber bridges, Huggins and others (1964) conducted a study on the effect of delamination on the static and repeated load strength for glued-laminated beams. After testing 175 small glued-laminated beams, of which 115 had

simulated splits or delamination, they concluded that shear span influences strength and delamination reduces ultimate strength. Additionally, they stated that shear strength is less under repeated loading than static loading, based on the 48 beams tested.

Fracture Mechanics

One method to evaluate the strength of a split, checked, or cracked beam is fracture mechanics. Fracture mechanics evaluates the state of stress at the end of a crack for three load cases. Mode I is an opening displacement; mode II is a sliding displacement; and mode III is a tearing displacement (Fig. 2).

Wood fracture was first investigated by Porter (1964). Since Porter's first study, wood fracture investigations have generally focused on mode I fracture with some limited studies on modes II and III fracture. A problem with mode II and III investigations is the lack of a standard test procedure to determine fracture properties. Recently, efforts have been made to standardize a test procedure for mode II fracture. General details of the use of fracture mechanics in wood research is summarized by Valentin and others (1991).

Barrett and Foschi (1977) numerically analyzed the influence of beam splits under concentrated and uniform loading. Based on their analysis, the following were developed to express the mode II stress intensity factor K_{II}:

$$K_{II} = \tau\sqrt{\pi a}H \qquad (3)$$

where τ is the shear stress in MPa; a is the split length; and H is a nondimensional factor that characterizes the loading and beam geometry. For concentrated loading, H takes the following form:

$$H = \frac{A + B\left(a/s\right)}{\sqrt{a/s}} \qquad (4)$$

Figure 2—Three modes of loading.

where A and B are functions depending on a/s and s/d, with s being distance from the load to the support and d the beam depth. Using Equation (3), Barrett and Foschi determined the critical stress intensity factor K_{IIc} value for select structural, No. 1, and No. 2 Western Hemlock.

Murphy (1979) used a boundary collocation method to develop a simplified equation to evaluate the effects of beam splits under concentrated and uniform loading. His equation for concentrated loading is

$$K_{II} = \left[-2.785\left(\frac{a}{d}\right) - 0.731\right]\frac{R}{b\sqrt{d}} \qquad (5)$$

where R is the support reaction nearest the split; a is the split length; d is beam depth; and b is the width of beam. Murphy used the work of Norris and Erickson (1951) to validate Equation (5) for Sitka Spruce beams.

Equations (5) developed by Murphy and (3) by Barrett and Foschi are approximately equivalent for all sized beams.

The previous two studies focused on end-split beams; however, a majority of actual defects are checks that are classified as a mode III fracture problem. Murphy (1980) applied mode III fracture mechanics to predict the effect of checks on beam strength. Correcting Sih's (1964) mode III solution, Murphy developed an isotropic two-dimensional expression for mode III fracture and validated it with Newlin and others (1934) data. Murphy stated that this expression could not explain the effects of shear span. Therefore, he developed an empirical expression to address this deficiency.

In the fracture research previously discussed, the focus was to determine the applicability of fracture mechanics to explain wood failure for simulated splits. In actual structural members, the geometry of the crock front is highly irregular. Sometimes the beam is completely split but more often the beam is checked on one or both sides. Further investigation into the application of fracture mechanics is needed to explain the effect of splits and checks.

Test Program

An investigation of shear strength is currently underway through a cooperative study with the USDA Forest Service, Forest Products Laboratory; Washington State University; and the Federal Highway Administration. This research was undertaken to investigate the green, unchecked shear strength, and the seasoned (checked or split) shear strength of solid-sawn beams. Brief descriptions of the procedures are discussed.

Table 1—Nominal size and number of beam shear specimens used in this investigation.

	Green unchecked material			Dry seasoned material			
				Southern Pine			
Specimen (mm)	Douglas Fir	Southern Pine	Engelmann Spruce	5-point bending	3-point bending	3-point bending[a]	Douglas Fir
51 by 102	40	56	57	60	60	80	40
51 by 203	—	42	40	30	30	30	—
51 by 254	40	—	—	—	—	—	40
102 by 203	40	30	30	59	59	—	40
102 by 305	20	25	30	29	30	32	20
102 by 356	20	30	30	30	30	30	20

[a]Simulated splits a 0.5*d*, *d*, and 1.5*d*.

Green Shear Strength

Douglas Fir, Southern Pine, and Engelmann Spruce specimens with nominal sizes ranging from 51 by 102 mm to 102 by 356 mm were tested to determine unchecked beam shear strength (Table 1). All specimens had moisture content levels of 20% or more. A total of 160 Douglas Fir, 183 Southern Pine, and 187 Engelmann Spruce beams were tested.

A two-span, five-point loading test, with each span length equal to five times the member depth, was selected to produce a significant percentage of beam shear failures. This test setup had been successfully used to create shear failures by Langley Research Center (Jegly and Williams 1988), Purdue University (Bateman and others 1990), and the Forest Products Laboratory (Rammer and Soltis 1994). Information recorded included maximum load, type and location of failures, material properties, beam geometry, moisture content, and specific gravity. Further details of the Douglas Fir testing are published by Rammer and others (1996) and the Southern Pine and Engelmann Spruce testing are published by Asselin (1995).

Dry or Seasoned Shear Strength

Only Douglas Fir and Southern Pine specimens were studied in a dry or seasoned condition at an average moisture content of 12%. Nominal specimen size ranged from 102 to 102 mm to 102 by 356 mm for both species (Table 1). All Douglas Fir specimens contained natural splits and checks after 1½ years of air drying and were tested in a single-span, three-point loading setup with a center-to-center span length of five times the member depth. A three-point configuration was used to locate the split in the high shear force region.

Three different tests were conducted on the Southern Pine specimens that were air-dried for 1 year before conditioning to 12% moisture content (Table 1). First, a five-point loading setup was used to determine dry shear strength. Maximum shear force occurs between the load points; therefore, only checks will influence the results as splits are predominantly located at the ends of the beam. Second, a three-point loading setup, with a center-to-center span length of five times the member depth, investigated the influence of natural checks and splits on shear strength. Finally, a three-point loading setup with saw kerfs at lengths of 0.5 *d*, *d*, and 1.5 *d* was conducted to examine the effects of manufactured defects of known size on shear failures.

Details of the Southern Pine experiments are given by Peterson (1995), and Douglas Fir details will be published in a USDA Forest Service research paper by Rammer.

Shear Block Tests

Small, clear ASTM D143 shear block specimens were cut in all the studies from each specimen after failure to benchmark the results to published shear strength values. Two shear block specimens were tested from the green, unchecked beam specimens. One specimen was tested at the moisture condition of the beam and one at 12% moisture content. Only one shear block specimen at 12% moisture content was tested from the air-dried, seasoned beam specimens.

Results

Green Shear Strength

Not all of the five-point loading specimens failed in a shear mode; a significant number failed in tension or from local instability. Therefore, true shear strength is best estimated by application of censored statistics. Censored statistics techniques were discussed and applied by Rammer and others (1996) to adjust the green Douglas Fir results. This same technique was applied to the green Southern Pine and Engelmann Spruce data. Estimated true shear strength values and coefficients of variation for these two species are listed in Table 2.

Table 2—Estimated mean and coefficient of variation (COV) green data considering censored data.

Size (mm)	Engelmann Spruce Shear strength (MPa)	Engelmann Spruce COV (%)	Southern Pine Shear strength (MPa)	Southern Pine COV (%)
51 by 102	8.52	20.9	10.17	8.2
51 by 203	8.13	29.1	7.86	22.0
102 by 203	7.20	19.7	7.10	9.1
102 by 305	4.34	17.0	5.94	11.6
102 by 356	3.96	13.4	5.12	18.7

The size effect for the different species is compared by plotting the ratio of estimated mean beam shear strength to mean ASTM shear block strength versus either shear area or volume (Fig. 3). In these plots, the beam and ASTM shear block strength values are not adjusted for moisture content or specific gravity. In addition, the mean beam shear strength and the 80% mean confidence limits are indicated to show the potential variability in the mean results. In Figure 3, the relative shear strength ratio increases with a decrease in the shear area or volume parameter. These trends are similar to glued-laminated beam shear results (Rammer and Soltis 1994, Longworth 1977). Plotted lines represent empirical relationships that relate beam shear strength to shear area (Rammer and Soltis 1994) and volume (Asselin 1995). In both cases, the curve predicts the means of the large members well, but underestimates the estimated average values for the small beams. This under estimation is a consequence of performing a regression analysis of data that only failed in shear and not considering the censored nature of the data. In almost every case, the empirical curves are conservative.

Figure 3—Five-point beam shear to ASTM shear block ratio versus beam size: (top) shear area, (bottom) beam volume.

Seasoned Five-Point Beam Test

Air-dried Southern Pine was tested in a five-point loading setup to determine the dry shear strength. Drying effects are most noticeable at the end of a beam; therefore, the five-point configuration results are influenced only by checks in the middle portion of the beam and should give a good approximation of the dry shear strength. Censored statistical techniques were again used to estimate the mean and coeffecient of variation of the air-dried Southern Pine (Table 3). Mean values for solid-sawn and glued-laminated (Rammer and Soltis 1994) Southern Pine beams and the 80% mean confidence levels are plotted in Figure 4. Comparison of the air-dried solid-sawn results with previously tested glued-laminated Southern Pine results indicates similar trends, but the solid-sawn material is slightly lower and more variable as a result of checking effects.

Table 3—Estimated mean and coefficient of variation (COV) 12% moisture content Southern Pine considering censored data.

Size (mm)	Shear strength (MPa)	COV (%)	Beam D/G[a] ratio	ASTM D/G[b] ratio
51 by 102	12.76	13.1	1.25	1.30
51 by 203	10.71	15.6	1.36	1.35
102 by 203	11.27	20.9	1.59	1.58
102 by 305	8.33	20.0	1.40	1.69
102 by 356	7.39	8.5	1.44	1.86

[a]Beam dry/green ratio based on Peterson (1995).
[b]Shear block dry/green ratio based on Asselin (1995).

Figure 4—Comparison of seasoned solid-sawn and glued-laminated Southern Pine by five-point beam test.

Typically the dry/green shear strength ratios for the individual Southern Pine species range between 1.45 and 1.75 (ASTM 1995b), and Kretschmann and Green (1994) recently found a 1.47 increase for the general Southern Pine classification. An estimated dry/green ratio based on the estimated means for the Southern Pine five-point specimen at each size was calculated with the shear block dry/green ratio found by Asselin, as shown in Table 3. Beam shear dry/green ratios tended to be smaller than values published in ASTM (1995b), but similar to dry/green ratios found by Asselin in shear blocks cut from smaller beam sizes. In the 102- by 305-mm and 102- by 356-mm sizes, the beam dry/green ratios are at the upper bound of the acceptable ASTM values and 20% lower than values develop from tested shear blocks.

Seasoned Three-Point Beam Test

Both Southern Pine and Douglas Fir beams with natural defects (splits and checks) were tested in three-point loading to determine the effects of both splits and checks on member strength. Of the 209 Southern Pine beams tested, 73 failed in shear; of the 160 Douglas Fir beams tested, 76 failed in shear. It was difficult in both studies to predict which split or check was critical prior to testing so that critical pre-test information could be gathered. After testing, beams were split open and the amount of lost area was calculated after testing. Lost area was determined by observing the transition zone between the glossy weathered to newly formed dull surfaces.

To show the effect of splits and checks on strength, shear strength versus lost area are plotted in Figure 5. Southern Pine beams showed little decrease in strength

Figure 5—Three-point shear strength results for beams failing in shear of seasoned (a) Douglas Fir and (b) Southern Pine.

as a result of splitting or checking. Douglas Fir beams, on the other hand, visually showed a stronger decreasing tend with increasing lost area. It also appears that the Douglas Fir members had a higher degree of splitting and checking.

Based on research to be published later in a Forest Products Laboratory report, Douglas Fir material checks dominated the 102-mm specimens; in contrast, splits dominated the shear failures in the 51-mm specimens. As indicated by Murphy (1980), the influence of checks on beam shear strength, characterized by mode III fracture, occurs when checks have depths greater than 15% of the cross sectional width.

Three-Point Beam Test With Saw Kerfs

Peterson's (1995) third testing series evaluated the effects of saw kerfs on shear strength. Application of a

Figure 6—Comparison of saw kerf Southern Pine beam with Equation (5). Solid line represents 51- by 102-mm strength prediction. Dashed line represents 102- by 356-mm strength prediction.

saw kerf increased the percentage of shear failures from 35% in the seasoned material to 68% in the cut specimens. To compare fracture mechanics approaches, a critical mode II (K_{IIc}) stress intensity property is needed. Kretschmann and Green (1992) determined the K_{IIc} for Southern Pine at several moisture levels using a center-split beam. At 12% moisture content, the K_{IIc} value is 2060 $kN \cdot m^{-3/2}$. Using this value of K_{IIc} in Murphy's Equation (5), mode II fracture yields a prediction for the shear strength of the beams. For this test configuration, Murphy's Equation (5), and Barrett and Foschi's Equation (3), yield similar results. Figure 6 compares the experimental and predicted shear strength to the split length to beam depth ratio.

The predicted values for the split beam shear strength were conservative at all sizes. This conservatism was probably because the derived solutions assume traction forces were not applied over the crack surfaces. Peterson (1995) observed crack closure and contact as the load was applied. This action could develop surface traction and frictional forces along the crack. To correctly model this type of fracture, crack closure should be considered.

Concluding Remarks

Several studies were conducted to determine the shear strength of wood beams. These studies were conducted on various member sizes of Douglas Fir, Engelmann Spruce, and Southern Pine beams. As a result of this research, the following are concluded.

- Unsplit, unchecked shear strength for all species varied with beam size and had similar trends after estimated beam strength was divided by ASTM shear block values to normalize material effects. An empirical expression based on both shear area and volume gave conservative results at smaller beam sizes after censored statistics techniques were applied.

- Air-dried Southern Pine material tested in a five-point loading configuration gave similar results to Southern Pine glued-laminated shear strength data. This is likely due to the lower incident of splits and checks as a result of drying in the region of maximum shear in a five-point configuration. The application of dry/green ratios for shear strength design should be further investigated. For larger-sized members, dry/green ratios developed from the beam shear tests were at least 20% less than ASTM dry/green ratios.

- Tests on naturally split and checked beams showed mixed results for Southern Pine and Douglas Fir specimens. Southern Pine specimens showed little change with increasing lost area. In contrast, Douglas Fir specimens indicated a decreasing trend with an increase in defected area. In both materials, shear failures were difficult to replicate and these tends are based on limited sample sizes. Further testing is needed to better conclude the effect of natural defects.

- Finally, a comparison of shear strength obtained on artificially split Southern Pine beams with predicted strength based on existing mode II fracture theories revealed the predictions are conservative.

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Effects of Treatment, Incising, and Drying on Mechanical Properties of Timber

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Abstract

The design demands of timber construction used in transportation structures, such as bridges, require extended durability and reliability. As such, preservative-treated timber, rather than naturally durable timber, is most often required by code. Design demands also require minimal strength loss during treatment or during the service life of the structure. This report critiques the literature on treatment effects and recommends treating procedures for controlling reductions in strength properties. Particular emphasis is given to the effects of pretreatment drying, incising, treatment processing factors, and post-treatment drying practices.

Keywords: Mechanical properties, preservative treatment, creosote, waterborne preservatives, air drying, kiln drying, timber, lumber, incising.

Introduction

Wood in the standing tree has high moisture content, usually in excess of 75 percent. Green wood must be seasoned (i.e., dried) before it can be adequately treated with preservative. With dimension lumber, smaller timber, and Southern Pine poles, pretreatment drying is often achieved using standard air- or kiln-drying techniques commonly used in the lumber industry. With the largest timbers, refractory poles, and/or piles, pretreatment drying is often performed within the treating cylinder immediately prior to treatment. Several researchers have studied the magnitude and severity of these pretreatment drying practices.

Drying of green wood decreases warp in service by removing much of the moisture and reduces transportation costs by reducing total weight. In addition, strength generally increases as wood is dried from the green condition to moisture contents below the fiber saturation point. Yet, strength can also be negatively affected by the temperatures used in the drying process and by the introduction of other materials, such as chemical preservatives or fire retardants.

The strength and stiffness of wood decreases when heated and increases when cooled. This temperature effect is immediate and, for the most part, recoverable for short heating durations. If wood is exposed to elevated temperatures for an extended time, strength is permanently reduced because of degradation of the wood substance with a corresponding loss in woody material and weight. The magnitude of any permanent effects depends upon moisture content, heating medium, temperature, exposure period, and, to a lesser extent, species and specimen size.

The treatment of wood with waterborne preservatives results in a product with high moisture content, usually well in excess of the fiber saturation point. Accordingly, post-treatment drying is sometimes undertaken to increase the market value of the treated

material by improving dimensional stability, assuring preservative fixation, and reducing shipping weight. Several researchers have shown that the severity of the post-treatment drying process is a significant co-contributor, along with the preservative chemical, to adversely affecting the mechanical properties of the treated and dried material.

This report critiques the literature on treatment effects and recommends treating procedures for controlling reductions in strength properties. Particular emphasis is given to the effects of pretreatment kiln drying and incising, treatment processing, and post-treatment drying practices.

Seasoning Practices in Wood-Treating Industry

In the lumber industry, standard drying practices and the effects of these practices on the mechanical properties of wood are well documented (Forest Products Laboratory 1987, Koch 1985, Rietz and Page 1971, Simpson 1991). These reviews of the research have shown that the negative effects of the initial kiln drying process on strength are negligible at 71°C (160°F) or below. At higher temperatures, however, the effects of the initial kiln drying processes, especially at or above the boiling point of water, can be significant.

In addition to the standard air- and kiln-drying practices used by the timber industry, several other drying methods are practiced by the treating industry. Most notable are two artificial drying methods performed within the pressure-treating cylinder: Boultonizing and steam conditioning. In Boultonizing, green wood is boiled in oil under vacuum. In steam conditioning, green wood is steamed for several hours, followed by a vacuum. In each case, moisture is evaporated from the green wood by either increasing the kinetic energy or reducing the vapor pressure within the retort. These techniques are generally used to partially dry large members prior to treating with oil-type preservatives and were thoroughly described by Henry (1973) and MacLean (1952). Much of the general data on the effects of these accelerated drying methods were summarized by MacLean (1951, 1952, 1953) and Thompson (1980, 1982). Treatment standards were modified to preclude strength losses in treated timber resulting from excessive thermal treatment. The American Wood Preservers' Association (AWPA 1995) Standards have strict limitations for treatment processing for both temperature and duration of temperature. Third-party sanctioned adherence to AWPA Standards will negate any need for design adjustments for strength loss resulting from in-cylinder seasoning.

Air Drying

Air drying and its analogs, such as controlled air-drying and tunnel drying, have been the primary methods used to dry large timbers and other large commodities prior to treatment. As such, air drying represents the standard to which most accelerated conditioning methods are compared. Air drying, if properly done with respect to snickering and stacking, is usually assumed to have no detrimental effects on wood properties.

Kiln Drying

Kiln drying is the most common method for pretreatment drying of softwood dimension lumber, but it is used to a lesser extent with large commodities such as posts, poles, pilings, and timbers. The negative effects of kiln drying are generally observed when the drying temperature exceeds the boiling point of water, commonly known as high-temperature drying. The strength reductions associated with high-temperature drying presumably arise from the hydrolysis of the amorphous carbohydrates by acetic acid liberated from acetyl groups in the wood (Hillis 1975), resulting in depolymerization, which has been correlated to strength loss (Ifju 1964). Elevated temperature and moisture content have been shown to accelerate the production of acid (Hillis 1975) and increase the rate of strength loss (Thompson 1969).

Koch (1985) summarized much of the work on high-temperature drying of dimension lumber. High-temperature drying has only a minimal effect on mechanical properties; modulus of rupture (MOR) varies from +5 to –10 percent and modulus of elasticity (MOE) from +2 to -3 percent in the Southern Pines (Koch 1971, 1976, Yao and Taylor 1979). High-temperature drying has a more severe effect on other softwoods, notably Douglas Fir, Hem–Fir, and Spruce-Pine-Fir; MOR varies from –9 to –21 percent and MOE from +3 to –4 percent (Cech and Huffman 1974, Comstock 1976, Gerhards 1979, Kozlik 1968, Salamon 1963, 1969). The limited data available on the effect of high-temperature drying on the strength of hardwoods indicate similar minimal reductions in mechanical properties—MOR from +1 to –10 percent and MOE from +4 to –4 percent (Gerhards 1983, Laden 1956, MacKay 1976). However, it should be understood that most studies on the effects of kiln drying were conducted at a dry-bulb temperature $\leq$ 116°C ($\leq$ 240ºF). Few, if any, recent studies have investigated initial kiln-drying temperatures from 116°C to 149°C (240°F to 300°F), which are sometimes used to kiln-dry lumber commercially.

Other Drying Methods

Several new drying techniques have recently been applied to wood. These methods include steam drying at or above atmospheric pressure, dehumidification

drying, solar drying, and vacuum drying. Of these methods, the first and last may have application in the treating industry. Southern Pine nominal 2 by 6 (standard 38 by 190 mm) lumber was dried in superheated steam at atmospheric pressure in less time than that required for high-temperature drying with equivalent results in terms of moisture contents and variation (Taylor 1985); no data on mechanical properties were presented. A subsequent study with Southern Pine landscape timbers showed no difference in the treatability of timbers dried using a conventional (82°C (180°F)), high-temperature (118°C (245°F)), or superheated (118°C (245°F)) steam schedule (Barnes and Taylor 1985). Again, no data on mechanical properties were presented.

Pressure steam drying of wood has been reported in a number of articles (McGinnis and Rosen 1984, Oswald and others 1984, Phelps and Cutter 1985, Rosen 1980, 1981, 1985, Rosen and others 1983). In this process, wood is dried in superheated steam above atmospheric pressure. Oswald and others (1984) studied the effects of pressure steam drying on the strength of yellow poplar dried at 116°C to 149°C (240°F to 300°F) for 1 to 30 h. Control samples were kiln dried using the continually rising temperature schedule T11–D4 (Simpson 1991) with a maximum dry bulb temperature of 82°C (180°F). No significant differences in bending properties, compression parallel to grain, or hardness were noted; toughness was slightly decreased.

Vacuum/radio frequency drying is a technique in which green wood is exposed to radio frequency energy while being held under vacuum. Radio frequency drying was found to have no deleterious effects upon mechanical properties (Harris and Taras 1985, Lee and Harris 1984).

Oil-Type Preservative Treatment

Oil-type preservative treatments are generally recognized as imparting superior dimensional stability and weatherability to treated wood products when compared to waterborne preservative treatments because of the nonpolar nature and natural water-repellency of oilbome systems (Barnes and Winandy 1986). Oil-type preservative treatments are also generally recognized as having less effect on mechanical properties than do waterborne treatments because they do not react with the wood cell wall material. The effects of oil-type preservative treatments, such as creosote and creosote-coal tar mixtures, on mechanical properties were extensively studied in the early part of the 20th century (Gregory 1915, Harkom and others 1930, 1931, Hatt 1906, Thunell 1941, Wilson 1930). These studies generally concluded that the effects of treating softwood timbers with oil-type preservative treatments generally resulted in no appreciable loss in stiffness or strength. However, when evaluating the effects of a treatment on strength. it is important to note the moisture content at time of test. Luxford and MacLean (1951) noted that several early investigators tested matched lots of untreated green timbers and compared them to retort-conditioned and treated timbers. The treated, but conditioned, material often had an average moisture content of about 22 percent, with surface moisture content of 17 percent. Thus, it is feasible that these differences in moisture content could erroneously account for the early findings of virtually no difference between semi-dry treated timber and green untreated timber. More recent work involving comparably conditioned materials investigated the effects of creosote treatment on softwood timbers at terrestrial retentions ($\leq$ 192 kg/m^3 ($\leq$ 12 lb/ft^3)) (Luxford and MacLean 1951), softwood piles at very high marine retentions (320 kg/m^3 (20 lb/ft^3)) (Eaton and others 1978, Resch and Parker 1982, and hardwood laminated veneer lumber ($\leq$ 192 kg/m^3 ($\leq$ 12 lb/ft^3)) (Kimmel and others 1994). As a result of these studies, it is now recognized that the effects of treating hardwoods, softwoods, solid-sawn lumber, and laminated veneer lumber with oil-type preservative treatments generally results in small, but measurable, losses in stiffness or average strength of about 5 to 10 percent.

Any loss in strength from treatment with oil-type preservatives can be greatly exaggerated if certain allowable in-retort seasoning parameters are exceeded (for Boultonizing, steam conditioning, etc.) and/or if excessive temperatures or pressures are employed during the actual treating process. The impact of thermal exposure during oil-type preservative treatments was extensively studied (Hunt 1915, MacLean 1927, 1951, 1952, 1953). As a result, specific limitations were implemented in AWPA Standards to minimize these effects (Barnes and Winandy 1986). As long as the processing limitations in AWPA Standards are strictly adhered to, there should be no adverse effect of oil-type preservative treatment on mechanical properties and, thus, no need to adjust design stresses.

Waterborne Preservative Treatment

Waterborne-preservative-treated wood represents a major segment of the forest products industry. In North America, the primary waterborne preservatives are chromated copper arsenate (CCA), ammoniacal copper systems, and amine copper systems.

The metallic oxides, commonly used in waterborne preservative formulations, physically react with the cell wall components by undergoing hydrolytic reduction upon contact with wood sugars. This process, known as fixation, oxidizes the wood cell wall components and may reduce wood strength. The CCA waterborne preservative formulations most commonly used today

are sufficiently acidic to cause cell wall hydrolysis, and, as with any chemical reaction, the addition of heat exacerbates the potential for further strength reduction.

Overall, the post-treatment strength of treated wood is affected by many factors. For treated products, specific questions of when and how much strength is affected are relative to initial material quality and its processing history. The magnitude of treatment effects is related to the composition of the treatment chemicals and the temperature, temperature duration, and temperature–moisture content history of the treating process and kiln drying after treatment.

Currently, few modifications are required to allowable stress design values if material is treated with waterborne preservative within the processing limits of AWPA Standards (AWPA 1995). However, recent research showed that treated materials exposed to impact loads do not experience the same increase in strength as does untreated material (Winandy 1995c). In response, the engineering design community recently adopted a standard design procedure that prohibits the application of the load-duration adjustment (C_D factor) of 2.0 to CCA-treated material exposed to impact loads ($\leq$ 10 s) (AF&PA).

With the exception of this case, strength loss in waterborne-preservative-treated material is minimized or controlled because several AWPA Standards for treating lumber have instituted post-treatment redrying temperature limits to control strength effects. This section will review the technical literature on waterborne preservative treatment-redrying effects and thereby document the reasoning behind the AWPA post-treatment redrying temperature limits.

The overall effects of waterborne preservative treatment appear to be directly related to several key pretreatment, treatment, and post-treatment processing factors:

- redrying temperature
- grade and size of timber
- type of preservative and/or chemical
- preservative retention
- initial kiln-drying temperature

Redrying Temperature

Redrying temperature appears to be the single most decisive processing factor that affects strength. Air drying after treatment and kiln drying at $\leq$ 71°C ($\leq$ 160°F) have been shown to have little practical effect on allowable design stresses (Winandy 1989, Winandy and Boone 1988). Above that level, the higher the redrying temperature the greater the negative effect on mechanical properties (Barnes and Mitchell 1984, Winandy and Boone 1988). However, the magnitude of the effects of post-treatment kiln-drying also depends on the initial kiln-drying temperature and the grade, size, and species of the treated wood.

Kiln redrying does have some advantages over air redrying when the environmental aspects of using preservative-treated wood are critical, such as uses associated with sensitive streams or biotic communities. Controlled kiln redrying significantly decreases the amount and rate of preservative leaching into the soil when the treated wood is placed in service (Boone and others 1995).

Grade and Size of Timber

Although little is known about timber-sized material, the higher grades of dimension lumber appear to be reduced in strength more than the lower grades. Barnes and Mitchell (1984) found that MOE and work to proportional limit were not affected. Average bending strength of the No. 1 and Better lumber was significantly reduced by 8 percent when the lumber was redried at 88°C (190°F) and by 12 percent at 116°C (240°F) when compared to untreated controls. Fifth percentile values were reduced more than the mean—14 percent when the lumber was redried at 88°C (190°F) and 19 percent at 116°C (240°F).

For lower grades (e.g., No. 2) of Southern Pine lumber, CCA treatment was found to significantly reduce average bending strength (Winandy 1989, Winandy and Boone 1988). However, unlike the results of earlier studies, few differences were found between untreated controls and CCA-treated groups in the lower tails of the bending strength distributions. The authors noted that higher redrying temperatures reduced strength over a broader range of the bending strength distribution than did lower redrying temperatures. The higher grades of treated lumber appeared to be reduced in strength more than the lower grade material. The authors concluded that the effect of CCA treatment and redrying on bending strength is related to both the grade and the quality level within that grade. Larger sizes are reduced in strength less than smaller sizes. This is probably related to the surface-to-volume ratio of each treated product. Recent technical reviews have shown that at comparable retentions, large timbers, poles, and piles are generally reduced in strength less than is dimension lumber, which, in turn, is reduced in strength less than are small specimens (Barnes and Winandy 1986; Winandy 1995a).

Preservative Chemical

The relative impact of various waterborne preservative systems is directly related to the system's chemistry and the severity of its fixation/precipitation reaction. Amine– and ammoniacal–copper preservative systems were once considered to have less effect on strength than do acidic copper systems like CCA, but recent studies with ammoniacal–copper systems have shown strength loss comparable to that associated with CCA (Barnes and others 1993, Winandy and Lebow 1996).

The relative impact of various CCA formulations appears to be related to chromium content (Bendtsen and others 1983). Thus, waterborne preservatives are now generally classified by their effects on strength as follows:

(least effect) (most effect)
amine Cu < ammoniacal Cu = CCA-C < CCA–A

Preservative Retention

For the most frequently used waterborne preservative retentions—0.25 to 0.60 lb/ft^3 (4.0 to 9.6 kg/m^3)—there appears to be little relative difference in their effect on strength when the lumber is redried at comparable temperatures (Winandy 1995a). However, the higher retention required for marine use (2.5 lb/ft^3 (40 kg/m^3)) does significantly reduce impact strength (Bendtsen and others 1983). This is why the newest design standards for wood construction (AF&PA 1996) will limit allowable design stresses for preservative-treated wood when the wood is exposed to impact loads (duration $\leq$ 1 s).

Recent work also showed that CCA-treated lumber mechanically tested at $\leq$ 15 percent moisture content experienced a greater loss in strength and experienced this loss over a greater range of the strength distribution than CCA-treated lumber tested at higher moisture content (Winandy 1995b). Although no action has yet been adopted by the standards- or code-writing communities, this result may have serious design implications for the use of CCA-treated materials in arid environments where wood moisture content might be low for long periods of the year.

Initial Kiln-Drying Temperature

While initial kiln drying of Southern Pine lumber at high temperatures of 100°C to 116°C (212°F to 240”F) for short durations apparently has little effect on its structural properties (Koch 1976, Yao and Taylor 1979), high temperature drying does result in more degradation of the cell wall than does drying at lower-temperature kiln schedules. Subsequent treating and redrying of material initially dried at high temperatures cause additional hydrolytic degradation.

Initial kiln drying at 113°C (235°F) resulted in slightly greater reductions in strength throughout the entire bending and tensile strength distributions than did initial kiln drying at 91°C (196°F) when those two sets of lumber were subsequently treated with CCA preservative and redried (Barnes and others 1990, Winandy and others 1992). Because most Southern Pine lumber is initially kiln dried at high temperatures, the implications of these results are significant. If initial kiln-drying temperature and redrying temperatures were unlimited, a sizable reduction in allowable design values would be required. However, if initial kiln-drying were limited to 116°C (240°F) and redrying temperatures were limited to 91°C (196°F), then allowable design stresses for bending and tensile strength apparently would not require adjustment. Accordingly, in 1991, the AWPA lowered the redrying temperature limit for waterborne-preservative-treated material from 88°C (190°F) to 71°C (160°F) in the AWPA Standards to preclude the need to adjust allowable design stresses.

Further, a systematic investigation of post-treatment strength effects resulting from pretreatment high-temperature kiln-drying in the current commercial temperature range of 116°C to 149°C (240°F to 300°F) and their interaction with waterborne preservative treatment effects has not been done. This is a significant gap in our knowledge base.

Incising

Regardless of whether waterborne- or oil-type preservative treatments are used, incising is required by AWPA Standards when treating difficult-to-treat species. Incising is a pretreatment mechanical process in which steel knives are used to make longitudinal incisions into the four sawn faces of lumber or timber. The incisions range in depth from 5 mm (0.2 in.) (for dimension lumber) to 25 mm (1 in.) for large timbers. Incising is also required for treating difficult-to-treat round materials, such as poles and piles. Poles may be incised to a depth of 64 mm (2.5 in.), or even through-bored in some cases. Although incising dramatically improves preservative penetration and distribution, it also reduces strength (Kass 1974, Lam and Morris 1991, Perrin 1978). However, it is generally agreed that this strength loss is beneficial in the long run because the increase in treatability provides a substantial increase in service life when compared to that of unincised, but treated, refractory material. For example, incised nominal 2 by 4 (standard 38 by 89 mm) lumber may be reduced in strength by 10 to 20 percent when compared to unincised and treated lumber of this size. However, if the two pieces of wood are placed in a decay-prone environment, the incised product will substantially outperform the unincised product. The strength of large timbers tends to be less affected by incising (Perrin 1978). The limited technical literature supports a 10- to 20-percent reduction in allowable design stresses for nominal 2-in.- (standard 38-mm-) thick lumber and a 0- to 10-percent reduction for thicker material (Kass 1974, Lam and Morris 1991, Perrin 1978). Additional cooperative research on incised dimension lumber is currently underway at the Forest Products Laboratory and Oregon State University.

Recommendations

In summary, oil-type preservative treatments have little effect on design stresses, if thermal-processing limitations in AWPA Treating Standards are strictly

followed. Waterborne preservative treatments generally reduce the mechanical properties of wood more than do oil-type preservative treatments because waterborne preservative chemicals physically react with the wood cell wall material. This effect is exaggerated when wood treated with waterborne preservative is processed at high temperatures either before, during, or after treatment (e.g., initial kiln-drying, in-treatment drying, post-treatment redrying). Similar to the history of creosote treatments in the early 1950s, these facts about waterborne preservative treatments were recently used to justify temperature limitations in AWPA Treating Standards for pretreatment, treatment, and post-treatment processing to avoid large reductions in strength. As a result, allowable stress design values do not need to be modified for waterborne-preservative-treated material because current AWPA Standards limit treating and redrying temperatures. Only two exceptions exist: for incised material and treated material exposed to impact loads.

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Mechanical Grading of Timbers for Transportation Industry

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Abstract

The grading of structural timbers is currently based on visual assessment of growth characteristics and defects, with properties derived from tests of small clear specimens. More efficient design of timber structures and more efficient use of the nation's forest resource demand more precise assignment of allowable properties than is possible with the visual grading system. In this paper, we summarize available data on the mechanical properties of structural timbers and evaluate an approach for mechanical grading based on nondestructive testing. Potential mechanical grades for timbers are discussed, and preliminary yield data are presented for oak timbers.

Keywords: Grading, mechanical, timber, transportation

Introduction

Timbers are lumber ≥ 127 mm (≥ 5 in.) in the least dimension. Structural timbers are used in a variety applications—from transportation structures such as railroad bridges to timber-frame buildings for farm, residential, and commercial use. At present, the grading of structural timbers is based on visual assessment of growth characteristics and defects, with allowable properties derived from tests of small clear specimens. For those not intimately familiar with the grading process, timber grading is very confusing. Unlike the descriptions for standard 38-mm- (nominal 2-in.-) thick visually-graded dimension lumber, grade descriptions for timbers are not standardized across all species. Descriptions are not even identical for the same species—grade name combinations if graded by different agencies. For most species, the grade descriptions for timbers to be used in bending (beams and stringers) are different from those to be used to support axial loads (posts and timbers). The classifications of "beams and stringers" and "posts and timbers" are based on cross-sectional dimensions. A comparison of design stresses between these classifications reveals that it is possible to increase certain allowable properties simply by changing the cross-section. This is an anomaly created by having a definition based on size.

Property assignment procedures for visually graded timbers may be very inefficient for some species. For example, the commercial grouping Red Oak is composed of nine species (Table 1), each with a different strength and stiffness value. Individual oak species cannot be identified by visual inspection of the sawn timbers. One species, southern red oak, has mechanical properties significantly lower than those of the other species. Thus, ASTM D245 (ASTM 1995) procedures limit some properties of the Red Oak grouping to those of southern red oak. Mounting a large-scale testing program on full-sized timbers would

Table 1—Mean clear wood properties of species in the Red Oak group.

Species	Modulus of rupture (MPa)	Modulus of rupture ($\times 10^3$ lb/in^2)	Modulus of elasticity (GPa)	Modulus of elasticity ($\times 10^6$ lb/in^2)
Black oak	95.8	13.9	11.3	1.64
Cherrybark oak	124.8	18.1	15.7	2.28
Laurel oak	86.9	12.6	11.6	1.69
Northern red oak	98.6	14.3	12.5	1.82
Pin oak	96.5	14.0	11.9	1.73
Scarlet oak	120.0	17.4	13.2	1.91
Water oak	106.2	15.4	13.9	2.02
Willow oak	100.0	14.5	13.1	1.90
Southern red oak	75.2	10.9	10.3	1.49

not remove this barrier because the weakest species would still control the properties of the commercial grouping. Furthermore, the general adjustment factor applied to all strength properties is 10 percent greater for hardwoods than it is for softwoods. Thus, everything else being equal, hardwoods will have allowable properties 10 percent lower than those of softwood species. This situation encourages overly conservative structural designs and may prevent producers from receiving full value for their local resource.

Mechanical grading was developed in the early 1960s for standard 38-mm- (nominal 2-in.-) thick lumber of softwood species to address limitations of the visual grading system when applied to engineered structures. Research has shown that mechanical grading can provide more precise property assignments for this size of lumber from hardwood species (Green and others 1993). However, mechanical grading procedures have not been developed for timbers produced from either softwood or hardwood species. The objectives of this paper are to summarize an ongoing program at the Forest Products Laboratory (FPL) to develop technical justification and procedures for the mechanical grading of structural timbers and to discuss potential advantages and limitations of these procedures.

Background

Mechanical Grading of Standard 38-mm (Nominal 2-in.) Lumber

Mechanical grading has been used commercially for standard 38-mm- (nominal 2-in.-) thick lumber since the early 1960s (Galligan and others 1978). Several procedures are available for machine grading of lumber. The traditional procedure of machine stress rating (MSR) relies upon the relationship between strength and stiffness to establish grade boundaries. Sorting efficiency for lumber grades is further controlled by visual restrictions on allowable edge-knot sizes (Galligan and others 1978). Qualitying lumber for an MSR grade is an iterative procedure in which deflection limits are set for individual grades and the resulting output is tested for conformance to claimed properties. For each MSR grade, six allowable properties must be assigned: bending strength (F_b), modulus of elasticity (E), ultimate tensile stress (UTS) parallel to grain (F_t), ultimate compressive stress (UCS) parallel to grain (F_c), shear strength parallel to grain (F_v), and compression strength perpendicular to grain (F_{cp}). A specific gravity (SG) value is also associated with each grade (AFPA 1991). Traditionally, modulus of elasticity (MOE) and modulus of rupture (MOR) values are used to establish the grade cut-off settings on an MSR machine. These settings and the visual quality criteria are used to obtain the F_b and E values. Other property values are determined either as a function of MOR (UTS and UCS) or by the clear-wood procedure of ASTM D245 and D2395 (F_v, F_{cp}, SG) (ASTM 1995). The UCS value is obtained using the following relationship (Green and others 1993):

$$\text{UCS/MOR} = 2.061(1/\text{MOR}) + 0.338$$
$$\text{if MOR} \geq 2.835 \times 10^3 \text{ in}^2 (\geq 19.55 \text{ MPa})$$

or

$$\text{UCS/MOR} = 1.06$$
$$\text{if MOR} < 2.835 \times 10^3 \text{ in}^2 (< 19.55 \text{ MPa})$$

The F_c values are obtained by dividing UCS by a factor of 1.9 (ASTM D2915, ASTM 1995). The 1.9 factor accounts for long-term in-service loading plus a factor of safety. The traditional relationship between UTS and MOR for MSR lumber is a discontinuous function (Green and Kretschmann 1991). The allowable F_t value is calculated by dividing the UTS value by 2.1 (ASTM D2915).

All of these relationships were developed using test results for softwood species (Green and Kretschmann 1991). The MSR procedure was recently extended to standard 38-mm- (nominal 2-in.-) thick lumber of hardwood species (Green and others 1993). Accomplishing this extension required research to show that the property relationships assumed to apply to softwood dimension were also applicable to hardwood species. First, we needed to demonstrate a significant relationship between MOR and MOE—of which there was no real doubt. Then, we had to establish relationships between other properties. For example, tests established that the relationship assumed between UCS and MOR of softwood species was also applicable to oak and maple. Finally, we needed to grade lumber by the MSR process at a mill and to verify through testing that the assumed

Table 2—Properties of lowest mechanical grade for standard 38-mm- (nominal 2-in.-) thick lumber compared to visual equivalents.[a]

Grade	Red Oak		Mixed Maple	
	F_b	E	F_b	E
MSR	1,650	1.4	2,100	1.8
Select Structural	1,380	1.4	1,300	1.3
No. 1	990	1.3	942	1.3
No. 2	960	1.2	910	1.1
No. 3	570	1.1	500	1.0

[a]Grade names represent allowable bending stress F_b expressed in pounds per square inch and allowable MOE in million pounds per square inch. 1 lb/in^2 = 6.894 kPa. Green and others 1993.

properties were actually being obtained. With Red Oak standard 38-mm by 210-mm (nominal 2- by 9-in.) lumber, the mechanical grading procedure was able to demonstrate that most of the lumber graded as No. 3 or better by visual grading procedures actually had properties equal to or better than those assigned to Select Structural (Table 2) (Green and others 1993). Even better results were demonstrated for Mixed Maple standard 38-mm (nominal 2-in.) lumber.

Historical Data on Property Relationships for Timbers

The most extensive data on the mechanical properties of timbers was obtained early in the 20th century. A bulletin by Cline and Heim (1912) contains the results of 2,760 tests of softwood species ranging from standard 89- by 89-mm (nominal 4- by 4-in.) to standard 286- by 286-mm (nominal 12- by 12-in.). Tests were conducted in both bending and compression parallel to grain and for both green and airdried material. Although it is not possible to use these data to assess the properties of lumber graded by today's standards, the data illustrate that there is a significant relationship between MOR and MOE for timbers. Some pieces called "green" in Cline and Heim's bulletin were actually quite dry, and some of the "airdried" pieces had moisture content over 19 percent. By first discarding all airdried members with moisture content over 19 percent and all green members with moisture content less than 28 percent, statistically significant MOE–MOR relationships are obtained for a wide range of species and sizes (Table 3). Some historical data from other sources are also included in Table 3, as well as results from two recent studies on Southern Pine and Mixed Oak. Although the coefficient of determination *(r^2)* values are often less than the value of 0.5 anticipated for standard 38-mm- (nominal 2-in.-) thick dimension lumber, they would not be unusual given the small sample sizes used with a number of the species and the lack of lower quality material tested in many historical studies.

Current Research on Mechanical Grading of Timbers

Establishment of Property Relationships

Research at FPL and West Virginia University (WVU) has focused on the properties of structural timbers. Work at FPL has focused on Southern Pine and Mixed Oak, and that at WVU on Eastern Hemlock and Mixed Maple. The FPL research will be discussed here. Just as was done with standard 38-mm- (nominal 2-in.-) thick hardwood lumber (Green and others 1993), it was necessary to establish relationships between properties that could be used to assign allowable design values to structural timbers. Bending and compression parallel-to-grain tests were conducted on 200 dry Southern Pine 6 by 6 timbers (Green and Kretschmann, submitted for publication) and 150 Mixed Oak 7 by 9 timbers (Kretschmann and Green, submitted for publication).[1] These data yielded relationships between MOR and MOE that could be used for initial proposals of grade boundaries (Table 3).

In producing MSR timbers, it is necessary to have a method for measuring MOE on every piece. Pulse-echo, longitudinal stress wave techniques provide an alternative that could be applied in a mill (Schad and others 1995; Kretschmann and Green, submitted for publication). In our research, we obtained the stress wave measurement of MOE (PE MOE) by placing a double-threaded screw in the end of the timber and attaching an accelerometer. Pulse energy was introduced to the specimens through a hammer impact on the opposite end. Echo waves were recorded to obtain a time between peaks on an oscilloscope. The PE MOE was calculated using the speed of the sound wave, *c*, and density, ρ, of each timber using the following equation:

$$\text{PE MOE} = K\, c^2 \rho \qquad (1)$$

where K is a conversion constant.

The MOR–MOE relationships for our Mixed Oak and Southern Pine timbers are given in Table 3. The r^2 value for the oak is about the value expected for standard 38-mm- (nominal 2-in.-) thick dimension

[1]Note: 6 by 6 timber refers to nominal 6- by 6-in. (standard 140- by 140-mm) timber; 7 by 9 timber refers to nominal 7- by 9-in. (standard 165- by 210-mm) timber.

Table 3—MOE-MOR relationships for timbers.[a]

Species	Size	Moisture content (percent)	N	MOR = A + B(MOE)[b] A	B	r^2	Ref[c]
Red Oak	7 by 9	Green	220	14.3	2.553	0.51	1
Eastern Hemlock	6 by 8	Green	80	4.6	3.009	0.47	2
Sitka Spruce	8 by 16	Green	20	10.5	2.259	0.28	2
Western Hemlock	8 by 16	Green	28	7.6	2.783	0.37	4
	8 by 16	Green	35	-7.5	4.394	0.46	3
Douglas Fir	6 by 12	Green	26	13.1	2.601	0.24	4
	5 by 8	Green	62	-2.6	3.657	0.69	4
	8 by 16	Green	166	5.0	3.487	0.44	4
Red pine	6 by 12	Green	15	4.3	2.990	0.71	4
Redwood	6 by 12	Green	14	14.1	2.255	0.32	4
	7 by 9	Green	14	-3.3	4.580	0.76	4
	8 by 16	Green	14	12.0	2.709	0.44	4
Southern Pine	5 by 8	Green	24	4.6	3.405	0.71	4
	5 by 12	Green	92	5.4	3.054	0.53	4
	8 by 12	Green	23	6.1	3.113	0.66	4
	8 by 16	Green	29	11.6	2.577	0.34	4
Eastern larch	6 by 12	Green	15	3.1	3.371	0.34	4
Western larch	5 by 8	Green	14	-0.2	3.741	0.70	4
	8 by 12	Green	30	6.5	3.268	0.58	4
	8 by 16	Green	32	7.9	2.743	0.33	4
Southern Pine	6 by 6	14	99	11.2	2.105	0.34	5
Sitka Spruce	8 by 16	17	20	-1.2	3.911	0.56	2
Western Hemlock	8 by 16	17	29	2.8	3.506	0.36	2
	8 by 16	18	42	1.7	3.603	0.42	4
Douglas Fir	6 by 12	17	28	-1.8	4.417	0.53	3
	5 by 8	15	30	-4.0	3.966	0.36	4
	8 by 16	17	37	-9.2	4.954	0.59	4
Western larch	8 by 12	18	26	-8.2	4.862	0.50	4
	8 by 16	18	18	-13.4	5.390	0.43	4

[a]For the Cline and Heim (1912) data, we dropped all "dry" pieces with moisture content > 20 percent and all "green" pieces with moisture content < 28 percent.
[b]Unit of measurement is giga Pascals for MOE and mega Pascals for MOR.
[c]Reference 1, Kretschmann and Green (submitted); 2, Markwardt (1931); 3, Littleford (1967); 4, Cline and Heim (1912); and 5, Green and Kretschmann (submitted).

lumber. This study was more comprehensive than the study on Southern Pine. The oak timbers came from West Virginia and Pennsylvania and were harvested in two installments 1.5 years apart. The r^2 of our Southern Pine data was slightly lower than expected, but nevertheless significant and useful. The study on Southern Pine had a more limited scope compared to the study on Mixed Oak, and the lower r^2 value is probably a result of the limited sample size.

Figure 1 shows the relationship between compression strength and bending strength. The relationship for timbers appears to follow the general trend found for standard 38-mm (nominal 2-in.) dimension lumber (Green and Kretschmann 1991). The UCS/MOR ratio for green timber was a little lower than that for dry timbers or dry dimension lumber. This decrease in the ratio with increasing moisture content was also found for standard 38-mm- (nominal 2-in.-) thick softwood lumber (Green and Kretschmann 1991). We conclude that the general UCS–MOR relationship used to assign properties to mechanically graded lumber could be applied to dry mechanically graded Southern Pine timbers and a slightly more conservative relationship could be used for Mixed Oak.

We did not conduct tensile tests on timbers. However, property relationships between MOR and MOE, and between UCS and MOR, seem to follow the trends expected for dimension lumber. Thus, we see no reason to doubt the ratio between tension and bending strength of 0.55 given in ASTM D245.

Figure l—Relationship of compression strength to bending strength for lumber.

Figure 2—Relationship of strength to stiffness for oak 7 by 9 lumber.

Potential for Mechanical Grading of Oak Timbers

Having established the technical foundation for mechanical grading, next we wanted to verify that the procedure would work in the field. This involved establishing some target grade boundaries, sorting timbers according to these boundaries, and testing a sample of the graded pieces to verify that they had the intended properties. We chose Mixed Oak timbers for this evaluation.

The first step was to establish a 95-percent lower confidence limit on the MOE–MOR regression. This could be used to set MOE and MOR grade boundaries for proposed mechanical grades. Non-strength-reducing characteristics, such as wane or decay, were set equivalent to those of No. 2 timbers. Which grades to sort was somewhat an arbitrary decision. To obtain additional information on potential grades to evaluate, we obtained stress wave values on an additional 168 green 7 by 9 oak timbers. Using these data, we chose a target grade with properties equivalent to those of Select Structural Southern Pine timbers (AFPA 1991). Thus, one of our target grades was 1500f-1.5E, or an allowable F_b value of 10.3 MPa (l.5 × 10^3 lb/in^2) and an E of 10.3 GPa (1.5 × 10^6 lb/in). For a higher grade, we chose properties equal to those of Dense Select Structural Douglas Fir–Larch beams and stringers (AFPA 1991). Our second target grade was thus 1900f-1.7E. Of the 168 mill-run pieces in this preliminary data set, 93 percent fit the 1.5E criterion and 70 percent fit the 1.7E criteria.

The next step was to grade 400 timbers that met the MOE and visual requirements. Of the timbers that met our proposed grade, 60 pieces for each of the two grades were randomly selected to be shipped to FPL for testing. Combining the data from the first 100 bending tests with the 120 pieces from the second sample yielded an MOE–MOR regression with an r^2 value of 0.51. This value is about what would be expected for standard 38-mm (nominal 2-in.) dimension lumber. Revised grade boundaries can be set by fitting a 95-percent confidence bound to the data (Fig. 2). The final step will be to obtain some yield information on other sizes of Mixed Oak timbers using the proposed grading criteria.

Conclusions

From the results of our research to date, no technical barriers have been identified that would hinder the implementation of a mechanical grading system for solid sawn timbers. Many practical concerns would still need to be addressed. Practical details relating to use of the system at specific sites would need to be considered, as well as details of a quality assurance system. Producers and users would have to agree on grades of mutual interest. Ideally, the system might become accepted by the American Lumber Standards Committee with monitoring by existing grading agencies. Without such acceptance, individual companies would have to proceed on a proprietary basis. Future research will address the application of this concept to other species and its use in grading recycled timbers or timbers “in-place” in structures.

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Shear Strength of Glued-Laminated Timber Beams and Panels

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Abstract

Five sizes of glued-laminated timber beams and panels are being tested in a five-point beam-shear test configuration to determine the shear strength capacity and size relationships. A total of 200 matched specimens will be tested: 100 loaded edgewise about the strong axis and 100 loaded flatwise about the weak axis. Statistical methods will be used to estimate mean and coefficient of variation considering censored data, test the significance of strength-size effect, and correlate between beam shear and ASTM D143 block shear. At the time this paper was written, only the beams loaded edgewise about the strong axis were completely tested along with the smaller-sized beams loaded about the weak axis. This paper presents results for the strong axis tests and a general discussion about the weak axis tests. To date, edgewise specimens had similar size–strength relationships as were observed in previous glued-laminated shear studies.

Keywords: Glued-laminated, shear strength, beams, panels, design, size effect

Introduction

Allowable shear strength values for wood construction are determined by adjusting ASTM D143 (1995a) shear-block results, according to methods specified by ASTM D245 (1995b) for visually graded lumber and ASTM D3737 (1995e) for glued-laminated timber material. Researchers have questioned the validity of predicting beam shear by ASTM shear-block testing (Ethington and others 1979; Radcliffe and Suddarth 1955). In 1994, Soltis and Rammer (1994) established a relationship between ASTM D143 shear block and beam shear by considering the size of the specimens. Although their research correlates beam shear to traditional testing methods, questions remain about shear design. One such question concerns the design of vertically laminated deck panels for timber bridges.

Deck panels loaded flatwise about the weak axis have a shear design value less than panels loaded edgewise about the strong axis (AITC 1993; AFPA 1991). When you consider the current ASTM standards, past and current shear research, and laminating effects, the difference in shear strength seems counter intuitive.

Objective

The general objective of this study is to improve the shear design criteria as it applies to deck panels. Specific objectives include the following:

- Improve the beam-shear database to include axial glued-laminated deck combinations.
- Determine if there is a correlation between shear strength and size for glued-laminated deck panels. Size parameters considered are beam width, shear area, or volume.

- Determine if the shear strength versus shear area equation derived for horizontally laminated beams is applicable to vertically laminated beams.
- Clarify the design method for highway bridge decks by investigation of the failure mode in panels bent about the weak axis.

These objectives will be met through an experimental testing program and statistical analysis of the experimental results.

Background

The ASTM D3737 method for determining shear strength of horizontally laminated glued-laminated material results in greater design values than does vertically laminated material. Lower design values for vertically laminated members occur because ASTM D3737 assumes every fourth lamination has a through width check or split that limits the strength ratio to one-half. In a four-lamination beam, this results in a total reduction of one-eighth. Based on rational thinking, this approach seems inefficient in light of the ASTM D3737 §4.5, current shear strength research, and laminating concept.

Currently, ASTM D3737 §4.5 states the following about the quality of the laminating member prior to fabrication:

> Lumber shall be free of shakes and splits that make an angle of less than 45° with the wide face of the piece. Pitch pockets shall be limited in size to the area of the largest knot permitted, and pitch streaks shall be limited to one sixth of the width of the lumber.

By this passage, ASTM is requiring a higher quality of material, with respect to splits, shakes, and checks.

Lumber grading rules also specify a maximum allowable split length as a function of the wide dimension of the member. For select structural (SS) and No. 1 material, maximum split length is limited to the width of the wide face. For No. 2 material, the split length is limited to one and a half the member width (WWPA 1991).

The typical probability of no cracks in a piece of wood is given in Table 1, according to the Canadian In-Grade Testing Program. Based on these Canadian data and species, only the 39- by 254-mm (2- by 10-in.) material did not meet or exceed the one in four condition for splits. In addition, the average split length is 0.75 times the width for Douglas Fir, 0.93 the width for Hem-Fir, and 0.62 for Spruce-Pine-Fir (Foschi and others 1989).

Split limits are applied to the lamination material before fabrication. When fabricated in a beam, splits will be restrained from propagation by the surrounding material and cannot develop a full-length split as assumed in the ASTM D3737 criteria. Therefore, quality requirements, split probability, and length characteristics along with split restraint after fabrication seem to make the one in four split assumption conservative.

Since the early 1970s, shear research has moved from the ASTM shear-block to beam-shear experiments. In the first comprehensive study on beam shear, Keenan (1974) indicated that shear strength of Douglas Fir glued-laminated timber beams depended on the sheared area. Also, he observed that the two-beam theory proposed by Newlin and others (1934) is not applicable to glued-laminated timber beams.

Foschi and Barrett (1976, 1977) applied Weibull's theory of rupture to shear strength of wood. The Weibull theory is a statistical theory that predicts the strength of material at a given probability of failure.

Longworth (1977) used a simple-span, Douglas Fir glued-laminated timber beam with symmetrically placed concentrated loads to show experimentally that ASTM shear-block strength is unrepresentative of beam-shear strength, and shear strength varies with beam size, sheared area, or volume.

Quaile and Keenan (1978) used a single-span beam test to investigate maximum shear in rectangular beams of specially designed glued-laminated material. This test specimen was successful in producing shear failures in 104 of the 108 specimens tested and gave further evidence that the ASTM shear-block test produces

Table l—Probability of no crack in different material (Foschi and others 1989).

	Douglas Fir (%)			Hem-Fir (%)			Spruce-Pine-Fir (%)		
Grade	39 by 254 mm (2 by 10 in.)	39 by 203 mm (2 by 8 in)	39 by 102 mm (2 by 4 in.)	39 by 254 mm (2 by 10 in.)	39 by 203 mm (2 by 8 in.)	39 by 102 mm (2 by 4 in.)	39 by 254 mm (2 by 10 in.)	39 by 203 mm (2 by 8 in.)	39 by 102 mm (2 by 4 in.)
SS	0.82	0.81	0.92	0.94	0.95	0.97	0.90	0.91	0.99
No. 2	0.72	0.76	0.95	0.78	0.86	0.96	0.78	0.87	0.98

lower strength values than is evident in small, rectangular beams. Quaile and Keenan also investigated the effect of growth ring orientation on shear strength. They concluded that radial shear strength values were statistically different than tangential values at a 0.01 level of significance. This indicates that orientation, either flatsawn or quarter sawn, might influence the shear strength of the beam. This orientation effect was also observed in Southern Pine shear-block results (Bendsten and Porter 1978).

Keenan and others (1985) used a special test specimen to determine the shear strength of glued-laminated spruce beams. They concluded that shear strength is not a function of volume but of sheared area.

Rammer and Soltis (1994) experimentally defined the beam-shear strength of Southern Pine and Douglas Fir glued-laminated timber beams. Strength was determined for various-sized beams, 39 by 102 mm (2 by 4 in.) through 127 by 610 mm (5 by 24 in.), by a five-point bending method.

Based on the five-point bending and ASTM D143 shear-blocks results, Soltis and Rammer (1994) empirically derived an equation that related the two findings. Their equation considers the stress riser at the re-entrant comer of the ASTM shear block, different wood species, and the effect of beam size on shear strength. The equation is

$$\tau = \frac{1.9\, C_f\, \tau_{ASTM}}{A^{1/5}} \qquad (1)$$

where

τ = beam shear strength, MPa,

C_f = 2 = stress concentration factor to adjust the ASTM shear-block strength to the true strength at failure,

τ_{ASTM} = ASTM D143 published shear-block values (MPa), and

A = shear area = area of beam subjected to shear forces (cm^2).

These results were comparable to Longworth's (1977) work on Canadian Douglas Fir glued-laminated timber and Yeh's (1993) work on Douglas Fir glued-laminated timber and laminated veneer lumber beams. Although Equation (1) captures the effect of size on shear strength, the testing program could not easily distinguish between shear area and volume. Therefore, volume could be the significant parameter.

Recently, Janowiak and others (1995) investigated the shear strength of vertically laminated 178- by 229-mm (7- by 9-in.) Red Maple glued-laminated beams. They found that the shear strength of these beams was 10% greater than the values predicted by Equation (1), which is applicable to horizontally laminated beams, but only eight beams of one size were tested.

Glued-laminated material has always boasted of the positive effects of lamination to resist bending forces: improved lumber utilization, more effective transfer of stresses around defects (knots), and reduced strength variability. Positive effects may also be experienced by a deck panel resisting shear forces. A simplistic analogy is used to explain this reasoning.

The major advantage of a glued-laminated beam is its ability to resist bending forces with the cross-sectional moment of inertia (full-composite action). Full-moment resistance may only be accomplished if shear forces are transferred completely through the cross section, from one elemental point to the next point. Inability to transfer shear in one lamination will lead to substantially reduced inertia values, and in the extreme equal, the sum of the individual lamination inertia values (no composite action).

In a beam with full-width horizontal laminations and loaded about its strong axis, shear strength will be governed by the weakest lamination within the middle section of the beam. Delamination and a reduction in the moment of inertia will occur when the shear strength of the weak lamination is exceeded. If the load is maintained or increased, this will lead to failure of the remaining section by tensile rupture or shear.

In a deck panel (same beam as just noted, but loaded about weak axis), the weak lamination will not entirely control the shear strength of the beam. When the weak lamination shear strength is exceeded, the lamination will break and shed load to the surrounding stronger laminations. This load sharing or shedding process will lead to increased performance The larger the deck panel, the smaller the increase of load to the individual surrounding members and the greater the performance.

These failure scenarios are indicative of the two system failure extremes. Essentially the edgewise bending shear failure is an example of a series of chain links; the weak link controls overall system strength. A series system is characterized by a brittle failure and higher variability. The flatwise shear failure is an example of a series of chains acting in parallel to resist the load. The failure of one chain will lead to a load redistribution, which may or may not break the system. Parallel systems are characterized by a more ductile failure.

You could hypothesize that laminated material loaded flatwise could have a shear strength equal or greater than the same material loaded edgewise because of the quality of lamination material, the dependence of shear strength on material orientation, and the reinforcing effects.

Test Program

All previous beam-shear studies concentrated on specimens loaded about its strong axis. In this study, we will test Douglas Fir glued-laminated timber beams to determine both flatwise and edgewise shear strength values and compare the applicability of size–strength relationships for flatwise shear.

A total of 200 matched specimens will be tested: 100 loaded edgewise about the strong axis and 100 loaded flatwise about the weak axis. At the time this paper was written, only the beams loaded edgewise about the strong axis were completely tested along with the smaller-sized beams loaded about the weak axis.

Specimens

To determine the beam-shear strength for various beam sizes, a test matrix covering a range of member sizes was investigated. A standard 1,219-mm (48-in.) wide deck panel cannot be tested due to testing machine limitations. Based on previous shear research, specimens with a shear area greater than 1,935 cm^2 (300 in^2) show slight changes in strength so that the 762-mm- (30-in.-) wide panel could approximate the 1,219-mm- (48-in.-) wide panel (Rammer and Soltis 1994). Tested material consists of Douglas Fir, AITC/ANSI (1995) Combination No. 2 panels. The number and size of specimens tested are listed in Table 2. All material was maintained in a dry but uncontrolled environment until testing. Moisture content ranged between 11% and 13% (on average) at testing.

Beam-Shear Test Setup

A five-point bending test is used to produce a high percentage of beam shear failures. This method has been used successfully to create shear failures by Langley Research Center (Jegly and Williams 1983), Purdue University (Bateman and others 1990), and the Forest Products Laboratory (Rammer and Soltis 1994, Rammer and others 1996). A beam specimen is tested over three supports, and a concentrated load is applied at the middle of each span. This arrangement produces a region of high shear force between the load point and the middle support, making shear failures possible. A schematic of the setup is shown in Figure 1 and corresponding setup dimensions in Table 3. Loading rate is such to cause failure between 5 and 20 minutes, ideally at 10 minutes. Information recorded includes maximum load, type and location of failure, material properties, beam geometry, moisture content (ASTM 1995e), and specific gravity (ASTM 1995c).

Table 2—Size and number of coast Douglas Fir specimens.

Beam size (mm)	Specimen length (m)		Number of specimens
	Flatwise	Edgewise	
130 by 152	1.63	1.70	20
130 by 305	1.63	3.10	20
130 by 419	1.63	4.50	20
130 by 572	1.63	5.89	20
130 by 762	1.63	7.29	20

Shear-Block Tests

Small, clear ASTM D143 shear-block specimens will be cut from the specimens after failure for correlation between previous ASTM D143 shear-block data and beam-shear tests. Shear blocks are cut from material near the failure and along the grain line but free from splits, checks, or knots. Testing will conform to ASTM D143 specifications.

Figure l—Five-point beam-shear test configuration.

Table 3—Beam shear test dimensions.

Orientation of specimen	Beam size (mm)	Span length (m) 2.5*d*	5*d*
Flatwise	130 by 152	0.33	0.65
	130 by 305	0.33	0.65
	130 by 419	0.33	0.65
	130 by 572	0.33	0.65
	130 by 762	0.33	0.65
Edgewise	130 by 152	0.38	0.76
	130 by 305	0.76	1.52
	130 by 419	1.05	2.10
	130 by 572	1.43	2.86
	130 by 762	1.90	3.81

Table 4-Beam shear strength results.

Specimen orientation	Beam size (mm)	Number of shear failures	Shear strength (MPa)	COV[a] (%)
Flatwise	130 by 152	9	10.0	8.3
	130 by 305	—	—	—
	130 by 419	—	—	—
	130 by 572	—	—	—
	130 by 762	—	—	—
Edgewise	130 by 152	18	7.71	9.7
	130 by 305	18	6.16	14.9
	130 by 419	19	5.81	12.0
	130 by 572	19	4.82	14.6
	130 by 762	18	4.63	10.2

[a]Coefficient of variation.

Calculations

For the assumed stress distribution, the longitudinal shear stress equation for a rectangular section is

$$\tau = \frac{VQ}{Ib} = \frac{3V}{2bd} \qquad (2)$$

where
τ = Shear stress (Pa)
V = Shear force (N)
b = Width of beam (m)
d = Depth of beam (m)
I = Moment of inertia (m^4)
Q = Statical moment of the area (m^3)

The general equation that relates failure load to shear strength for the five-point test configuration (Fig. 1) is

$$\tau = \frac{33}{32}\frac{P}{bd} \qquad (3)$$

where P is the load applied to one span.

Based on orthotropic analysis procedures (Gerhardt and Liu 1983) of the shear failure region, Equation (3) is within 10% of elastic theory results.

In previous shear strength work, small-sized beams experienced a lower percentage of shear failures. Rammer and others (1996) applied censored statistical techniques to estimate the uncensored mean and coefficient of variation. These statistical techniques will again be applied in this report for analysis of the flatwise beams. In addition, an Analysis of Variance (ANOVA) will be performed to determine if size of beam significantly affects beam-shear strength.

Results

The following summarizes the experimental results for the beam-shear and shear-block testing. Complete details of results and procedures will be published in a future research paper.

Beam-Shear Tests

Five-point bending tests about the strong axis (edgewise) resulted in 92 shear failures out of 100 specimens tested for a 92% shear failure rate. Table 4 lists the average shear strength of only those specimens failing in shear. The edgewise specimen shear strength decreased as the size of the specimen increased, as indicated in Table 4. The coefficient of variation values for Douglas Fir shear strength reported here, 9.7% to 14.9%, are slightly larger than the 8% values observed in glued-laminated Douglas Fir beams (Rammer and Soltis 1994), but are typical for shear strength variance (ASTM 1995d). Note that four beam specimens failed at loads lower than the 3rd percentile values.

Shear-Block Tests

Average shear-block strength and coefficient of variation for edgewise specimens was 7.83 MPa (1,136 lb/in^2). The specific gravity was 0.46 at a moisture content of 10.7%. The shear-block values are similar to the published average strength and variability values (ASTM 1995d).

Discussion

In general, shear failures generated a crack between either load point or middle support. Figure 2 shows the general failure for the 130- by 762-mm (5 1/8- by 30-in.) and 130- by 305-mm (5 1/8- by 12-in.) sizes that represent the upper region of shear strength distribution. Four failures occurred at lower load values than expected. Upon investigation of these four failure

Figure 2—Shear failures in (top) 130- by 762-mm (5 1/8- by 30-in.) and (bottom) 130- by 305-mm beam specimens.

Figure 3—Incomplete glueline bonding in specimen 130 by 305 mm (55 1/8 by 12 in.).

surfaces, one failure occurred at a location of white rot, and two failures were attributed to incomplete bonding of the glueline, (Fig. 3). Incomplete glueline bonding acted like an artificial check or split beam, and the white rot was a plane of extreme shear weakness. These cases illustrate the importance of bonding and wood quality on shear strength. All these failures occurred at loads lower than 3rd percentile levels.

An ANOVA was performed on the edgewise shear strength data, including the four lower load specimens. ANOVA calculations were performed using a general linear model and Tukey's studentized range test for multiple hypothesis comparison at 0.05 level of confidence (SAS 1988). Based on the ANOVA, three significant groupings resulted. The 130- by 172-mm (5 1/8- by 6-in.) specimens were grouped alone as having the highest shear strength. Next, both the 130- by 305-mm (5 1/8- by 12-in.) and 130- by 419-mm (5 1/8- by 16 1/2-in.) specimens were grouped together. Finally, both the 130- by 572-mm (5 1/8- by 22 1/2-in.) and 130- by 762-mm (5 1/8- by 30-in.) specimens were grouped together, having the lowest shear strength values. The ANOVA indicates that shear strength significantly varied with size, with the smaller-sized specimens having greater strength.

To compare these data with previous glued-laminated shear strength results, the ratio of beam-shear strength to ASTM shear-block strength is compared with shear area and beam volume (Fig. 4). Shear area is taken as the length of beam under high shear forces (distance from load-point to load point = 2.5d, Fig. 1) times the width of the beam, and volume is taken as shear area times beam depth. The edgewise material falls within the variability of previous glued-laminated beam-shear

Figure 4—Beam-shear to ASTM shear-block ratio compared with beam size: (top) shear area (bottom) beam volume.

Figure 5—Cross section of failure paths in 130- by 152-mm specimens: (top) edgewise tests (bottom) flatwise.

studies. Results for the flatwise shear are not plotted, because testing was not complete at the time of writing this report.

Flatwise shear strength tests were not completed at the time of report writing, but general observations on smaller beams can be discussed in comparison to the edgewise specimens. In comparing flatwise and edgewise results, it was observed that flatwise values tended to be greater than edgewise values. In fact, a T-test revealed a statistical difference at the 95%. confidence level.

Edgewise shear specimens failed immediately, whereas flatwise failures occurred progressively as indicated by load versus time plots. Applied load increased steadily until a initial shear crack developed and slightly reduced the load. On continued loading, this load reduction was recovered and exceeded, but at a slower loading rate because of the loss of beam stiffness. Subsequently, additional load drops caused by shear or bending failure of the remaining laminations occurred until complete failure of the beam. Therefore, shear failure of the flatwise glued-laminated beams is progressive.

In addition to load-sharing mechanisms, the orientation of the laminations might attribute to increasing the load capacity of the small, flatwise shear results and compared to the edgewise data. All test specimens had a predominant number of flatsawn laminations. This results in edgewise specimens resisting shear forces in the tangential-longitudinal (TL) shear plane, whereas flatwise specimens resisted shear forces in the radial-longitudinal (RL) plane

(Fig. 5). Researchers (Bendsten and Porter 1978, Quaile and Keenan 1978) have indicated that shear strength along these planes is significantly different, with the RL being stronger than the TL. Because edgewise specimens typically failed along the weaker TL plane compared to flatwise specimens that failed across the stronger RL plane, flatwise specimen results could be greater based on material orientation.

In addition to the apparent orientational differences in shear strength, van der Put (1993) formulated that failure criterion is also a function of the material axes. He stated that compression only effects the RL shear strength and has little effect on the TL shear strength. The RL shear strength increases with compression, whereas TL shear strength is constant or slightly decreases. Considering that laminations are predominantly sheared across the TL plane in the edgewise test compared to the flatwise tests and compression has little interaction on the TL shear plane, this would lead to lower shear strength values for the edgewise tests compared to the flatwise tests. Flatwise tests are predominantly shear across the RL plane, which might increase strength with compression.

It should be stated that these comparisons are only based on one specimen size (130 by 152 mm (5 1/8 by 6 in.)); therefore, these observations might not be general for all sizes tested.

Concluding Remarks

An experimental study to investigate the shear strength performance of axial combination Douglas Fir glued-laminated beams and panels is being conducted. Edgewise shear tests are completed and the results fall within the variation of previous glued-laminated shear strength results. An ANOVA indicated a statistical difference in the shear strength with beam size.

Flatwise shear tests are being conducted with only the smaller-sized beams completed. Based on observations of 130- by 152-mm (5 1/8- by 6-in). flatwise tests, a different failure mechanism is presented. Flatwise tests are a progressive failure because each lamination fails in sequence by shear or bending. This progressive failure leads to shear strength results that might be greater than edgewise results. In addition, failure paths in both laminated panels are different.

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Long Term Load Performance of FRP Reinforced Glulam Bridge Girders

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Abstract

The use of high-strength-fiber-reinforced plastic (FRP) as a reinforcement for glulam bridge girders is now a commercial reality. To assist in the assessment of in-service characteristics of these new girders, a 49.6 m HS-25 highway bridge constructed in August of 1995 was equipped with internal strain gauges for long-term load effects monitoring. This report represents the findings to-date from these tests. Further, the economic, environmental and maintenance advantages of reinforced glulams are presented.

Keywords: Reinforced glulams, glulam bridge girders

Background

A bridge composed of FiRP™ Reinforced glued laminated timber (glulam) girders and a transverse conventional glulam deck was constructed over the Clallam river near Sekiu, Washington. It consists of two simple spans, each 24.8 m, for a total span of 49.6 m with an HS-25 load rating. Six Douglas-fir (DF), L2 girders spaced at 1.60 m center-to-center are used for each span. A composite modulus of elasticity (E) of 14480 MPa was used to estimate deflections. The exterior girders are 222 mm wide by 1499 mm deep with four layers of 3.81 mm thick aramid reinforced plastic (ARP) FiRP™ Reinforcement. The interior girders are 171 mm wide by 1499 mm deep with seven layers of 1.78 mm thick carbon-m-amid reinforced plastic (CARP) FiRP™ Reinforcement. The bridge deck is constructed of 52 conventional glulam deck panels oriented transversely to the girders. Also, glulams were used for deck stiffeners and curbs. All the lumber was pressure treated with pentachlorophenol. Figures 1 and 2 show side and end views of the Clallam Bay bridge, respectively.

The bridge was designed by Western Wood Structures, Inc. of Tualatin, Oregon and manufactured by American Laminators, Inc. of Swisshome, Oregon. Monitoring of the bridge was undertaken in a joint effort by the Wood Science & Technology Institute (N.S.), Inc. and the Forest Products Laboratory at Oregon State University, both of Corvallis, Oregon.

The FiRP™ Reinforcement technology was implemented in the girders because of the significant cost and weight savings it provides as compared to conventional glulams. The reinforcement allows low grade lumber to be utilized throughout the depth of the beam. The reduced cross section, increased bending stiffness, and low grade lumber substitution often provide a substantial size and cost reduction compared to unreinforced glulam beams.

Another significant advantage of FiRP™ Reinforced glulam beams is the reduced variation in the modulus of elasticity and the strength. The reduced variation permits significantly increased design properties and a closer approximation of the deflections.

Figure 1-Side view of the Clallam Bay bridge.

Figure 2-End view of the girders.

Figure 3-Plan view showing the location of gauged girders in the Clallam Bay bridge.

Figure 4-Plan view of strain gauge layout in the girders.

The estimated unreinforced glulam beam size is 273 mm x 1524 mm for the interior and exterior girders. Based on an estimated wood cost of $775/MBF for reinforced Douglas-fir L2 and $850/MBF for unreinforced Douglas-fir 24F-V4, the overall cost savings for the bridge girders is approximately 24% (compared to the unreinforced option) which includes $250/MBF for treatment costs. FiRP™ Reinforced girders also provide an estimated 10% cost savings and less than 10% of the weight compared to precast concrete.

Objective

The purpose of this study is to investigate the long term performance of FiRP™ Reinforced glulam bridge girders subjected to in-service conditions such as dead loads, live loads, and environmental factors such as moisture content and temperature fluctuations.

Procedure

Both electronic data acquisition of strain gauges and direct surveying using precision optical levels are used to assess the response of the Clallam Bay bridge to applied loads and environmental conditions.

Strain Gauge Data Acquisition

Three of the main girders, two exterior and one interior, were fitted with internal strain gauges internally on the reinforcement and on the wood. Figure 3 shows the location of the gauged girders in the bridge. Figures 4 and 5 show the location of the internal gauges. The strain gauges are located at the center span point and at 3.05 m and 6.10 m toward each end from the centerline.

General purpose strain gauges (Measurements Group, Inc., Raleigh, North Carolina), type EA-06-10CBE-120) with a 25.4 mm effective gauge length and 120.0 ± 0.15% ohms resistance at 24° C are used for the long term monitoring. This type of strain gauge has a working temperature range from -75° C to 175° C and a maximum of 5% strain.

The wires from each gauge are connected to the monitoring apparatus in a central instrument panel located near the centerline bridge pier. For the first portion of this study, battery power was provided in a separate enclosure to power the instrumentation. Strain gauge data is collected at 108 min. intervals and is routed to a 32 channel multiplexer for each girder then stored in the memory of a Campbell datalogger.

Figure 5-Photogragh of the northwest exterior girder showing location of the strain gauges.

Sensors for temperature, relative humidity and wood moisture content, a modem interface to allow remote user access to the data, and power line from a utility pole to provide electricity to the instrumentation will be installed in the summer of 1996.

Strain Gauge Data Processing

Strain gauge data down loaded from the Campbell datalogger was processed in Microsoft Excel 5.0 (1994). The voltage output from the gauges was converted to microstrain through a calibration constant of 1904 $^{\text{microstrain}}/_{\text{Volt}}$. The strain was subsequently zeroed-out based on the first recorded strain reading.

Survey of the Northwest Exterior Girder

Additional deflection measurements are taken by direct surveying. Locations on the bottom of the northwest exterior girder corresponding to the internal strain gauge locations are marked to allow consistent level measurements to be made. Benchmarks are located at the supports and the center pier. The initial survey took place on September 21, 1995 and the elevation of the deflected shape of the northwest exterior girder was established. During this time, only the dead load of the girder and the bridge deck contributed to the deflection. Upon returning to the bridge on January 26, 1996 (after the construction was complete), another survey was conducted to measure the deflected shape of the northwest exterior girder.

Results

Strain Gauge Analysis

Figure 6 displays the axial strain on the wood of the northwest exterior girder at the centerline, 3.05 m from the centerline and 6.10 m from the centerline. As shown in Figure 6, the axial strain in the northwest exterior reinforced glulam beam in the extreme tension zone shows an initial increase due to application of the asphalt concrete wearing surface. However, after the initial strain increase, no significant creep can be seen. A significant amount of variation in the strain is apparent.

A Fourier analysis was conducted to transform the time-domain to the frequency-domain using a fast Fourier transform algorithm to check for the influence of cyclic events. Figure 7 displays the Fourier amplitude spectrum of a strain gauge mounted on the wood between the bottom lamination and the reinforcement on the northwest exterior girder. Perhaps the most important result from the Fourier analysis is the presence of a spike at the one-per-day frequency indicating that there is significant cyclic

Figure 6-Response of strain gauges located at the centerline of each of the three gauged girders in the tension zone.

Figure 7-Fourier amplitude spectrum of a typical strain gauge response.

behavior at one-day intervals (the corresponding harmonics can be seen at higher frequencies). All of the strain gauges experienced the cyclic behavior. The most reasonable explanation of this response is day-to-day temperature fluctuations.

Generally, the strain gauges placed on the two exterior girders exhibited a larger degree of sensitivity to temperature than the interior girder due to the more direct exposure to the environment.

Figure 8-Survey results of the northwest exterior girder.

Survey of the Northwest Exterior Girder

Using a precision automatic level, the elevation of the bottom of the northwest exterior girder was measured in reference to a brass benchmark located on the north side of the center pier (assumed elevation of 6.096 m). Figure 8 displays the results of the two surveys. A difference in the centerline deflection of 1.016 mm can be seen from Figure 8 as a result of the increased dead load from the asphalt wearing surface.

Conclusions

The advantages of using FiRP™ Reinforced glulams were found to be

- Reduced cost due to low grade wood substitution
- Small cross section
- Substantially lower treatment costs
- Lower transportation weight
- Decreased coefficient of variation of the modulus of elasticity and strength
- Deadweight reduction allowing for simplified erection
- Utilization of low grade wood

The data collected to-date shows no appreciable creep in the reinforced girders. The slight increase in strain can be attributed to temperature effects.

The strain gauge output from the datalogger is heavily influenced by environmental conditions. It is apparent that modifications need to be implemented to the strain gauge acquisition system to account for the influence of environmental conditions. Planed modifications to be implemented during the summer of 1996 are:

- Control strain gauge, subject only to thermal stress to account for temperature variations.
- Thermometer
- Relative humidity indicator
- Modem for periodic downloading of data
- Electrical outlet for the data acquisition system

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Effect of FRP Reinforcement on Low Grade Eastern Hemlock Glulams

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Abstract

The benefits of reinforcing glulam beams made with eastern hemlock, an under-utilized wood species in the state of Maine, are discussed. Nine beams reinforced with fiber-reinforced plastics (FRP) on the tension side and three unreinforced controls were instrumented and tested to failure in four-point bending. Low, medium, and high quality wood were used in the experimental study. FRP reinforcement ratios ranged from 0.3% to 3.1%. A nonlinear numerical model that predicts the performance of the FRP-glulam beams through the entire load range was developed and its predictions are compared with the test results.

Keywords: glulam, FRP, reinforcement, nonlinear model, strength, stiffness, ductility.

Introduction

Glued laminated wood (glulam) has been in use since the late 1800's. Research on glulam at the USDA Forest Products Laboratory in Madison, Wisconsin began in the 1930's. Development of glulam in following years in the United States was encouraged by the lack of adequate solid timbers and the high demand for large timbers created by the World War II effort (Freas and Selbo, 1954).

Glulam can be fabricated in many shapes and sizes, and has been used in numerous applications including keels for boats, arches for airplane hangers, churches, timbers for floor and roof systems, dome structures, transmission poles, along with girders and decks for timber bridges.

With recent changes in availability of forest resources, high quality laminations necessary for glulam design "have become increasingly difficult to procure, and more expensive as well." (Leichti, 1993, p. 3) Moreover, glulam, like reinforced concrete, can be reinforced in tension to more efficiently utilize the wood's compressive strength. Fiber reinforced plastics (FRP) offer good promise to serve both as a substitute for the high quality wood laminations and as reinforcement for glulam beams.

Over the past decades both FRP and non-FRP materials have been used to reinforce or prestress wood beams. With regard to non-FRP materials, Mark (1961) studied the effects of bonding aluminum to the compression and tension faces of wood core sections of eight different wood species. Sliker (1962) bonded aluminum sheets between various layers of laminated wood beams. Bohannan (1962) reinforced glulam beams of low-grade Douglas-fir using pretensioned steel wire strands in the tension zone.

Peterson (1965), in a study similar to Bohannan (1962), reinforced low-grade Douglas-fir glulam beams with a prestressed flat steel strip bonded in the tension zone. Lantos (1970) reinforced rectangular laminated wood beams with steel rods. Stem and Kumar (1973) studied the effect of steel plate reinforcement for vertically laminated timber beams. Coleman and Hurst (1974) reinforced No.2 southern pine beams with light gage steel reinforcement. Hoyle (1975) tested members composed of nominal dimension lumber with toothed steel plates between lumber pieces. Bulleit, Sandberg, Woods (1989) reported on Spruce-Pine-Fir glulam beams reinforced in the tension zone with special steel-reinforced tension laminations.

Prior to 1990, a number of studies on wood beams reinforced with fiber and FRP materials were also conducted. Wangaard and Biblis (Wangaard, 1964; Biblis, 1965) studied the effect of bonding unidirectional fiberglass/epoxy reinforced plastic to the compression and tension faces of wood cores of various species. Theakson (1965) studied the feasibility of strengthening both laminated and solid wood beams with fiberglass. Krueger and Sandberg (1974 b) studied laminated timber reinforced in the tension zone with a composite of high-strength bronze coated woven steel wire and epoxy. Krueger and Eddy (1974 a) carried out research similar to that of Kruegar and Sandberg (1974 b). Spaun (1981) studied finger-jointed western hemlock cores reinforced with wood veneers and fiberglass rovings.

In the nineties, research on wood beams reinforced with fiber and FRP materials has increased. Plevris and Triantafillou (1992) studied the effect of reinforcing fir wood with carbon/epoxy fiber-reinforced plastics. Plevris and Triantafillou (1995) also discussed the creep behavior of FRP-reinforced wood. Triantafillou and Deskovic (1992) studied the effect of prestressed carbon/epoxy FRP (CFRP) reinforcement bonded to European beech lumber. Davalos, Salim, Munipalle (1992) discussed the response of small yellow-poplar glulam beams reinforced on the tension side with glass/vinylester FRP. Tingley and Leichti (1993) discussed glulam made from lower grade ponderosa pine reinforced in the tension zone with pultruded kevlar and carbon FRP. Abdel-Magid, Dagher, and Kimball (1994) studied nominal 2x4 hemlock beams reinforced tension with carbon/epoxy and kevlar/epoxy FRP. Sonti, Davalos, Hernandez, Moody, and Kim (1995) discussed yellow-poplar glulam reinforced with pultruded glass/vinylester FRP in tension or both in tension and compression. Dailey, Allison, Minneci, and Bender (1995) studied glulam reinforced in the tension zone with pultruded glass/resorcinol-modified phenolic FRP sheets.

While timber has been successfully reinforced over the past few decades using various materials and reinforcing techniques, very few of these methods of reinforcing timber have reached the commercial market (Bulleit, 1984). As argued by Bulleit et. al. (1989, p. 433), there are several possible reasons for this lack of commercialization of reinforced timber: "(l) The material used to reinforce the wood was not commonly used in building applications; (2) the reinforcing material was too expensive; and (3) the fabrication required an additional and, thus, cost-increasing step in the laminating process. " Another reason may be the incompatibility between the reinforcing material and the wood.

FRPs are a versatile class of materials that can be engineered to overcome the incompatibility problems with the wood. Because of FRPs falling cost and potentially simple incorporation into existing glulam manufacturing processes, this class of materials offers a good potential as a reinforcement for wood (Kimball, 1995). This paper describes an experimental and numerical study on reinforcing eastern hemlock glulams with FRP. Eastern hemlock is a relatively inexpensive, abundant and under-utilized Maine wood species with relatively low mechanical properties.

Experimental Work

A total of twelve glulam beams were fabricated and tested statically to failure (See Table 1). The beams had a clear span of 16 feet and a cross-section of 3 3/16 inches by 12 inches. The twelve beams consisted of three control (unreinforced) beams and nine beams reinforced with varying amounts and types of FRP in the tensile zone. Because of variations in lay up, three of the reinforced beams cannot be directly compared with the controls. These are described as Non-Comparison beams in Table 1.

All beams used No.2 and better 2x4 eastern hemlock. The No.2 visual grade material occupied over 75% of the sample. The lumber was condition to a 12 percent moisture content prior to laminating. Using MOE data, the wood was divided into three quality categories: Low, Medium and High. Both unreinforced controls and reinforced beams were constructed of each of the three wood categories.

Control Beams

Unreinforced control beams consisted of eight 1 3/8 inch laminations, with a 3/4 inch 'bumper-strip' lamination added to the outer wood tensile lamination.

FRP-Reinforced Glulam Beams

The reinforced beams were designed in the same way as the control beams with the exception of FRP placed between the 'bumper strip' and the outer tensile wood lamination. A transformed section of a typical reinforced beam is shown in Figure 1. Table 2 summarizes the properties of the two FRP types used in the beams.

Testing

The glulam beams were tested in four-point bending over a simple span as shown in Figure 2. The beams were braced to prevent lateral-torsional buckling and were tested according to the procedures outlined in ASTM D198-84.

Strain gages were applied throughout the depth of the beams within the constant moment region. A dial gage and LVDT were used to measure deflections. The beams were loaded at a rate of approximately 1000 pounds per minute. Readings included beam load, load head displacement, strain gage, LVDT and dial gage over the duration of the test.

Table 1 - Beam Characteristics

Beam No.	FRP Type	Beam Depth (in)	Reinforce-ment Ratio (%)	Wood Quality
Comparison Beams				
10	-	11.75	0	L
9	-	11.75	0	M
11	-	11.75	0	H
2	1	12.12	3.1	L
1	1	12.12	3.1	M
3	1	12.12	3.1	H
6	2	11.88	1.1	H,M
5	2	11.88	1.1	M
7	2	11.88	1.1	H
Non-Comparison Beams				
4	1	12.12	3.1	N/A
8	2	12.00	2.1	L
12	2	11.79	0.3	L

L=Low, M=Medium, H=High, N/A=Not Applicable

Figure 1 - Transformed Section

Figure 2 - Test Setup

Table 2 - FRP Properties

Property	FRP1	FRP2
Ultimate Tensile Strength (ksi)	114	137
Longitudinal Tensile MOE (Msi)	6.7	19.1

Load Deflection Data

For the low (L) quality comparison beams, the reinforced beam was 43% stronger and 31% stiffer than the control. In addition, the reinforced beam was more ductile than the control beam, deflecting 42% more at failure.

Figure 3 shows the experimental load-deflection curves for the medium (M) quality comparison beams including the control, FRPl and FRP2 reinforced beams. The FRPl reinforced beam was 51% stronger and 32% stiffer, while the FRP2 reinforced beam was **33%** stronger and 37% stiffer than the control. The FRPl reinforced beam showed improved ductility over the control beam.

Table 3 - Bending Test Data

Beam No.	FRP Type	Rein-force-ment Ratio (%)	Wood Quality	Max. Load (lb)	Max. Deflec-tion (in)	Beam MOE (Msi)	Str. Incr-ease (%)	MOE Incr-ease (%)
Comparison Beams								
10	-	-	L	8200	2.12	1.28	-	-
9	-	-	M	10780	3.08	1.26	-	-
11	-	-	H	12040	3.25	1.38	-	-
2	1	3.1	L	11750	3.02	1.68	43	31
1	1	3.1	M	16250	4.48	1.66	51	32
3	1	3.1	H	14900	3.57	1.78	24	29
6	2	1.1	H,M	15070	3.82	1.61	25	28
5	2	1.1	M	14310	3.05	1.72	33	37
7	2	1.1	H	15270	3.76	1.72	27	25
Non-Comparison Beams								
4	1	3.1	N/A	9040	1.94	1.74	-	-
8	1	2.1	L	12800	3.29	1.58	56	23
12	2	0.3	L	10930	3.54	1.26	33	-2

Figure 3 - Load-Deflection Curves for the Medium Quality Beams

The high (H) quality comparison beams include the control, FRP1 and FRP2 reinforced beams. The reinforced beams showed increases in strength of approximately 24 to 27% over the control. The reinforced beams showed increases in stiffness of 25 to 29% over the control.

With regard to the three non-comparison beams, the FRP1 reinforced beam with 3.1% reinforcement ratio failed prematurely in a wood tension lamination due to wood shake. Therefore, it is not compared to a control beam. The FRP1 reinforced beam with 2.1% reinforcement ratio showed an increase in strength and stiffness of 56% and 23%, respectively, over the low (L) quality control. Beam 12, which had only 0.3% FRP reinforcement, showed 33% gain in strength over the low (L) quality control beam but practically no change in stiffness.

Strain Gage Data

Stresses in the FRP at failure were calculated using strain data and beam MOE values. The tensile stresses in the FRP at onset of beam failure remained well below ultimate strengths in all cases. The stress in the FRP1 at beam failure was about 25% of its ultimate strength. The stress in the FRP2 at beam failure was less than 50% of its ultimate strength. However, the FRP2 reinforcement failed in interlaminar shear, thereby initiating overall beam failure.

Nonlinear FRP-Glulam Model

A nonlinear numerical model was developed to study the behavior of FRP-glulam beams for all stages of loading through failure. One objective of the model was to predict the full load-deflection curve of the laboratory beams. Using the nonlinear properties of the constituent materials, the moment-curvature relationship of a section is first determined. The moment-curvature relationship, together with the beam geometry and loading configuration, is then used to determine the load-deflection curve of the beam.

The model follows in some respects work by Plevris and Triantafillou (1992), Bazan (1980), and Buchanan (1990). The numerical model was implemented using two computer programs. The first computer program determines the moment-curvature relationship of a FRP-glulam section. The second computer program uses this moment-curvature relationship to determine the load-deflection curve for a beam loaded in four-point bending.

Comparison of Numerical and Experimental Results

To illustrate the application of the numerical model, the numerical and experimental load-deflection curves for reinforced beam 1 are compared in Figure 4. Also, the actual and predicted ultimate loads and deflections for the three medium (M) quality beams (beams 1, 5 and 9) are compared in Table 4.

Figure 4 - Comparison of Experimental and Numerical Load-Deflection Curves for Beam 1

Table 4 - Comparison of Experimental and Numerical Results for Beams 1, 5 and 9

Beam Type	Analytical Load (lb)	Experimental Load (lb)	Error (%)	Analytical Deflection (in)	Experimental Deflection (in)	Error (%)
Beam 1	17,430	16,250	7.3	4.43	4.48	-1.1
Beam 5	15,990	14,310	11.7	2.96	3.05	-3.0
Beam 9	10,240	10,780	-5.0	2.52	3.08	-18.2

Discussion

Glulam made from one of Maine's relatively weak and under-utilized wood species, eastern hemlock, was reinforced with FRP in the tension zone. The reinforced beams performed very well and showed substantial gains in strength (up to 56%) and stiffness (up to 37%) by the addition of 1-3% FRP reinforcement.

The increased beam strength was due in part to the more efficient utilization of the compressive strength of the wood. Using familiar reinforced concrete terminology, the beams tested performed as over-reinforced beams. In an over-reinforced beam, failure occurs by compression of wood fibers near the top of the beam. The region of failed wood in compression propagates from the top of the beam down until the beam ultimately fails. The ductile failure of wood in the compression zone leads not only to increased strength but also to increased ductility of the reinforced beam. It should be noted that in contrast with reinforced concrete, an over-reinforced wood beam is ductile whereas an under-reinforced concrete beam is ductile.

In general, the largest increases in strength were obtained with the lower grades of wood. It appears therefore that the highest value-added benefits resulting from this technology may occur with the lower grades of wood. This is due to the lower grades of wood having a larger difference in relative tension/compression strength values, which can be remedied by adding FRP tension reinforcement.

A nonlinear numerical model was developed to study the ultimate strength behavior of FRP-glulam beams. The model was relatively successful in predicting the performance of the beams. It will be a useful tool in optimizing the lay-up of glulam beams.

Concluding Remarks

Fiber reinforced plastics appear to have good potential to serve as a substitute for the high quality wood laminations necessary in glulam. Placing the FRP in the beam tension zone uses the FRP's high tensile strength and stiffness to boost the strength and stiffness and ductility of the hybrid beam. Commercial success will ultimately depend upon the future savings of removing wood laminations being greater than the future expense of adding FRP reinforcement.

In addition to the cost/benefit issue, further research is necessary before FRP-reinforced wood beams are widely used in bridge applications. One major concern is the long-term durability of the FRP-wood interface in a bridge environment. The in-service hygro-thermal mechanical stresses that will develop at the wood-FRP interface need to be evaluated carefully. In addition, the interaction between moisture, temperature, fatigue, and their effect on bond strength and creep behavior of the system are not entirely understood. Fundamental research at the University of Maine is on-going to address these and other related issues.

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Initial Tests of Kevlar Prestressed Timber Beams

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Abstract

The high strength, high modulus of elasticity, and the increased availability of composite fibers and fiber reinforced plastics makes them a fitting candidate for reinforcement in many structural timber components. Advanced methods of wood construction utilizing composite reinforcement may enable timber to be put to more efficient structural applications. In this work, the authors establish a procedure for reinforcing glued laminated timber beams by bonding woven layers of non-stressed and prestressed bare unidirectional Kevlar fibers between selected laminations. This research evaluates the flexural performance of the reinforced and prestressed beams as compared to similar unreinforced control beams. Additional studies are presented that examine the shear strength of the Kevlar-wood bond, finger joint effects, and time-dependent behavior of the non-stressed and prestressed Kevlar reinforced timber beams.

Keywords: glulam, timber, beams, prestressed, Kevlar, FRP

Introduction

The ability to utilize laminated wood members for certain structural applications is often limited by their relatively low bending strength and stiffness when compared to other materials such as concrete and steel. A possible method for improving these properties is to use high strength fiber reinforced plastic (FRP) to reinforce timber glued laminated timber (glulam) members. Advances in fiber-reinforced plastics coupled with the increased availability of synthetic fibers have made fiber reinforced wood composites a viable alternative for reinforcing and prestressing timber. Glulam beams reinforced with FRP materials, when designed to fail plastically, generally exhibit higher bending stiffness, bending strength, and ductility while also displaying a reduction in mechanical variability. In addition, initial prestress of the member by pre-tensioning the FRP reinforcement may further increase the bending strength of the member. Initial prestress of the member may be used to control deflections and tension failures in much the same way it does for prestressed concrete. These advances in the structural properties and behavior of glulam beams may enable smaller wood members or members with lower grades of wood to be substituted for larger members made completely of wood.

Past research indicates that as the grade of lumber decreases in glued laminated timber beams, the beams

become more dependent on the tensile strength of the wood rather than the compressive strength. Therefore, the placement of reinforcement with a high modulus of elasticity and high tensile strength in the tension zone of flexural members may improve the flexural strength, stiffness, ductility, and potentially lower the mechanical variability of beams when compared to unreinforced beams. Generally, to achieve a higher flexural strength and ductility, the reinforcement ratio and placement of the reinforcement are designed so that the beams have the equivalent of a very high tensile lamina. This shifts the failure to wrinkling in compression zone prior to failure of the tension laminates.

In addition to non-stressed reinforcement, prestress has been utilized in flexural timber members to allow such members to develop their full bending strength. Through the prestressing process, stresses are induced in the member to offset a proportion of the stresses due to an applied load. One method of prestressing that may produce these desired stresses is to place highly stressed Kevlar reinforcement longitudinally between laminations. The prestressed reinforcement is placed in the area where tensile stress develops in the beams under expected loading. By placing the prestress in the tension zone of glued laminated beams, the prestress force creates compressive stress in the tension zone. The compressive stress helps to counteract tensile stress expected under loading. Figure 1 illustrates Kevlar reinforced and prestressed glued laminated timber beams.

Figure 1 - Kevlar Reinforced and Prestressed Glued Laminated Beams

Research Objectives and Approach

This research examines the performance of glued laminated timber beams reinforced with non-stressed and prestressed Kevlar reinforcement. The primary objective is to determine how the Kevlar reinforcement and prestress affect the flexurat strength and stiffness of glued laminated beams. The flexural stiffness and strength of both non-stressed and prestressed Kevlar reinforced glulam beams are compared against non-reinforced control beams. Other research objectives include the bond strength of the Kevlar fiber reinforcement to wood interface, finger joint effects, and the time-dependent behavior of prestressed Kevlar reinforced timber beams.

In order to achieve these objectives, a series of prestressed reinforced, unstressed reinforced and control beams were constructed and tested in flexure. The test matrix, shown in Figure 2, illustrates the different types of beams that were tested. Glued laminated beams with one and two layers of prestressed reinforcement were evaluated. In addition, a set of beams with non-stressed Kevlar reinforcement was also evaluated. Most of the beams were constructed with No. 2 grade Southern Pine (SP) lumber. However, one set of beams was tested that included one layer of prestressed Kevlar glued between two No. 1 Grade Southern Pine laminations on the bottom of the beam.

The beams were constructed from 9 laminations of nominal 51 mm by 102 mm Southern Pine lumber. The test beams have a width of 90 mm, a height of 314 mm, and a length of 5.49 m. Indspec R600 adhesive was used in the manufacture of the beams to glue laminations together. The layers of Kevlar reinforcement consist of a woven tape of bare unidirectional Kevlar fibers. The area of one layer of Kevlar reinforcement is 29.4 sq. mm. The prestressed beams were manufactured by pre-tensioning the layer of Kevlar reinforcement prior to gluing it between selected laminations in the beams. Typical prestressing forces range from 40-47 kN for beams with one layer of prestressed reinforcement and from 67-79 kN for beams with two layers of prestressed reinforcement.

Flexural Tests

The beams were tested according to the flexure test standard specified by the ASTM D 198-84 “Standard Method of Static Tests of Timber in Structural Sizes”.

Figure 2 - Test Matrix of Reinforced, Prestressed, and Control Beams

The test setup is illustrated in Figure 3. The beams are supported by pinned and roller supports at the ends. Lateral supports prevent any lateral torsional buckling in the beams as they are tested. The load was determined using a 245 kN load cell and recorded by using the Keithley data acquisition system. The deflection was found by using a direct current displacement transducer (DCDT). The curvature was determined by mounting three DCDTs longitudinally in the constant moment section on one side of the beam. The data from the DCDTs were also recorded by using the Keithley data acquisition system. All beams were fabricated and tested at a moisture content less than 15%. The load, P, was then used to calculate the moment. The moment, rotational strains, and deflection were recorded for each beam.

Figure 3 - ASTM D 198 Flexural Test Setup

The of the flexural tests compared the ultimate moment capacities of the prestressed and non-stressed reinforced Kevlar beams to similar unreinforced control beams. In addition to the moment, rotation, and deflection data, the failure mode, the stresses at failure, and the initial camber of the prestressed beams were measured.

Shear Test of Kevlar-Wood Interface

In addition to the flexural tests, a series of shear tests were conducted to determine the bond strength of the FRP-wood interface. These tests were conducted in accordance with ASTM D 905 - 86 "Standard Test Method for the Strength Properties of Adhesive Bonds in Shear by Compression Loading." Several shear block specimens were constructed by gluing blocks of wood together using several different adhesives. A compressive force was then applied to shear the blocks apart. In addition, shear block specimens without Kevlar and with both non-stressed and prestressed Kevlar were also constructed. The objective of the shear test research is to determine which adhesive

produces the highest shear bond between the layer of Kevlar reinforcement and the wood. Results from this research also determine the limiting level of initial prestress that can be applied so that the Kevlar-wood bond will not fail due to the shear under expected loads.

Finger Joint Test

To determine the effect of the finger joints on the flexural performance of the Kevlar non-stressed and prestressed beams, a series of tensile tests were conducted on the finger joints and the wood in the beams. Tension strips of timber from the beams with and without finger joints were constructed and tested in accordance with ASTM D 4688 "Standard Test Methods for Evaluating Structural Adhesives for Finger Jointing Lumber." This test gives higher strengths for clear lumber than a full size finger joint, however, the test has the benefit that samples may be prepared from the actual beams. The tensile strength of the finger joints was compared to the tensile strength of the clear wood and wood with knots.

Sample sets of tension strip specimens that failed due to knots, finger joints, and clear wood failure were prepared and tested, The sample size of tension strip specimens that failed at a knot, a finger joint, and in clear wood were 45, 40, and 80 specimens, respectively. From this data, the comparative tensile strength of the knots, finger joints and clear wood in the beams was determined. In addition, a sample set intended to represent the timber in the beams was computed. This sample set combines an appropriate percent of knot and clear wood test specimens and represents the percentage of knots in the beams. The tensile strength of the representative wood sample set of tension strips was then compared against the tensile strength of the finger joint sample set. From this data, more information was determined as to how the finger joints and strength reducing characteristics of the lumber (such as knots) affected the flexural performance of the beams.

Time-Dependent Properties

Time-dependent properties, such as the creep deflection and the loss of prestress force, are vital to the performance of the non-stressed and prestressed Kevlar reinforced glued laminated timber beams. To address these concerns, tests were conducted to evaluate the midspan deflection and the strain in the bond line between the Kevlar and the wood. These parameters beams were measured over an extended period of time.

Midspan Creep Deflection Test - The mid-span creep deflection of both prestressed beams with one layer of prestressed Kevlar reinforcement (P1/T0) and corresponding control (R0/T0) beams loaded with a uniform load were measured over a period of 245 days. Three prestressed beams (Pl/T0) and three control (R0/T0) beams were loaded with a uniform load of 1.68 kN/m. All six beams spanned 5.49 m between simple supports. See Figure 4 for the midspan creep deflection test setup.

Figure 4 - Midspan Creep Deflection Test Setup

Dial gauges were used to measure the midspan deflection of the beams. Initial deflections due to the application of the load were measured. Then the deflection was continually measured over a period of 245 days after the application of the load. Both the initial and long-term deflections of the prestressed beams were compared to the control beams.

Bond Line Strain Test - In addition to the midspan deflections, the strain in the bond line between the prestressed Kevlar and the timber was evaluated over an extended period of time. Strain gauges were placed on the top of the bottom wood lamination of one prestressed beam with one layer of Kevlar reinforcement (P1/T0), Figure 5. A separate strain gauge was placed on a glue line in a small wood sample to serve as a control for moisture change and glue effects. The strain gauges measured the strain in the wood at the interface between the prestressed Kevlar layer and the wood. Strain gauges were placed at several points along the length of the beam from the end of the beam to midspan of the beam. The strains

were measured before and after the release of the prestress into the beam. Then the beam was loaded with a uniform load identical to the test setup shown in Figure 4. The strains were measured before and after the application of the load. The strains were continually measured over a period of 245 days after the application of the uniform load.

Strain gauge # 1 is located near the end of the P1/T0 beam while strain gauge # 10 is located near midspan, The beam was placed on the simple supports so that strain gauges # 1 through # 10 were located 25 mm, 89 mm, 165 mm, 241 mm, 394 mm, 546 mm, 851 mm, 1.156 m, 1.765 m, and 2.832 m from the end of the beam, respectively.

Figure 5 - Location of Strain Gauges on Prestressed (P1/T0) Beam.

Test Results

Flexural Test Results

The ultimate moment capacities determined experimentally from the ASTM D 198 flexure test do not match well with predicted ultimate moment capacities of the glued laminated beams based on the mean tensile strength of SP lumber provided by the Forest Products Research Lab. The ultimate moment capacities of the prestressed beams without tension laminations (P1/T0 and P2/T0) are much lower than predicted. Alternatively, the non-stressed reinforced beams (R1/T0) and the prestressed beams with one layer of Kevlar reinforcement and two grade one tension laminations (Pl/T2) exhibited a significant increase in bending strength compared to the anticipated strength gain. The prestressed beams with number 1 grade SP also exhibited larger strength gains than originally anticipated. These strength gains were determined by comparing the average ultimate moment capacities of the non-stressed and prestressed Kevlar reinforced timber beams to the ultimate moment capacities of the comparative unreinforced control beams. Table 1 compares the experimentally determined strength gains and the predicted strength gains.

Table 1 - Strength Gains from the ASTM D 198 Flexure Test Compared to Predicted Strength Gains

Beam Type	Average Strength Gain (%) (From Flexure Test)	Predicted Strength Gain (%) (Based on FPL mean values for SP lumber)
P1/T0	1.3	20
P2/T0	16.3	36
R1/T0	22.9	3
P1/T2	24.3	13

One possible explanation for the deviation of experimental results from the theoretical predictions is the influence of finger joints and knots. Every beam failed within the elastic region due to a tension failure of the bottom laminations. Results indicate that 15 of 28 test beams failed at a finger joint on the bottom lamination, 5 of the 28 test beams failed at a knot on the bottom lamination, and 8 of the 28 test beams failed on the bottom lamination due to clear wood tension failure. The modulus of rupture for knot failure is lower than the modulus of rupture for finger joint failures of the bottom lamination. The modulus of rupture for the clear wood failure of the bottom lamination is greater than both the modulus of rupture for both knot and finger joint failures. Table 2 provides failure modes and corresponding moduli of rupture from the flexural tests.

Table 2 - Failure Mode of Bottom Lamination and Corresponding Moduli of Rupture

Failure Mode	Modulus of Rupture (MPa)	No. of test beams that failed (#/28)
Knot	32.0	8
Finger Joint	35.6	15
Clear Wood Tension	42.3	5

The difference in failure mode, clear wood, finger joint, or knot partially explains why the experimental strength gains did not match the predicted strength gains. The average ultimate capacities of the seven P1/T0 beams was compared to the average moment capacity of the six R0/T0 beams. However, the failure modes of the seven Pl/T0 beams were different than that of the R0/T0 control beams, Table 3. All of the seven Pl/T0 beams failed due to a knot or a finger joint on the bottom lamination. Alternatively, of the six R0/T0 control beams, 1 failed at a knot, 3 at a finger joint and 2 due to clear wood tension on the bottom lamination. Since the two sets have different failure modes, it is difficult to compare the average strengths.

Table 3 - Failure Modes of Bottom Lamination for the Control, Non-stressed, and Prestressed Beams

Beam Type	No.	Knot Failure	Finger Joint Failure	Clear Wood Failure
P1/T0	7	3	4	0
P2/T0	4	0	3	1
P1/T2	3	0	1	2
R1/T0	4	0	2	2
R0/T0	6	1	3	2
R0/T2	4	1	2	1

To reduce the variability of data due to different failure modes, comparison of the beams that only failed at the finger joints was conducted. The predicted strength was based on the FPL mean values for SP lumber and were corrected to the stress levels determined by the strip finger joint tests. Restricting the data sample to just these beams provides very good correlation between actual strength gains and predicted strengths gains, Table 4. In analyzing Table 4, only 15 beams were in the sample set, so drawing statistically valid conclusions is not warranted.

Table 4 - Strength Gains from the ASTM D 198 Flexure Test Compared to Predicted Strength Gains for Beams Failing at Finger Joints Only

Beam Type	Average Strength Gain (%) (From Flexure Test)	Predicted Strength Gain (%)
P1/T0	30	25
P2/T0	29	44
P1/T2	9	5

Finger Joint Test Results

Tension strip tests conducted on sample sets of specimens with knots, finger joints, and clear wood appear to validate the findings from the modulus of rupture. There is a difference in the tensile strength of knots, finger joints, and clear wood. In addition, the variation and standard deviation of the tensile strength of the finger joints is much less than the variation in the tensile strength of the knot and clear wood sample sets, Table 5. The average tensile strength of a "typical" wood sample set was established by combining proportional parts of clear, finger joint, and knot samples. The difference in tensile strength between the "typical" wood in the beams and the finger joints is approximately 7.6 MPa.

Table 5 - Tension Strip Test Results

Sample Group	No.	Average Tensile Strength (MPa)	Standard Deviation (MPa)
Knot	45	21.7	11.8
Finger Joint	40	31.2	6.7
Clear Wood	80	48.9	14.5
"Typical" Wood	125	39.1	18.9

Shear Strength of Kevlar-Wood Interface Results

ASTM D 905 shear tests reveal that the shear strength of the Kevlar-wood interface decreases with increased pretension force in the Kevlar. Shear block specimens with Kevlar tape glued between them had a higher shear stress than shear block specimens without Kevlar, if the pretension is low (0-7.5 kN). However, as the pretension in the Kevlar tape increases (20-52 kN) the shear strength is lower than the shear tests without Kevlar. Tables 6 and 7 present shear strength results.

Table 6 - ASTM D 905 Shear Stress Tests Without Kevlar

No. Tested	Adhesive (Hardener)	Average Stress (psi)	Shear (MPa)
13	Indspec R600 (H30M)	715	4.9
15	Borden LT-75 (FM260)	574	4.0
5	Indspec R600 (H30M)	634	4.4
5	Borden LT-75 (FM 260)	421	2.9

Table 7 - ASTM D 905 Shear stress tests with Kevlar

No. Tested	Adhesive (Hardener)	Kevlar Pretension (kN)	Average Shear Stress (MPa)
5	Indspec R600 (H30M)	0	7.8
5	Borden LT-75 (FM260)	0	8.5
10	Indspec R600 (H30M)	7.5	5.6
10	Borden LT-75 (FM260)	6.4	5.6
9	Borden RS 240 MD (FM 124D)	7.5	6.2
8	Indspec R600 (H30M)	20.0	2.9
9	Indspec R600 (H30M)	52.3	2.3

All adhesive tests were conducted following the manufacturer's recommendations for proportioning and mixing. No attempts were made to modify or optimize the mix proportions.

Midspan Creep Deflection Test Results

The midspan creep deflection test results are shown in Figure 6. The initial deflections of the beams after the uniform load was applied were approximately the same for both the prestressed and the control beams, Table 8. In addition, the average creep deflections for both the prestressed and the control beams are similar. However, the prestressed beams had an initial camber so that the relative deflection from horizontal for the P1/T0 beams is less than that of the R0/T0 control beams..

Figure - 6 Midspan Creep Deflection Test Results

Figure - 7 Bond Line Strain Test Results

Table 8 - Initial, Total, and Creep Deflections of Prestressed and Control Beams Due to Uniform Loading

Beam Type	Initial Deflection due to Uniform Load (mm)	Total Deflection after 245 days (mm)	Creep Deflection (mm)
P1/T0	7.93	16.9	8.97
P1/T0	8.41	16.2	7.82
P1/T0	8.92	20.7	11.79
R0/T0	9.63	12.6	3.00
R0/T0	9.30	23.1	13.82
R0/T0	8.59	20.7	12.14

The average initial deflection of the prestressed (P1/T0) beams due to the application of the uniform load is 8.42 mm while the average initial deflection of the control (R0/T0) beams is 9.17 mm. The average creep deflection of the prestressed (Pl/T0) beams due to the application of the uniform load is 9.53 mm while the average initial deflection of the control (R0/T0) beams is 9.65 mm. (See Table 7) However, the average initial camber of the prestressed beams is 7.78 mm. Therefore, the total deflection from horizontal for the prestressed beams is 10.8 mm while the average total deflection from horizontal for the control beams is 18.82 mm, Table 9.

Table 9 - Initial Camber and Total Deflection from Horizontal for Prestressed and Control Beams

Beam Type	Initial Camber due to Prestress (mm)	Total Deflection after 245 days (mm)	Total Deflection from horizontal after 245 days (mm)
P1/T0	8.13	16.9	8.76
P1/T0	3.20	16.2	14.86
P1/T0	12.01	20.7	8.69
R0/T0	-	12.6	12.62
R0/T0	-	23.1	23.11
R0/T0	-	20.7	20.73

Bond Line Strain Test Results

The bond line strain test results are shown in Figure 7. Initial strain readings before and after the release of the prestress were highly variable due to the large strains, the possibility that the glue was not fully cured, and the vibrations in the manufacturing plant. Therefore, the

accuracy of the initial strain readings are questionable. However, once the beams were returned to the University of Wyoming structural research lab, the strains became more constant. From Figure 7, it can be seen that after the application of the uniform load, the strains in all of the strain gauges increased. After the application of the uniform load, approximately 15 days after the release of the prestress into the beam, the strains in the wood near the prestressed Kevlar layer became fairly constant and remained constant for the remainder of the test.

Conclusions

Flexural Strength

The coefficient of variation in the test results and the small number of total samples must be considered when evaluating the following conclusions. Examining the entire data set of 28 beams indicated that the flexural strength gains determined experimentally did not agree well with predicted results. This was due to the influence of the finger joints and knots. Every beam tested failed in an elastic-type tension failure of the bottom lamination. There is a distinct difference between the ultimate moment capacity and the moduli of rupture for beams that failed due to a knot, finger joint, or clear wood failure of the bottom lamination. The Pl/T0 beams failed due to knots and finger joints and had lower moduli of rupture than that of the comparative control beams. Examination of beams that failed only at the finger joints suggest that the strength gain predictions were more reliable than an examination of the total test data and that the prestressing did strengthen the beams.

Shear Strength of Kevlar-Wood Interface

ASTM D 905 shear tests reveal that the shear strength of the Kevlar-wood interface decreased with an increase of the pretension force in the Kevlar. Shear block specimens with Kevlar tape glued between them show a higher shear stress than shear block specimens without Kevlar, if the pretension is low (0-7.5 kN). However, as the pretension in the Kevlar ribbon increases (20-52 kN) the shear strength is lower than the shear tests without Kevlar. Therefore, as the prestress is increased, the shear strength of the wood-Kevlar bond decreases. The initial prestress must not be so high that it decreases the shear strength at the Kevlar-wood layer below expected shear stresses due to the applied loads.

Finger Joint Effects

The finger joint and knots did affect the flexural behavior of the beams. Both the modulus of rupture and the tension strip test results revealed a difference in tensile strength between knots, finger joints, and clear wood. In addition, there appears to be a difference in tensile strength between the actual timber in the beams to the finger joints. The finger joints were weaker than the surrounding wood. This is why many of the flexural test beams failed at a finger joint on the bottom lamination. To account for this weakness in finger joints, additional prestress or additional non-stressed reinforcement is required to strengthen the finger joints.

Time-Dependent Properties

The initial camber in the beams matched what the predicted camber based on the prestress force in the beams. The initial and creep deflections of the beams under the application of the load are approximately the same for both prestressed and control beams. The initial camber of the prestressed beams were retained over sustained loading over an extended period of time. Therefore, the prestress was not lost over time. The prestressed Kevlar reinforcement did not appear to reduce the initial deflections or deflections over time. This is due to the fact that there is only a small difference in stiffness between the P1/T0 beams and the R0/T0 beams because of the small area fraction of Kevlar reinforcement used.

Current and Future Research

Currently, research is being conducted to examine the initial camber and stiffness (EI) of the prestressed beams and correlate this information to the amount of prestress force in each prestressed beam. In addition, examination of weak axis bending (sweep) due to accidental eccentricity of the Kevlar layer in the weak direction in the beams is being investigated. Future research must address the minimum reinforcement and prestressing levels to effectively control the tensile failures of these beams.

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Reliability Analysis of Plank Decks for Bridges

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Abstract

The major parameters which determine performance of wood structures are modulus of rupture (MOR), modulus of elasticity (MOE), and loads. These parameters are a subject to considerable variation and therefore, they are treated as random variables. The objective of this study is to develop an efficient procedure for evaluation of the structural reliability. The work will focus on wooden plank decks for highway bridges. In particular, the paper is focused on distribution of truck load to plank decks. The current AASHTO Specifications are unrealistic and too conservative in most cases. A plank element is modeled as a beam on elastic supports. Material properties, in particular MOE and MOR, are based on the actual test data. The major part of the load model is axle load, and in particular wheel load. The statistical analysis is based on the available weigh-in-motion measurements data. It was observed that truck parameters are strongly site-specific. For plank decks, live load consideration is focused on axle weights and wheel loads rather than whole vehicles. The reliability model will be used to develop rational design criteria for plank decks.

Keywords: Plank decks, Bridges, Reliability analysis, Wooden structures.

Introduction

The objective of this study is to develop an efficient procedure to perform reliability based evaluation of plank decks for highway bridges. In particular, it has been observed that the current AASHTO Specifications (1992, 1994) provisions for load distribution for plank decks are not realistic. The problem was identified by the AASHTO Subcommittee on Bridges, Committee on Timber Bridges, as a priority item requiring an urgent solution.

A typical plank deck consists of planks placed on stringers as shown in Figure 1 and 2. There are two categories of plank decks depending on the direction of planks vs. the direction of traffic: transverse deck and longitudinal deck.

For a typical transverse plank deck the span of the deck is perpendicular to the direction of traffic. The typical span length of stringers is usually 5-6 m (16-20 ft). In older structures it can be up to 11 m (36 ft). Stringers are spaced at, s = 300-600 mm (12-24 in) center-to-center, mostly 300-450 mm (12-18 in). Stringers are made of sawn lumber, typical Southern Pine size 150x450 mm (6x18 in), or Douglas-Fir (larger size).

Figure 1 - Typical Transverse Plank Deck

The planks are typically 100x250 mm (4x10 in) or 100x300 mm (4x12 in), and their length can be 3.5-11 m (12-36 ft). Planks are nailed to stringers.

A longitudinal plank deck, as shown in Figure 2, is placed parallel to the direction of traffic. The major design parameter determined by the designer is the spacing between stringers.

The paper deals with the current design provisions for plank decks. It is assumed that stringers have an adequate load carrying capacity and that they provide a sufficient support for planks. The design of stringers is not considered.

The evaluation procedures for plank decks are developed assuming that the design truck load is as specified in the code. Two codes are considered: AASHTO (1992) and LRFD AASHTO (1994).

The study is focused on distribution of the truck load to plank decks. A plank element is modeled as a beam on elastic supports. It is assumed that the elastic support parameters vary depending on stiffness of

Figure 2 - Typical Longitudinal Plank Deck

individual stringers. Material properties, in particular the modulus of elasticity (MOE) and the modulus of rupture (MOR), are based on the actual test data. Statistical models for MOR and MOE for various wood species and sizes were derived by Nowak (1983).

The major part of the load model is axle load, and in particular wheel load. The statistical analysis is based on the available weigh-in-motion measurements data (Nowak et al. 1994). The contact pressure between truck tire and road surface are modeled using the available literature.

The current design criteria's in AASHTO (1992) and LRFD AASHTO (1994) were evaluated using the procedure described in the paper.

Current Design Provisions

Two design codes are considered: AASHTO (1992) and LRFD AASHTO (1994). Both codes specify live load as HS-20 truck. The resulting design wheel load is 16 kips.

AASHTO Specifications (1992)

The design formula for a plank deck is

$$[D + L\ (1 + I)]/S < F_b \qquad (1)$$

where D = dead load moment, L = live load moment, I = impact factor, S = section modulus, F_b = allowable stress. Dead load is very small compared to live load. The live load is specified as the moment due to wheel load. Impact is not considered for wood components. The section modulus is

$$S = b_p\ d_p^2/6 \qquad (2)$$

where b_p = plank width and d_p = plank depth (thickness), in most cases it is 90 mm (3.5 in). The wheel load associated with HS-20 truck is 70 kN (16 kips). The tire contact area is defined as a rectangle shown is Figure 3. The length in direction of traffic, L_t, and width of tire, b_t, can be determined from the following two equations,

$$L_t\ b_t = 0.01\ P \qquad (3)$$
$$b_t/L_t = 2.5 \qquad (4)$$

where P **z** wheel load in pounds. For P = 70 kN (16,000 lb), L_t = 200 mm (8 in) and b_t = 500 mm (20 in).

It is assumed that wheel load is distributed depending on the direction of planks in relation to the direction of traffic. For transverse planks, it is assumed that the

Figure 3 - Tire Contact Area

wheel load is uniformly distributed over the contact area defined as a rectangle shown in Figure 3. The live load moment is calculated for a simply supported beam, as shown in Figure 4(a), with a span equal to clear distance between the stringers plus one-half the width of one stringer. For continuous planks, the moment is taken as 0.8 of the calculated value.

Figure 4 - Load Distribution (AASHTO - 1992)

For longitudinal planks, in the direction of span, the wheel load is considered a point loading, and normal to the direction of span, the wheel load is distributed over the width of plank (Figure 4(b)). The live load moment is calculated for a simply supported beam, with a span equal to clear distance between the stringers plus one-half the width of one stringer. For continuous planks, the design moment is taken as 0.8 of the calculated value.

Following the AASHTO (1992) provisions, the maximum stress due to live load is calculated for stringer spacing from 300 mm through 600 mm (12 in through 24 in), and for plank widths 100 through 300 mm (4 in through 12 in). For transverse plank decks the maximum bending stress ranges from 1.7 MPa to

34.5 MPa (250 psi to 5000 psi). For longitudinal planks the stresses are as much as 66 MPa (9500 psi) for smaller deck sizes.

For comparison, the allowable stress for Douglas-Fir Grade 1 is 12.4 MPa (1798 psi) including a flat-wise use factor of 1.11 and a moisture compensation factor of 1.08 (from AASHTO 1992).

LRFD AASHTO Specifications (1994)

The design formula for a plank deck is

$$1.25\ D + 1.75\ L\ (1 + I) < 0.85\ F_b\ S \qquad (5)$$

where D = dead load moment, L = live load moment, I = impact factor, S = section modulus (Eq. 2), F_b = nominal resistance (stress). Dead load is practically negligible. Live load is specified as the moment due to HS-20 truck wheel load. For longitudinal planks, live load includes the HS-20 truck load and lane load of 9.3 kN/m (640 lb/ft). Impact is 0.33 of truck load effect, but for wood components it is reduced by 50 percent, therefore, I = 0.165.

The wheel load is 70 kN (16 kips). The tire contact area is defined as a rectangle shown is Figure 3. The width of tire, b_t, is 500 mm (20 in) and the length in direction of traffic, L_t, can be determined from the following equation,

$$L_t = 1.75\ (1 + I)\ P / 2.5 \qquad (6)$$

where P is the wheel load, for P = 70 kN (16 kip), L_t = 330 mm (13 in). The wheel load is distributed depending on the direction of planks in relation to the direction of traffic.

For transverse planks, shown in Figure 5(a), it is assumed that the wheel load is uniformly distributed over the area of a rectangle with the length of 500 mm (20 in) plus the thickness of plank (in the direction of span) and width equal to plank width. For longitudinal planks, the wheel load is distributed over the area of a rectangle with the length 330 mm (13 in) plus the plank thickness (in the direction of span) and width equal to plank width as shown in Figure 5(b).

The live load moment is calculated for a continuous beam, with each span equal to the center-to-center distance between the stringers.

Following the LRFD AASHTO (1994) provisions, the maximum bending stress due to live load is calculated for stringer spacing from 300 mm - 600 mm (12 in through 24 in), and for plank widths 100 mm - 300 mm (4 in through 12 in). The stresses in transverse

Figure 5 - Load Distribution (LRFD AASHTO - 1994)

decks vary form 2.4 MPa to 36.2 MPa (350 psi to 5250 psi). For longitudinal decks the stresses are ranging from 4.5 MPa to 50 MPa (650 psi to 7200 psi).

The nominal resistance, F_b, is a product of base resistance and the size effect factor, C_F. C_F is taken to be 1.54 for flat-wise planks. For comparison, the base and nominal resistance's for Douglas-Fir Grade 1 and 2 are 19 MPa and 30 MPa (2800 psi and 4310 psi), respectively (from LRFD AASHTO 1994),

Reliability Analysis

The structural performance of plank decks is determined by loads and resistance. Both load and resistance are random variables. Therefore, reliability analysis is used to evaluate the current design provisions. The reliability index, β, is considered as the measure of safety.

Load Model

The live load model is developed on the basis of actual truck measurements. Extensive weigh-in-motion (WIM) measurements were carried out by researchers at the University of Michigan (Nowak et al. 1994). The study provided statistical data on gross vehicle weights (GVW), axle weights and axle spacings. It was observed that truck parameters are strongly site-specific.

For plank decks, live load consideration is focused on axle weights and wheel loads rather than whole vehicles. The cumulative distribution functions of axle weight from citation data are plotted on normal probability paper in Figure 6. The vertical axis is the

Figure 6 - Cumulative Distribution Function of Axle Weight

inverse normal distribution function and it corresponds to the probability of being exceeded. The measured axle weights vary from site-to-site. The average weights are ranging from 40 to 55 kN (9 to 12 kip). The maximum values were observed on 13 bridges. They vary form 95 to 220 kN (21 to 50 kip). The cumulative distribution function of the maximum axle weights for the considered bridges is also plotted on normal probability paper in Figure 6. Based on these results, the expected mean maximum live load is a wheel load of 51.2 kN (11.5 kip) distributed over the rectangular contact area. The coefficient of variation is 14 percent without impact, which could be reduced for smaller size planks to account for better load sharing. The wheel load for a dual tire assembly is taken to be 100 kN (23 kip).

Contact Area

Dimensions of the contact area are based on the available literature (Pezo et al. 1989 and Kim et al. 1989). The length of the contact area is 250 mm (10 in) in the direction of traffic and the width is 200 mm (7.5 in) in the other direction. The pressure under tire is 1 MPa (150 psi). The contact area for a two wheel unit weighing 100 kN (23 kip) is shown in Figure 7.

Load Distribution

For transverse planks (planks perpendicular to the direction of traffic), the wheel load is applied as a uniform pressure, over the area shown in Figure 7. If plank width is larger than the length of contact area, 250 mm (10 in), then it is reasonable to assume that the load is distributed over the whole plank width. If the plank width is less than 250 mm (10 in), then the plank takes only a portion of wheel load proportional to the ratio of plank width and 250 mm (10 in).

For longitudinal planks (planks parallel to the direction of traffic), the wheel load is also applied as a uniform pressure, over the area shown in Figure 7. If plank width is larger than the width of contact area, 200 mm (7.5 in), then it is reasonable to assume that the load is distributed over the whole plank width. If the plank width is less than 200 mm (7.5 in), then the plank takes only a portion of wheel load proportional to the ratio of plank width and 200 mm (7.5 in).

In practice, a transverse plank is usually resisting two

Figure 7 - Proposed Tire Contact Area

Figure 7 - Modulus of Rupture for Douglas-Fir Grade 1 and 2

wheel loads, while a longitudinal plank resists only one wheel load.

The live load moment is calculated assuming the plank is a continuous beam on elastic supports. The actual dynamic load is about 0.05 of live load. However, the flexural strength of wood is significantly larger for short duration loads, therefore, dynamic load (impact) is not considered in the analysis.

The calculations were carried out for a wide range of stringer spacings and plank widths. The flexural stresses due to a dual tire assembly with 100 kN (23 kip) on transverse planks vary from 5.5 MPa to 23.5 MPa (800 psi to 3400 psi). For longitudinal planks it ranges form from 5.5 MPa to 16.6 MPa (800 psi to 2400 psi).

Resistance Model

The resistance of wood is determined by MOR. For plank decks, the wood sections are used flat-wise. For the considered plank species, sizes and grades, the mean MOR's are available only for edge-wise use. The flat-wise MOR's are extrapolated from the available data, as shown in Figure 7. Experiments are currently being carried out at University of Michigan, to determine the actual statistical properties of wood section for flat-wise use. The resistance (MOR) has a log-normal distribution. The coefficient of variation of MOR is taken to be about 20 percent.

Reliability Analysis

The reliability analysis is carried our using the procedure developed for calibration of the LRFD AASHTO (Nowak 1995). Reliability is measured in terms of the reliability index, β. The analysis is performed for plank decks designed using AASHTO (1992) and LRFD AASHTO (1994).

The dead load moment due to self weight of plank and the asphalt pavement are very small and each is about 10 percent of the total moment. The live load moment is the primary load component and it has a log-normal distribution. Hence, the total moment can also be assumed to have a distribution similar to that of the live load. Since, both the load and resistance are log-normally distributed, the reliability index, β, can be calculated using the following equation

$$\beta = \frac{\ln(m_R/m_Q)}{\sqrt{V_R^2 + V_Q^2}} \tag{7}$$

where, m_R and m_Q are the mean of resistance and load, respectively, and V_R and V_Q are the respective coefficient of variations.

The procedure outlined above can be used to perform reliability based evaluation of the wooden plank decks for bridges. The reliability indices were calculated for current code provisions. They show that the current

provisions have conservative load distribution, specially for the smaller sizes, because the effect of load sharing among adjacent planks is not considered. Therefore, much higher values of reliability indices are obtained for smaller sizes (4"x4" to 6"x4") as compared to larger sizes (10"x4" and 12"x4"). Which penalizes the use of wooden planks with smaller widths. Although, based on reliability analysis, the smaller sizes should be preferred as they provide better load sharing and improved redundancy. New code provisions for the design of wooden plank decks will be developed using the procedure described in this paper.

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Cold Temperature Effects on Stress-Laminated Timber Bridges

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Abstract

Stress-laminated bridges perform well, if adequate bar force is maintained to provide the interlaminar friction and load transfer between adjacent deck laminations. Stress-laminated decks utilize both wood and steel components; therefore, different material thermal properties may cause bar force to change as the temperature fluctuates. In response to concerns about the performance of stress-laminated bridges in extremely cold climates, a cooperative research project with the University of Minnesota; the USDA Forest Service, Forest Products Laboratory; and the Federal Highway Administration was initiated to evaluate system performance at temperatures ranging from 21.1°C to –34.4°C. Stress-laminated bridge deck sections, constructed of Red Pine lumber and high strength steel stressing bars, were placed in cold temperature settings of –12.2°C, –17.8°C, –23.3°C, –28.9°C, and –34.4°C, while bar force measurements were collected. Testing was completed at three different moisture content levels: >30, 17, and 7 percent. At –34.4°C, bar force losses were high when the deck moisture content was greater than 30 percent and were moderate to low when the moisture content was less than 18 percent. In all cases, bar force loss was fully recovered after temperatures increased to 21.1°C.

Keywords: Stress laminated, bridge, timber, Red Pine, temperature, bar force.

Introduction

Stress-laminated bridge decks consist of a series of wood laminations that are placed edgewise between supports and stressed together with high strength steel bars (Ritter 1990). The bar force, which typically ranges between 89 and 356 kN, squeezes the laminations so that the stressed deck acts as a solid-wood plate. (See Table 1 for English conversion factors.)

Stress-laminating technology was originally developed in Ontario, Canada, in 1976 as a means of rehabilitating nail-laminated lumber decks that delaminated as a result of cyclic loading and variations in wood moisture content. In the 1980s, stress-laminating technology was adapted for the design and construction of new bridges, and several structures were successfully built in Ontario using the stress-laminating concept.

Table I—Factors for converting SI units of measurement to English units

SI unit	Conversion factor	English unit
millimeter (mm)	0.0393	inch (in.)
meter (m)	3.2808	foot (ft)
newton (N)	0.2248	pound (lb)
pascal (Pa)	0.0001451	lb/in^2 (stress)
°C	1.8(°C) + 32	°F

The first stress-laminated bridges in the United States were built in the late 1980s. Since then, several hundred stress-laminated timber bridges have been constructed throughout the United States, primarily on low-volume, rural roads. As a result of the experimental nature of the new bridge system, extensive field evaluations were implemented by the USDA Forest Service, Forest Products Laboratory (FPL), and the Federal Highway Administration (FHWA) to evaluate long-term field performance (Hislop and others 1996; Kainz, 1996; Ritter and others 1995; 1996a,b; Wacker and others 1996; Wacker and Ritter 1992; 1995a, b).

Preliminary data from several field bridges in the FPL/FHWA monitoring program indicate that bar force decreases when the temperature decreases. The magnitude of this decrease depends on a number of factors, including temperature decrease, duration of cold temperature, wood species, and moisture content. Short-term temperature declines, 24 hours or less, seem to have little effect on bar force because thermal conductivity of wood is very low. This cold temperature effect appears to be fully recoverable, and the bar force returns to the original level when the temperature increases.

Several stress-laminated bridges are located in northern climates where sustained cold temperatures are not uncommon. Therefore, concerns were raised that bar forces may temporarily drop below safe levels during the winter months. To further investigate this cold temperature effect on stress-laminated bridges, laboratory studies were necessary to evaluate the effect of subfreezing temperatures on bar force levels. This study, in conjunction with forthcoming field results, will determine if stress-laminated bridges in cold climates require special design considerations.

Background

Thermal effects are usually not a consideration in the design of wood highway bridges. However, some bridge designs use materials that have unique thermal properties. If not compensated for, any differential between material thermal properties can lead to performance problems when exposed to large temperature changes. In these cases, the design needs to compensate for differential movement of bridge components while preserving structural integrity.

Because wood laminations are compressed with steel bars to form the stress-laminated deck, bar force levels could be adversely affected by large temperature fluctuations. The different materials will expand and contract at various rates. The thermal coefficient for wood is approximately twice that of steel; therefore, temporary temperature-induced bar force losses may occur as both materials contract under cold temperature conditions. This thermal interaction is complicated by the thermal properties of wood, which is dependent upon many factors, including grain orientation. For stress-laminated decks, the grain orientation is randomly mixed between radial and tangential directions, which prevents a simplified mechanical analysis.

Thermal contraction of small, clear wood specimens has been investigated (Kubler 1973). An accelerated rate of thermal contraction was observed for moist samples at temperatures less than 0°C. At temperatures less than 0°C, a type of internal drying occurs and moisture diffuses out of the wet cell walls and condenses as ice crystals in the cell cavities. This internal drying causes additional shrinkage that superimposes with pure thermal contraction as a result of temperature change. Although these tests were not performed on wood members in a stress-laminated configuration, important information about moist wood behavior at cold temperatures was gained.

Other laboratory studies have investigated the effect of cold temperatures on stress-laminated bridge configurations. Pilot studies initiated at the University of Minnesota (Erickson and others 1990) and at the FPL (Kainz 1994) shortly after this phenomenon was suspected in field bridges found that bar forces decreased as the temperature dropped below 0ºC. In addition, moisture content of the wood laminations seemed to influence the magnitude of bar force loss.

Based on these laboratory and field studies, stress-laminated bridges located in U.S. regions where extreme cold temperatures are common were suspected to encounter temporary bar force loss. In response to these concerns, a cooperative study was initiated with the University of Minnesota, FPL, and FHWA to evaluate stress-laminated deck bar forces over a wide temperature range and to further investigate this phenomenon and its possible design implications for cold climates.

Objective

The objective of this research was to determine how bar force levels in stress-laminated decks change when exposed to temperatures between 21.1°C and –34.4°C.

Test Methods

Testing was completed in the laboratory under controlled temperature and moisture content conditions.

Deck sections were sequentially placed in cold settings of –12.2°C, –17.8°C, –23.3°C, –28.9°C, and –34.4°C, while bar forces were monitored. To detect moisture content effect, three different moisture content levels were used in testing.

A typical configuration of the stress-laminated deck section used for testing is shown in Figure 1. Three identical deck sections were constructed to the following dimensions: 3.05 m long, 1.07 m wide, and 305 mm deep. The untreated lumber laminations were full-length Red Pine lumber (No. 2 grade), measuring nominal 51 by 305 mm. The prestressing elements were high strength steel bars, measuring 15.9 mm in diameter, and were spaced at 610-mm intervals, beginning 305 mm from the ends. Bar anchorage was provided with rectangular steel bearing plates and flat hex nuts. To measure bar force levels, three steel hollow-core load cells were placed at the interior bar locations of each deck section, between two steel anchor plates.

Testing Procedures

Testing was completed in three phases using similar procedures (Table 2). The moisture content of the deck laminations differed in each phase and was greater than 30 percent for phase I, approximately 17 percent for phase II, and approximately 7 percent for phase III. For each phase, the same five temperature runs were conducted to increasingly lower temperatures to –34.4°C. All tests were initiated with bar forces at approximately 120 kN, which is equivalent to an interlaminar compression level at 690 MPa. The procedures used during each phase were repetitive and are described as follows.

Load Cell Calibration

A total of nine steel, hollow-core load cells were fabricated in accordance with previously developed specifications (Ritter and others 1991). The cells were designed to compensate for temperature-induced strain variations and were verified at cold temperatures prior to testing. Before installing load cells, they were calibrated for compression loads up to 133 kN. In addition, the load cells were recalibrated with the data acquisition system prior to beginning each test phase.

Table 2—Moisture content and temperature conditions used during study.

Phase	MC (%)	Temperature runs	°C	°F
I	>30	1	21.1 to –12.2	70 to 10
II	17	2	21.1 to –17.8	70 to 0
		3	21.1 to –23.3	70 to –10
III	7	4	21.1 to –28.9	70 to –20
		5	21.1 to –34.4	70 to –30

Wood Conditioning

The Red Pine laminations were purchased in "green condition" with a high variability in moisture content, between 40 and 70 percent. The laminations were air-dried for several weeks in an attempt to condition them to approximately 35-percent moisture content for phase I. Kiln drying was not used initially because of concerns that some laminations would drop below fiber saturation. Therefore, the actual moisture content of the laminations during phase I testing varied between 40 and 60 percent. For phases II and III, kiln drying was utilized to reduce the lamination moisture content to the desired level and provide more uniform lamination moisture content levels. Throughout testing, an electrical-resistance moisture meter and individual lamination weights were used to measure wood moisture content.

To prevent fluctuations in moisture content within each phase, deck laminations were wrapped with polyurethane plastic.

Deck Assembly and Stressing

After the laminations were conditioned to the desired moisture content for each test phase, the deck sections were assembled and the bars were tensioned with hydraulic equipment. To assemble the deck sections, the laminations were placed on edge and stressing bars were inserted through 19-mm-diameter predrilled holes. Thermocouples were embedded at several locations within the deck to provide interior wood temperatures. Bearing plates and anchor nuts were then attached. At the completion of phase I testing, the laminations were numbered and the grain orientation was noted so they could be reassembled in the same manner for future test phases.

After assembly, the stressing bars were tensioned using a hydraulic pump and a single hollow-core ram. Beginning at one end, each bar was tensioned to 120 kN in a sequential manner until the entire deck was stressed together. Plate crushing into the outer laminations was a concern, especially at the high moisture content levels used in phase I, because it would cause bar forces loss and dilute any temperature effect data. To detect wood crushing, the outside laminations were visually inspected after removing bearing plates at the completion of each phase.

Figure 1—Typical configuration of stress-laminated deck section used in testing.

Data Collection

After the decks were stressed together, data collection was initiated. A data acquisition system was used to monitor the bar forces, ambient air temperature, and interior wood temperature at 1-hour intervals for all temperature runs. Data collection typically began several hours before the decks were placed in the freezer room to ensure that the bar force was stabilized at a constant level. By ensuring constant bar force prior to initiating temperature runs, bar forces would be influenced solely by temperature effects. For each temperature run, the decks remained in the freezer until the embedded thermocouples indicated temperature equilibrium. Each temperature run was terminated several hours after removal from cold temperatures when the temperature equilibrated to 21.1°C.

Results and Discussion

Results from all temperature runs for the three phases are summarized in Figure 2. Measured bar force changes from all three deck sections were approximately equal and were averaged for each test phase. To aid interpretation, data were normalized to the same initial bar force level and test duration. For all test phases, the initial bar forces were within 10 percent of the target level (120 kN), which corresponds to 690-MPa interlaminar compression. The test duration for the individual tests varied between 500 and 800 hours, with more time required to cool the decks to lower temperatures. In addition, no crushing was detected on the exterior laminations.

Phase I

Bar force losses were high during phase I, at a moisture content greater than 30 percent. These losses became significantly greater as the temperature decreased from –12.2°C to –34.4°C. For the first temperature run to –12.2°C, bar force losses totaled approximately 40 kN, or 33 percent of the original bar force. Bar force losses for intermediate temperature runs to –17.8°C, –23.3°C, and –28.9°C were 52, 70, and 81 percent of the original bar force, respectively. For the last temperature run to –34.4°C, bar force losses were approximately 102 kN, or 85 percent of the original bar force.

Phase II

Bar force losses were moderate during phase II, at a moisture content of approximately 17 percent. The losses became only slightly greater as the temperature decreased from –12.2°C to –34.4°C. For the first temperature run to –12.2°C, bar force losses were approximately 22 kN, or 19 percent of the original bar force. For the last temperature run to –34.4°C, bar force losses totaled approximately 29 kN, or 22 percent of the original bar force. Bar force losses for temperature runs to –17.8°C, –23.3°C, and –28.9°C were intermediate to these values.

Figure 2—Normalized bar force from the three test phases. (m.c. is moisture content.)

Phase III

Bar force losses were low during phase III, at a moisture content of approximately 7 percent. The losses were essentially constant as the temperature deceased from –12.2°C to –34.4°C. For the first temperature run to –23.3°C, bar force losses totaled approximately 6.7 kN, or 6 percent of the original bar force. For the last temperature run to –34.4°C, bar force losses were

approximately 8.9 kN, or 7 percent of the original bar force. Bar force losses for temperature runs to –17.8°C, –23.3°C, and –28.9°C were intermediate to these values.

Phase Comparison

By comparing the bar forces from all three phases in Figure 2, a distinct moisture content effect is evident. Assuming a constant rate of thermal contraction for the steel bars, the wood laminations seem to contract under cold temperatures at an accelerating rate as the moisture content increases. At the lower moisture content levels of 17 and 7 percent used in phases II and III, the magnitude of bar force losses is small and increases only slightly as the temperature decreases from –12.2°C to –34.4°C. But at a moisture content greater than 30 percent in phase I, the magnitude of the bar force loss is greater and increases rapidly as the temperature decreases from –12.2°C to –34.4°C. Similar results from a study by Kubler (1973) using small wood samples also describe an accelerated rate of thermal contraction at higher moisture content levels.

These results indicate that large bar force losses are possible in stress-laminated bridges located in cold temperatures climates. For these temporary bar force losses to occur, however, the lamination moisture content must be greater than 30 percent and the air temperature must remain below –17.8°C for several days.

Results from monitoring several field bridges located in cold temperature climates have not detected substantial bar force losses as a result of temperature effects. This phenomenon has not been a problem for field bridges because it is very unusual for timber bridges to have a moisture content greater than 30 percent. Current AASHTO bridge specifications require that all lumber be conditioned to a maximum moisture content of 19 percent at installation (AASHTO 1992). In addition, a field study inspected several timber bridges across the northern United States and found that the deck moisture content averaged approximately 18 percent after several years in service (McCutcheon and others 1986).

Additional results from laboratory and field studies will be published soon (Kainz and others [in progress], Ritter and others [in progress]).

Concluding Remarks

Bar forces were monitored on three stress-laminated deck sections as they were placed in cold temperature settings of –12.2°C, –17.8°C, –23.3°C, –28.9°C, and –34.4°C. Testing was completed at moisture content levels of 7, 17, and >30 percent. Based on the bar force measurements, the following conclusions are presented:

- Bar force loss observed at all cold temperature and moisture content conditions was fully recovered at the completion of each temperature run.

- At a given moisture content level, the maximum bar force loss was observed during the coldest temperature drop to –34.4°C.

- The moisture content of the laminations has a significant effect on the magnitude of bar force loss. Maximum bar force loss totaled 85 percent of the original bar force at greater than 30-percent moisture content (phase I), 22 percent at 17-percent moisture content (phase II), and 7 percent at 7-percent moisture content (phase III).

- Although there was a significant reduction of bar forces at temperatures less than –17.8°C and moisture content in excess of 30 percent, there is no indication that this phenomenon has been a problem for existing bridges. Field monitoring studies are currently being conducted by the FPL on several bridges in cold climates and additional findings will be published soon.

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CONVENTIONAL LUMBER DECKING: EFFECT OF BOARD SIZES.

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ABSTRACT

The introduction of stress laminated lumber decks has helped in the elimination of the problems associated with conventional lumber decks. However, there are thousands of conventional lumber decks that will continue to require maintenance. In some other instances, some of these conventional wood bridges may be kept as historical landmarks. One problem that needs attention on these conventional lumber decks is the one caused by dimensional instability of lumber, when used in a varying environmental condition. Proper material selection and proper board placement may help in minimizing the effect of these dimensional instabilities. However, when we have wetting from one side only, there is a cupping tendency regardless of material selection and placement. In this paper results of a study aimed at the determination of forces developed during the cupping of lumber subjected to uni-directional wetting are presented. The forces developed during cupping depends on the depth and width of the lumber.

Keywords: Lumber cupping, lumber decks, dimensional instability, bridges

INTRODUCTION AND BACKGROUND

Sawn lumber plank decks may be very practical in low-volume roads and special use bridges such as those servicing forest plantations. These decks are constructed of lumber planks, 75 to 150 mm (3 to 6 inches) thick and 250 to 300 mm (10 to 12 inches) wide, that are placed flat-wise and spiked to supporting beams (USDA Forest Service 1992).

Dimensional instability of lumber when used in varying environmental conditions is a major problem in any wood structure. Dimensional changes, especially differential changes, may lead to serious aesthetic and structural problems. These problems may range from damage to finishes in buildings, failure of pavements laid over lumber bridge decks, uneven riding surface on unpaved lumber decks, and withdrawal of nails and spikes due to cupping of flat lumber boards. It may be possible to minimize the problem of board cupping by proper selection of material and orientation of the boards. A primary contributing factor to the cupping of wood, when under uniform moisture change is the higher expansion in the annual ring orientation. Flat sawn boards exhibit greater shrinkage on the “back” side, and thus the common rule of thumb is “back side down” for laying boards. Quarter sawn material shows significantly less cupping than flat sawn. Lumber which falls between these two categories will fall between the two in terms of cupping tendency as well. In addition, the size of the lumber plays an important role. As the thickness increases, we tend to get more vertically oriented annual rings, and thus resulting in rhomboid shapes rather than cupping.

When you have non-uniform moisture change, noticeable cupping will occur, regardless of material selection or board orientation. The fibers on the wet side will expand while the fibers on the dry side remain unchanged. This will occur no matter whether the board is a quarter cut or a flat cut.

For example, in the sketch shown below (Fig. 1), the board is laid with "bark" side down. When the upper side becomes wet, say due to rain, the upper side elongates while the lower side, which remains dry, will remain unchanged. This variation in elongation will lead to a cupping tendency up to when the wetness reaches the lower side. It is only when the wetness reaches the lower side will the minimization of cupping by proper material selection and proper board orientation will be of consequence. Therefore, it can generally be urged that when there is differential wetting of flat boards, as may be in exposed lumber decks, large cupping forces will develop no matter what material or board placement is adopted. For properly selected and placed materials, most of these forces will be dissipated once the moisture reaches the other side of the board. If damage to finishes and other components supported by the boards is to be avoided or minimized, adequate connectors should be provided to restrain the cupping up to when the moisture reaches the other side of the board.

Fig. 1 Behavior of a Lumber Board When Wetted from One Side

To the knowledge of the authors of this proposal, there is very limited literature on stresses developed in lumber due to environmental changes and its effect on the performance of structures. Lang et. al. (1994) report on studies conducted to determine the performance of oak strip flooring systems under changing environmental conditions. In these studies, fastener withdrawal forces greater than 800 N are reported. The wood industry has been conducting studies on lumber deformations during the drying process (Hsu 1975). There are similarities between lumber cupping associated with the wood drying process and the moisture variation expected in lumber exposed to annual weather variations.

The board cupping effect, as shown in Fig. 1, coupled with the fact that gaps between the lumber boards will vary with respect to changes in environmental conditions, will cause cracking in a pavement laid on a conventional lumber deck. Thus, it is not practical to lay pavement over a conventional lumber deck. During the cupping process the connectors holding the boards to the stringers will be subjected to withdrawal forces. It can be seen from Fig. 1 that the cupping of a tide board will have more significant impact than a narrower board, and thus requiring connectors that can withstand higher withdrawal forces. There is need for quantifying the forces developed during the differential moisture variation induced cupping. These forces will vary with lumber width, thickness, wood species and the environmental conditions of the deck. The AASHTO Standard Specification for Highway Bridges (American.. 1992) specifies the minimum lumber dimensions (Sect. 13.9.4) while remaining silent on maximum width dimensions.

Recently, a $2.5 million, 300 m (1000 ft) fishing pier in south Florida had to have its lumber decking reconstructed before its commissioning. The original deck was made up of a combination of 3 by 10 and 3 by 12 lumber boards, nailed to stringers spaced at 1150 mm (46 in). Before turning the project over to the owners, there was evidence of lumber distortion and evidence of nail withdrawal. This fishing pier deck, which was evaluated by the author of this paper, is a good example of problems that may result from cupping of lumber due to varying environmental conditions.

CUPPING STUDIES

The problem mentioned above motivated the author of this paper to conduct some studies on the behavior of wide lumber boards subjected to differential moisture variation. In the experiment, the lumber specimens were restrained flat-wise with three sets of frames equipped with proofing rings (see Fig. 2 below).

VIEW B–B

FRONT VIEW OF EXPERIMENTAL SETUP

Fig. 2. A Schematic Diagram of Test Setup for this Study

Two wood screws with large washers were attached to the lower side of the lumber specimen at each frame location. The plunger of the proofing ring rest on a wood screw with a set of washers as shown in Fig 2. The spacing employed on the frames varied as a function of the thickness of the lumber boards.

Simulated rain, in the form of shower sprinklers, was allowed to fall on the upper sides of these specimens. The readings of the various dial gauges were then monitored during the testing of each specimen. The fifteen minute interval monitoring was continued until a stable reading was achieved. At this stage the rain was terminated. Shortly after, the cupped lumber was compressed to its original flat position. This was achieved by inducing a jacking force to the frames. Flatness of the specimen was judged by a beeper like device made to check return contact of the flat posture. The force required to attain this flat posture was then recorded by the proofing ring. Data pertaining to three types of outdoor deck specimens were collected. Their denominations being 1 x 6, 1 x 8 and 1 x 12 specimens. The other specimen examined was that of the 2 x 12 which is sometimes used in the construction of bridge decks.

RESULTS AND DISCUSSIONS

Presented in Fig. 3 are the variation of the forces required to bring the lumber specimens to original position after the cupping process. The values plotted are the average of three data samples. Due to the limited nature of these preliminary results no much statistical analysis can be performed. However, the preliminary trend of the results presented in Fig. 3, suggest a linear variation as the board width is increased. The change of lumber thickness from a nominal dimension of 25 mm to 50 mm (for the 300 mm wide boards) approximately doubles the magnitude of the cupping forces. We can therefore say, as expected, the magnitude of the forces increases with respect to lumber width and lumber depth.

PRACTICAL APPLICATIONS

According to National Design Specifications (NDS) (NFPA, 1991) the allowable nail withdrawal design values are a function of nail type, nail size, specific gravity of the lumber and the condition that the wood has to remain wet. NDS specifies a modification factor of 0.25, to be applied to the tabulated allowable values, for situations where the lumber is subjected to wetting and drying, as may be the case in exposed lumber decks.

Assuming all other factors, such as temperature factors, size factor, et., will not apply, the required connector (nail or lag screw) penetration in the holding (point) end can be computed as follows:

$$L_p = \frac{4F}{P} \qquad (1)$$

where F = cupping force

Lp = required minimum connector penetration in the point side

P = NDS tabulated allowable values

The required minimum penetration length computed according to Eq. 1, for the cupping forces observed in this study, are presented in Table 1 for the different lumber sizes studied. As shown in Fig. 1, only connectors in the middle of the board will resist significant cupping forces. Thus the computed minimum penetration length has to be provided by one or two connectors only. This fact has significant impact on the nature and size of connectors to be specified.

CONCLUSIONS AND RECOMMENDATIONS

From the preliminary results of the presented on going studies, the following conclusions can be drawn:

1) There are significant cupping forces developed when a lumber board is exposed to wetness from one side.
2) The magnitude of the cupping force increase with increases in both lumber width and lumber depth.
3) Lumber boards used in cyclic wetting and drying conditions require large point end penetration lengths, if the board is to be prevented from cupping.

The following recommendations can be put forward:

1) Nails, especially common wire nails, should be avoided as connectors for lumber boards exposed to cyclic wet/dry conditions.
2) There is need to continue studying the cupping behavior of lumber boards and determine “muscle” techniques of holding down the lumber during the short periods when the moisture is not uniform.

Fig. 3 Variation of Forces Produced by Lumber Cupping

Table 1. Required Minimum Penetration Depth Based on NDS Design Approach and Observed Cupping Forces

Connector Type	Connector Size mm(in)	NDS-Tabulated Allow. Values N (lb)	Required Min. Penetration in Holding End for: 25 x 150 mm (in)	25 x 200 mm (in)	25 x 300 mm (in)	50 x 300 mm (in)
	3.0 (0.120)	182 (41)	139 (5.5)	367 (14.4)	681 (26.8)	1294 (50.9)
Threaded	3.4 (0.135)	205 (46)	124 (4.9)	327 (12.9)	607 (23.9)	1153 (45.4)
Nails	3.8 (0.148)	223 (50)	114 (4.5)	301 (11.8)	559 (22.0)	1061 (41.8)
	4.5 (0.177)	263 (59)	96 (3.8)	255 (10.0)	474 (18.6)	899 (35.4)
	5.3 (0.207)	312 (70)	81 (3.2)	215 (8.5)	399 (15.7)	758 (29.8)
	6.4 (1/4)	1157 (260)	22 (0.9)	58 (2.3)	107 (4.2)	204 (8.0)
	7.9 (5/16)	1366 (307)	19 (0.7)	49 (1.9)	91 (3.6)	173 (6.8)
	9.5 (3/8)	1566 (352)	16 (0.6)	43 (1.7)	79 (3.1)	151 (5.9)
Lag	11.1 (7/16)	1758 (395)	14 (0.6)	38 (1.5)	71 (2.8)	134 (5.3)
Screws	12.7 (1/2)	1945 (437)	13 (0.5)	34 (1.4)	64 (2.5)	121 (4.8)
	15.9 (5/8)	2296 (516)	11 (0.4)	29 (1.1)	54 (2.1)	103 (4.0)
	19.1 (3/4)	2634 (592)	10 (0.4)	25 (1.0)	47 (1.9)	90 (3.5)
	22.2 (7/8)	2955 (664)	9 (0.3)	23 (0.9)	42 (1.7)	80 (3.1)
	25.4 (1)	3266 (734)	8 (0.3)	20 (0.8)	38 (1.5)	72 (2.8)

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Effect of Preservative Treatment on Bar Force in Stress-Laminated Bridge Decks

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Abstract

Stress-laminated timber bridge decks have gained increased popularity in the United States since the late 1980s. Like all exposed wood, the laminations in these bridges must be treated with wood preservatives to prevent deterioration from decay and insect attack. Traditionally, stress-laminated bridges are constructed using wood treated with an oil-type preservative. Waterborne preservatives are not widely used because of concerns about dimensional stability of the wood laminations. This paper describes a 2-year study that examines the performance of nine stress-laminated timber test decks constructed of Southern Pine lumber. The decks were treated with seven different preservatives to assess comparative bar force retention over time. In addition, three different bar anchorage systems were evaluated on decks treated with the same wood preservative. Preliminary results indicate that there are subtle differences in short-term bar force performance between decks treated with oil-type and waterborne preservatives. In addition, the bar force retention magnitude was more varied in the decks treated with waterborne preservatives than in the decks treated with oil-type preservatives. However, little difference was noted in the performance of the three bar anchorage systems.

Keywords: Preservative, timber, stress laminated, bridge, deck, bar force, retention, performance, bar anchorage

Introduction

It is widely recognized that a major problem is posed by the condition of the infrastructure in the United States. Of the 237,000 deficient and/or obsolete bridges, nearly 75 percent are located on secondary and rural highways (USDA 1995). Wood is an excellent material for bridge construction and rehabilitation, and often is the most economical and convenient option for rural highway bridges with spans under 12.2 m (40 ft). Research to better understand the behavior of timber bridges has been in progress at the USDA Forest Service, Forest Products Laboratory (FPL), and several universities. This research is directed at the structural behavior, construction methods, preservatives, and economic evaluation of modern timber bridges.

Background

One relatively new timber bridge system, the stress-laminated deck, was first developed in Ontario, Canada, as a rehabilitation method for nail-laminated decks. During the past 10 years, the popularity of these decks has increased significantly in the United States. A stress-laminated timber deck bridge consists of multiple longitudinal laminations that are placed on edge to form a wood plate. High strength steel bars are then inserted through holes in the wide face of the laminations. The bars are anchored to the deck edges using either a steel bearing channel or discrete bearing plates. The steel bars are tensioned with a hydraulic jack until the wood

laminations reach a compressive interlaminar stress of approximately 690 kPa (100 lb/in^2) (Ritter 1990). This interlaminar compression creates an orthotropic wood plate that improves structural performance compared with conventional nail laminated decks (Oliva and others 1990). Interlaminar compression depends on dimensional stability of the wood laminations; therefore, any dimensional change in the wood will affect bar force.

As with all wood used in exposed environments, the wood for a stress-laminated bridge must be treated with chemical preservatives to protect it from decay and insect attack. In general, there are two types of preservatives: oil-type and waterborne. Currently, oil-type preservatives, such as creosote and pentachlorophenol (penta), are the primary types of wood preservatives used in structural highway applications. For these preservatives, the preservative and/or oil carrier provides a natural resistance to moisture penetration that protects the lumber from dimensional changes caused by moisture content fluctuations. Waterborne preservatives, such as copper chromated arsenate (CCA), are also used, but there is a reluctance to use these preservatives because they provide no moisture barrier, which leads to increased dimensional changes in wood members. As a result, there is concern regarding the construction of stress-laminated timber bridges using laminations treated with waterborne preservatives, because dimensional stability of the laminations is critical to maintaining force in the deck (Ritter and others 1990).

In response to these concerns, a cooperative study with Florida State University–Florida Agriculture and Mining University (FSU/FAMU), College of Engineering; FPL; and the Federal Highway Administration (FHWA) was initiated in January 1994 to examine the comparative performance of wood preservatives in Southern Pine stress-laminated bridge decks. This paper describes this study through the first 2 years of monitoring, ending March 1996.

Objective

The objective of this study was to compare the bar force retention of stress-laminated decks treated with several wood preservatives. In addition, the performance of three bar anchorage types were examined.

Research Methods

The cooperative study plan called for constructing and monitoring nine test decks on the FSU/FAMU College of Engineering facility in Tallahassee, Florida. Monitoring including data collection of bar force, lamination moisture content, relative humidity, and temperature for approximately 2 years. A discussion of the research methods follows.

Test Decks

Each test deck measured 1.5 by 6.1 m (5 by 20 ft) and was constructed of 40 nominal 51- by 305-mm (2- by 12-in.) sawn lumber laminations (Fig. 1). The deck laminations were visually graded No. 2 or better, Southern Pine sawn lumber of random lengths. The lamination lengths were not full span; therefore, butt joints were provided at a frequency of not more than one joint in any four laminations over a 1.2-m (4-ft) distance.

Preservatives-- Seven different wood preservatives were used in this study: four waterborne and three oil-type. Each preservative complied with American Wood Preserver's Association (AWPA) Standard C14 (AWPA 1992). The four waterborne preservatives were variations of the CCA formulation as determined by American Wood Preserver's Association Standard P5 (AWPA 1990). In an effort to curb the movement of moisture in and out of the wood, three of the CCA treatments included proprietary water-repellent additives, designated as Type 1, Type 2, and Type 3. The oil-type preservatives were penta/heavy oil, creosote, and CCA in an oil emulsion. The CCA in oil is a proprietary formulation of standard CCA and is generally not considered an oil-type preservative because it only has 10- to 20-percent oil. However, it was grouped with the oil-type preservatives for this study.

Bar Anchorage Systems— The three most common bar anchorage types for stress-laminated bridge decks were also examined during the testing period. These included a continuous channel, a semi-discrete plate system, and discrete plate system (Fig. 2). All decks were configured with the discrete bearing plate system, except for two decks treated with CCA/Type 1, which were provided with one of the other systems. According to design specifications (Ritter 1990), the discrete bearing plate system creates the most critical value for compression perpendicular to grain on the outside laminations. Therefore, if all the anchorage systems are designed correctly, the discrete plate system has the highest compression underneath the plate. Following the design procedures for the discrete plate system, a design compression stress of 2,275 kPa (330 lb/in^2) is obtained. This value is less than the wet-use, compression perpendicular-to-grain allowable stress of 2,606 kPa (378 lb/in^2) for Southern Pine lumber.

Figure 1—Test deck configuration.

Table 1 summarizes the deck numerical designations, preservative type, and bar anchorage system.

Deck Construction— After fabrication, the laminations were pressure treated with preservatives. The material was then shipped to an open field adjacent to the college. Each deck was assembled by placing the laminations on two concrete barriers used as abutments. After the laminations were in place, 15.9-mm-(5/8-in.-) diameter high strength steel bars, complying with the requirements of ASTM A-722 (ASTM 1988), were inserted through predrilled holes. The bars were spaced at 610-mm (24-in.) intervals, beginning 305 mm (12 in.) from the deck ends (Fig. 1). The bars were tensioned to 117.4 kN (26,400 lb), which corresponds to 690-kPa (100-lb/in^2) interlaminar compression. The bar tensioning was completed in three stages during 4 weeks, starting January 1994. The decks were designed to the same standards used by the USDA Forest Service for stress-laminated decks (Ritter 1990). The decks after construction are shown in Figure 3.

Performance Monitoring

To evaluate performance and bar force retention of the test decks, ambient air temperature, relative humidity, bar force, and moisture content data were collected.

Figure 2—Bar anchorage configuration.

Table 1—Deck preservative and anchorage layout.

Deck number	Preservative type	Anchorage system
1	CCA/Type 1	Continuous channel
3		Semi-continuous plate
5		Discrete plate
2	Penta	
4	CCA/Oil Emulsion	
6	CCA/Type 2	
7	Creosote	
8	CCA/Type 3	
9	CCA	

Figure 3–Test site on FSU/FAMU College of Engineering Campus.

In addition, visual inspections were completed in order to assess the general condition of the decks in an exposed environment. A remote data acquisition system (data logger) was installed at the site to collect bar force and temperature data. The methods used to acquire data follow.

Temperature and Relative Humidity— Ambient air temperature was measured with a thermocouple attached to the data logger and recorded at 6-hour intervals. Relative humidity was measured daily with a hand-held humidity probe.

Moisture Content— Electrical resistance moisture content measurements were obtained at four locations (at center span and quarter span every week) on each deck: two from the deck and two from the bottom. The measurements were taken at 25-, 51-, and 76- mm (1-, 2- and 3-in.) pin penetration depths. In addition, moisture content cores were taken from the decks on three occasions (January 1994, August 1994, October 1995) in similar locations as the moisture content readings. The moisture content from the cores was determined at FPL.

Bar Force— The force in two bars of each deck was measured with load cells. The load cells were calibrated at FPL using a strain box and the data logger. Each load cell was directly connected to the data logger, and bar force readings were recorded at the same 6-hour intervals as the temperature readings. Data from the data logger were downloaded to a lap-top computer on a periodic basis.

Visual Inspections— Visual inspections were conducted periodically throughout the monitoring period to monitor the condition and performance of the test decks.

Research Results

The following preliminary study results include a presentation of temperature and humidity data, lamination moisture content, bar force retention, and a comparison of bar anchorage performance.

Temperature and Humidity

The temperature and humidity data reveal, as expected, that the weather conditions for the test decks are most severe in the summer months, between May and September when the temperature and relative humidity in the Tallahassee area are highest. The average measured temperature and humidity during this period was approximately 26.7°C (80°F) and 90 percent relative humidity.

Moisture Content

Data from moisture meter readings and core samples revealed that the moisture content of the test decks was between 15 and 20 percent at the time of construction During the monitoring period, the moisture content increased slightly to 18-22 percent at the conclusion of the monitoring period. The overall increase in moisture content is the result of the high temperature and relative humidity in the testing area. This increase in moisture content is relatively small; therefore, the effect on the bar force is minimal. However, short-term changes in humidity and temperature have a limited effect on wood moisture content and ultimately bar force in stress-laminated decks. This phenomenon is discussed in detail in the next section.

Bar Force

Figure 4 presents the average bar force data from deck 5 for the entire monitoring period, beginning January 22, 1994, and ending March 8, 1996. This plot is representative of the general bar force trend experienced by all nine test decks. The two areas of bar force data interruption noted on the graph were due to flooding, which made the data unreliable. No deck was submerged during the flooding; therefore, it is assumed that the bar force trend during the interruptions is approximately linear, as displayed.

Figure 4—Bar force in deck 5; January 22, 1994, to March 8, 1996.

After the third stressing, all test decks were at a bar force of approximately 100 kN (22,500 lb). Bar force then decreased at a steady rate until it stabilized in the spring of 1995. All decks were restressed in October 1995. The bar force in the decks has not decreased significantly since the second restressing. Complete results from all decks will be presented in an FPL research paper that will be published when testing is completed.

The results that are presented in the remaining paragraphs of this section are from May through September 1995. As mentioned, these months are representative of critical temperature and humidity experienced during the monitoring period.

Figure 5 presents the bar force for the waterborne preservatives in decks 5, 6, 8, and 9. Although the magnitude of the bar force is different, the trend is similar for each deck, with an average maximum variation over time of approximately 11.1 kN (2,500 lb). After 1-1/2 years of monitoring, it appears that CCA/Type 3 retained nearly 75 percent of the design bar force. Decks treated with CCA/Type 2, CCA, and CCA/Type 1 did not perform as well with bar force retention levels of 55, 43, and 34 percent, respectively.

A comparison of bar force for the oil-type preservatives in decks 2, 4, and 7 is presented in Figure 6. The bar forces underwent similar peaks and valleys, with an average variation of approximately 4.44 kN (1,000 lb). The test deck treated with creosote retained nearly 65 percent of the design bar force after 1-1/2 years of monitoring. The decks treated with penta and CCA/Oil Emulsion performed similarly, with bar force retention levels of 60 and 50 percent, respectively.

The average bar force for all decks treated with oil-type preservatives and waterborne preservatives is shown in Figure 7. It is apparent that the time variation of bar force in decks treated with oil-type preservatives is smaller than that for decks treated with waterborne preservatives. The variations in bar force are a result of short-term temperature and moisture content changes in the decks. The decks treated with waterborne preservative do not have a moisture barrier. Therefore, changes in relative humidity have a greater effect on the dimensional stability of the lumber laminations, which ultimately creates a greater variation in bar force.

As shown in Figures 5 and 6, the bar force magnitude range is smaller for the oil-type preservative decks than for the waterborne preservative decks. The bar force levels range from 60 to 80 kN for the oil-type preservatives and 40 to 80 kN for the waterborne preservatives.

Figure 5—Bar force variation in stress-laminated decks treated with waterborne preservatives.

Figure 6—Bar force variation in stress-laminated decks treated with oil-type preservatives.

Figure 7—Bar force comparison in decks treated with oil-type and waterborne preservatives.

These differences in bar force magnitude will be investigated in detail after testing is completed in August 1996.

Bar Anchorage Performance

Bar force data from the three decks with different bar anchorages indicate that performance is similar. Each deck had similar variations in bar force, with the maximum variation being about 12.45 kN (2,800 lb). In addition, examinations of the exterior laminations during bar restressing in October 1995 gave no indication of crushing. Similar bar force performance and satisfactory crushing resistance show that all three anchorage systems can be used adequately on CCA-treated Southern Pine stress-laminated bridges.

Concluding Remarks

The following conclusions are based on preliminary research results obtained from this study:

- There was little difference in the global moisture content change during the monitoring period for decks treated with various preservatives. The average global moisture content increased approximately 3 to 4 percent for all decks during the 2-year monitoring period.

- There was less relative fluctuation in bar force in decks treated with oil-type preservatives than in decks treated with waterborne preservatives. The relative bar force fluctuations for all decks treated with oil-type preservatives were very similar, with a maximum variation of 4.44 kN (1,000 lb) during the four summer months. Decks treated with waterborne preservatives experienced greater changes in relative bar force (11.1 kN (2,500 lb)) for the same period.

- After 1-1/2 years of field evaluation, bar force retention in decks treated with oil-type preservatives was very similar and ranged from 60 to 80 kN. For the same period, bar force retention in decks treated with waterborne preservatives was more varied and ranged from 40 to 80 kN. Although bar force retention in the waterborne preservative decks was more variable, the deck treated with CCA/Type 3 had the least bar force loss of all decks during this period. The relative bar force fluctuations in the waterborne preservative decks seemed to have had little or no long-term effect on bar force retention. Except for the deck treated with CCA/Type 1, the bar force of the test decks remained above the minimum bar force design value.

- There was little difference in bar force retention or performance of the continuous channel, semi-continuous plate, discrete plate bar anchorage systems. It can be concluded that when properly designed, the anchorage system has a negligible effect on the bar force retention of Southern Pine stress-laminated decks.

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Treatability and Durability of Heartwood

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Abstract

Reduced treatability in heartwood can cause reduced retention and penetration of preservatives into heartwood. This can reduce the durability of heartwood compared to the treated sapwood. From an anatomical point of view, the factors which contribute to the lower treatability of heartwood include its high extractive content, high rate of aspirated pits, and smaller pore sizes. Generally, in wood members that consists of treated sapwood and untreated heartwood, the durability of treated sapwood is much higher. Untreated heartwood of southern pine, Douglas-fir, Englemann spruce, eastern spruce, red pine, white pine, eastern larch, eastern hemlock, and eastern fir lasts longer in outdoor exposure in the colder northern regions (Wisconsin and Maine) than in warmer southern regions (Mississippi). In the southern regions, the median life of the stakes for these species ranges from about two to four years with the exception of southern pine which ranges from about five to seven years. Eastern spruce and white pine appear to possess lower durability (about two to three years) among all the species studied. In the northern region, the median life of the species range from five to ten years with the exception of southern pine which lasts about eight to fifteen years.

Keywords: heartwood, transition zone, sapwood, preservative treatability, natural durability.

Introduction

The sapwood of most tree species can be successfully impregnated (with some important exceptions such as the spruces), whereas the heartwood is more difficult to treat by conventional methods (USDA Forest Product Lab 1987). From an anatomical point of view, the difficulty in treating heartwood can be attributed to factors such as small pore sizes in the heartwood (Stamm 1970; Petty and Preston 1969), the generally irreversible nature of pit aspiration in the heartwood (Thomas and Nicholas 1966; Thomas and Kringstad 1971), the amount and type of extractives deposited on pit membranes during the formation of heartwood (Hillis 1987; Siau 1984; Cote 1990; Panshin and DeZeeuw 1980), and, in addition, for hardwoods, tyloses formation in the heartwood (Siau 1984; Cote 1990; Panshin and DeZeeuw 1980). For species with naturally durable heartwood such as the cedars or redwood, preservative treatment of the heartwood might not be as critical. However, for wood species with less or no naturally durable heartwood, treatment of the heartwood is necessary for construction of durable timber bridges.

Besides heartwood, in studying the treatability of wood materials, it seems that the emphasis on more understanding of the transition zone between sapwood and heartwood is relevant. In some species, especially softwoods and diffuse-porous hardwoods, a transition zone between the sapwood and heartwood is present (Hartig 1894; Craib 1923). Characteristics of the

transition zone include low permeability, pale or white color, lack of starch, and reduced moisture content (Hillis 1987). The width of the transition zone is usually about 1-3 growth rings (Hillis 1987). The implications of the transition zone on treatability of wood articles containing both heartwood and sapwood is not well understood.

In order to treat wood effectively, the treating fluid must be able to penetrate to some given depth into the wood. Generally speaking, there are two broad categories of treating processes: pressure treatment and nonpressure treatment. Of the two kinds of processes, the pressure process is used more often due to its effectiveness, controllability, and speed. In pressure processes, the penetration of the treating fluids is predominantly through bulk flow (Siau 1984). As a first approximation, the parameters which influence the bulk flow, and therefore treatability, can be appreciated through Poiseuille's equation for steady laminar viscous flow (White 1974):

$$V = \frac{\pi r^4 \Delta p}{8 \mu L} t \quad (1)$$

Where

V: Volumetric flow, cm^3
r: The radius of the cylindrical capillary through which the fluids flow, cm
μ: The viscosity of the treating fluids, poise
Δ p: The gradient of the treatment pressure, N/cm^2
L: The length of the capillary pore across which the pressure gradient, Δ p, is applied, cm
t: Processing time, s

Equation (1) was derived for a cylindrically shaped capillary under steady state laminar flow conditions. Solutions also exist for other capillaries with various cross-sectional shapes (Berker 1963). Equation (1) is a much simplified expression for the actual treatment processes. When the unsteady nature of the flow is considered, for example, a different expression was obtained which suggested less dependence of the volumetric flow on the pressure gradient and viscosity (Siau 1970). Aside from their qualitative differences, the principles in these results point to three aspects in examining the treatability of wood materials: 1) the anatomical characteristics of the wood species; 2) the characteristics of the treating fluids; and 3) the parameters of the treating processes.

Treatability

1. Softwood

A typical softwood species contains about 90 to 94% longitudinal tracheids, 5 to 10% rays, and 0.1 to 1% longitudinal resin canals. The sizes of the longitudinal tracheids in several softwood species are listed in Table 1. The amount of longitudinal parenchyma is negligible (Panshin and deZeeuw 1980). The ends of the tracheids are closed. Communication among tracheids and rays are mainly through the bordered or non-bordered pit pairs. Most of these pits reside on the radial surfaces of the tracheids or rays (Panshin and deZeeuw 1980).

Table l—Average Tracheid Diameters and Lengths of the Selected Softwood Species
(Panshin and deZeeuw 1980)

	Diameter, μm	Length, mm	L/D
Douglas-fir (*Pseudotsuga menziesii*) (4)*	55	3.0	54.5
		3.32	60.4
		3.88	70.5
Eastern hemlock (*Tsuga canadensis*) (2)	50	2.81	56.2
		2.91	58.2
		3.10	62.0
Norway pine (*Pinus resinosa*) (2)	45	2.51	55.8
		2.63	58.4
		2.67	59.3
		2.70	60.0

* Meaning of number in parentheses is the index of heartwood treatability according to MacLean (1935): 1: Easy to penetrate; 2: Moderately difficult; 3: Difficult; 4: Very difficult

Table 2—Radii of the Effective Pit-Pore Openings (mm)
(Stamm 1970; Sebastian et al 1965; Petty and Preston 1969)

Species	Heartwood	Sapwood
E. larch (*Larix laricina*) (4)* [c]**	0.008	0.500
Incense cedar (*Libocedrus decurrens*) [c]	0.012	0.085
E. red cedar (*Juniperus virginiana*) [c]	0.013	0.12
N. white cedar (*Thuja occidentalis*) [c]	0.017	0.170
Douglas-fir (*Pseudotsuga menziesii*) (4) [c]	0.025	0.170
Redwood (*Sequoia sempervirens*) [c]	0.047	0.100
Sitka spruce (*Picea sitchensis*) [d]	0.20 - 0.26	0.74 - 0.98
White spruce (*Picea glauca*) (3) [TEM]	0.58	0.75

* Meaning of number in parentheses is the same as in Table 1
** c: From Stamm (1970) measured using capillary method; d: From Sebastian (1965) using permeability measurement; TEM: From Petty and Preston (1969) using Transmission Electron Microscope.

Because the average diameter of the pit pores is much smaller than that of the tracheid lumens, the treatability of wood is largely dependent on the size and condition of the pit structure. Therefore, parameters such as effective pore size, number of pit openings per unit area, probability of aspiration and deaspiration, encrustation of the pit membranes, and length of the tracheids are directly related to the treatability of softwood species. Besides the pit openings, tiny pores in the cell wall also play a role in the treatability, especially when polar solvents are used for the preservatives (Nicholas and Siau 1973). However, since these pores are much smaller than the pit pore openings, their role in treatability maybe secondary when the pits are effective in transporting fluids.

The size of pit openings has been measured with various techniques: electron microscopy (Cote and Krahmer 1962, Thomas and Nicholas 1966), electro-osmotic flow techniques (Stamm et al 1968; Stamm 1970; Siau 1981), filtration of aqueous suspension of particles of known sizes (Liese 1965; Megraw 1967), and the permeability measurement using the Adzumi equation (Siau et al 1981; Bao, Siau, and Avramidis 1986; Comstock 1967; Sebastian et al 1965). As indicated in Table 2, there is a substantial difference between the pit-pore radii of sapwood and heartwood. The volumetric flow rate is proportional to the fourth power of the pore size. Therefore, one would expect much reduced treatability of the heartwood of these species given the other conditions being constant. For example, as shown in Fig. 1, experimental results from about 100 southern pine specimens indicated that the total retention in preservative treated wood decreases exponentially as the percentage of heartwood increases (Huffman 1996). This is attributable to the fact that heartwood possesses much lower permeability,

Even though the permeability of an unaspirated pit membrane in a softwood is relatively high due to the

Fig. 1—Relation between amount of heartwood and retention in southern pine (Huffman 1996).

presence of the open margo structure, the function of the open margo can cease upon aspiration of pit membranes (Cote 1990). During drying prior to preservative treatment, the pit membranes aspirate as water is removed from the cell and the air-water meniscus passes through the membrane. The aspirated pit membrane results in a tight seal between the membrane and the pit opening due to hydrogen bonding, capillary forces (Thomas and Kringstad 1971), and, in heartwood, the adhesion of extractives.

The degree of pit aspiration and encrustation with extractives is closely related to the location of the wood in the tree. After investigating about 200 *Pinus radiata* trees of 27 years old grown in Australia, Harris (1954) concluded that the mean percentage of the aspirated pits in the sapwood, transition zone, and the heartwood are 30%, over 90%, and 96%, respectively. The increase in pit aspiration through these regions is relatively gradual. However, Nobuchi and Harada (1983) found that the percentage of aspirated pits in *Cryptomeria japonica* increased sharply at the border of the sapwood and transition zone from 10% to 60% and with a further slight increase in the heartwood. Similar changes in encrustation percentage have been found in

Table 3—Volume Percentages of the Cells in the Selected Hardwoods
(Panshin and deZeeuw 1980)

Species	Vessels	Fibers	Rays	Parenchyma
Hard maple (*Acer saccharum*)	21.4	66.6	11.9	0.1
Red maple (*Acer rubra*)	18.0	68.0	13.3	0.1
Tulip poplar (*Liriodendron tulipifera*)	36.6	49.0	14.2	0.2
Cottonwood (*Populus deltoides*)	33.0	53.1	13.7	0.2
Red oak (*Quercus rubra*)	21.6	43.5	21.4	13.5
Sweetgum (*Liquidamber styraciflua*)	54.9	26.6	18.3	0.2

Pinus strobus, P. densiflora, and *P. banksiana* (Yamamoto 1982). Yamamoto found that the degree of encrustation of the bordered pit membranes in tracheids was essentially zero in the sapwood, gradually increasing through the transition zone to almost 100% in the heartwood. Not only does the heartwood have a larger percentage of aspirated pits, the reversibility of the pit aspiration is also reduced. In sapwood, pit aspiration is partially reversible under certain conditions such as treating pressure or resoaking in water (Thomas and Nicholas 1966). However, the reversibility is reduced in heartwood due to the adhesion of extractives between the aspirated pit membrane and the pit opening (Thomas and Nicholas 1966). Therefore, it is seen that the reduction in treatability due to pit aspiration and pit encrustation has greater implications in heartwood than in sapwood (Krahmer and Cote 1963; Liese and Bauch 1967).

Because most of the pits are concentrated on the radial surfaces, rays may be the major conduits for radial flow (Erickson 1970; Cote 1963). The function of ray tracheids is especially important for the impregnation of preservatives into poles or flat sawn lumber (Behr et al 1969; Cote 1989). Ray tracheids are constant features of the woods of *Pinus, Picea, Larix, Pseudotsuga, and Tsuga* (Panshin and deZeeuw 1980). Due to the function of these rays, for some wood species in these genera such as *Pinus sylvestris,* the permeability along the radial direction is higher than that along the tangential direction (Banks 1970). It appears that the effectiveness of the ray tracheids depends on their volume percentage in the wood and the condition of their pits. In studying the treatability of spruce wood, it has been suggested that the low ray tracheid content is a major factor contributing the refractory nature of these species (Baines and Saur 1985). Liese and Bauch (1976) concluded that the primary reason for the poor treatability of *Picea abies* is due to the low concentration of the ray tracheids. The refractory nature of *Pseudotsuga menziesii* was also attributed to the pit encrustation and the low volume fraction of the ray tracheids. Ray tracheids are generally more effective for conducting radial flow than the ray parenchyma cells (Buro and Buro 1959; Cote 1963; Erickson and Balatinecz 1964). This may explain why the ray tracheids surrounding the ray parenchyma cells are often impregnated by preservative, while the parenchyma cells are not impregnated (Liese and Bauch 1967).

2. Hardwoods

Most hardwood species contain four major cell types: vessels, fibers, rays, and axial parenchymas. Their volume percentages for selected hardwood species are shown in Table 3.

During preservative treatment, the treating fluids flow in an interconnected network of these cells as illustrated in Figure 3 (Siau 1984). Therefore, all the components including the conducting channels and their connections contribute to the treatability of hardwoods. However, experimental results in the literature generally show that vessels are by far the major avenue in conducting treating fluids. Therefore, the size, distribution, and condition of the vessels are important factors affecting the treatability of hardwoods. Vessel diameter of the diffuse-porous species is generally between 20 and 100 μm. Average vessel diameter of the ring-porous species varies conspicuously between the earlywood and latewood. Average vessel diameter in the earlywood ranges between 50 and 400 μm, while that in latewood ranges between 20 and 50 μm. The typical concentration of vessels when counted in cross section is on the order of 15,000/cm^2 (Siau 1984).

Vessels are composed of vessel elements connected end to end through perforation plates. The resulting vessels can be long and continuous. For example. vessels up to three meters in length have been reported (Thomas 1981) Tyloses and various gummy, resinous, and chalky exudates often form in the vessel lumens within the heartwood and transition zone (Hillis 1987) The formation of these materials will substantially reduce the treatability of the heartwood and the transition zone (Kumar and Dobriyal 1993; Perng, Brebner, and Schneider 1985) In their pioneer work, Teesdale and MacLean (1918) found that the treatability of hardwood was directly related to whether the vessels

contained tyloses and, if tyloses were present, the completeness of the vessel blockage by the tyloses. In a more recent work by Thomas (1976), the role of tyloses in reducing the treatability of hardwood is further confirmed. A practical example of the role of tyloses in blocking fluid flow is the use of white oak to make barrels for wine and whisky (Cote 1990) The vessels in the heartwood of most white oaks me completely blocked by tyloses. Besides tyloses, gummy and chalky extractives can also reduce the treatability of hardwoods (Siau 1984) It worth noting that since tyloses and extractives are more likely to be found in the heartwood (Panshin and deZeeuw 1980), one would expect that these factors are responsible for the reduction of the heartwood treatability.

Rays and fibers occupy about 10 to 20%, and 25% to 75%, respectively, of the total volume of hardwoods. When the vessels are occluded with tyloses and extractives, the literature seems to indicate that the rays and fibers could function as fluid conducting channels. One may also be able to deduce this conclusion from Figure 3. It has been experimentally observed that there is a relatively high concentration of preservatives in the ray tissues when various hardwoods were treated with CCA and creosote (Greaves and Levy 1978; Bosshard 1961). It has also been reported that ray parenchyma cells are important channels for radial flow in some species (Behr et al 1969). However, Teesdale and MacLean (1918) concluded that hardwood rays are not important in transverse distribution of creosote preservatives. Fibers in hardwoods are elongated cells with thick walls and small lumen diameters. The fiber lengths range from 600 to 2,300 μ m with a length to diameter ratio on the order of 100. It has found that the conducting channels in the heartwood of hickory were essentially the fibers (Thomas 1976; Teesdale and MacLean 1918) due to a multitude of factors such as the blockage of pits and vessels, isolation of vessels, and the low volume of vessels. Behr et al (1969) observed that treating fluids are transported from fiber to fiber via pits. In the oaks, the vasicentric tracheids can also function as communication means between the intervessel fluid transport (Wheeler and Thomas 1981).

Fluid transport between cells is through the bordered or half bordered pit pairs (Siau 1984; Perng, Brebner, and Schneider 1985). The pit membranes in hardwoods lack the open margo structure of softwood species. Besides this difference, the hardwood pits are generally smaller with a chamber diameter on the order of 6 μ m (Siau 1984). Studies show that the pit membrane in hardwoods is actually permeable despite the lack of a margo (Siau 1984; Thomas 1876; Cote 1963). Apparently the hardwood pit membrane acts like a filter in that the fluid flows circuitously in the membrane. This type of membrane exists in the intervessel bordered pit pairs, between vessel and fiber tracheids, and also between longitudinal and ray parenchyma.

Similar to softwoods, the pit membranes of hardwoods can also exhibit aspiration and encrustation in the heartwood (Panshin and deZeeuw 1980; Hillis 1987). These factors also contribute to the reduced treatability of heartwood in hardwoods (Cote 1963; Krahmer and Cote 1963).

3. Flow Characteristics

Under given treating conditions, the penetration depth of a preservative also depends on its flow characteristics in the interconnected wood cells. Depending on the Reynolds number, the flow of the preservative undergoes in the wood cell network could manifest in three forms: 1) laminar flow, 2) nonlinear flow, or 3) turbulent flow. For flow in a circular tube, the Reynolds number can be expressed as (White 1974, Siau 1984):

$$Re = 2ru\rho/\eta \qquad (2)$$

Where

- Re: Reynolds number
- r: A characteristic dimension of the flow pathway, which depends on the shape of the cross section of the flow pathway. For circular cross-section, r is the radius, cm
- u: The average linear fluid velocity, cm/s
- ρ: The density of the fluid, g/cm^3
- η: The viscosity of the fluid, poise

In laminar flow, the preservatives flow along well defined streamlines. Laminar flow generally occurs when a fluid flows in a long tube at a moderate velocity. In laminar flow, the flow rate is proportional to the pressure gradient, Δp. However, if the length-to-diameter ratio of the capillary is small, flow could become nonlinear. In this case, there will be kinetic energy loss due to the end effects when fluid enters or exits the short tube. Due to the energy loss, flow rate is proportional to $\Delta p^{0.57}$ (Siau 1984). Turbulent flow is due to the breakdown of the laminar flow at high velocity. In turbulent flow, there will be heat energy dissipation due to friction. Due to this energy loss, the flow rate is proportional to $\Delta p^{0.5}$ (Siau 1984). Therefore, in both the nonlinear and turbulent flows, the flow rate would be reduced at a given pressure gradient due to energy losses. The criterion at which each flow occurs can be expressed as (Siau and Petty 1979; White 1974):

Laminar flow	$Re < 0.8\ L/r$	(3)
Nonlinear flow	$Re > 0.8\ L/r$	(4)
Turbulent flow	$Re > 2000$	(5)

Where

L: The length of the capillary, cm
r: The radius of the capillary, cm

The above discussions are based on the assumption that the inside wall of the capillary is smooth. However, it is known that the inside walls of the longitudinal or transverse cells are generally not smooth. For example, the inside walls of the tracheids in Douglas-fir contain spiral thickenings (Panshin and deZeeuw 1980). These thickenings impart spiral "ridge and valley" features on the inside wall of the tracheids. Due to this roughness, the flow characteristics inside the tracheids will be affected. For example, the turbulent flow may develop at a lower velocity than in the otherwise smooth walls. The study of the roughness effect on the flow behavior was not found in the literature.

Durability

In two FPL studies, untreated solid wood stakes, nominally 2" X 4" X 18" (5 X 10 X 45 cm) in dimensions, were vertically set into ground to a depth of half of their length in three locations: Mississippi, Wisconsin, and Maine. The species included southern pine, Douglas-fir, and Englemann spruce installed in December, 1975 and May, 1976 (Gutzer and Crawford 1995); and white pine, red pine, eastern spruce, eastern hemlock, eastern larch and balsam fir (Gjovik and Schumann, 1992) installed in Mississippi and Maine in 1985. The southern pine 2" x 4" (5 X 10 cm) source material contained some sapwood, but an attempt was made to eliminate all sapwood when cutting stakes from that material. The other species were almost entirely heartwood.

In a study at the University of Florida, stakes, nominally 2" X 4" X 24" (5 X 10 X 610 cm), were installed near Gainesville, Florida. Stakes contained both heartwood and sapwood. Before installation, the Florida stakes were treated with 1% of copper naththenate in mineral spirit in a steaming and soaking procedure (Huffman 1996).

From Fig. 2 and Fig. 3, it is seen that stakes of all wood species survived longer in the northern regions than in the southern Mississippi. In southern Mississippi, virtually all heartwood stakes failed within a decade of exposure. Only one southern pine heartwood stake survived for more than ten years. It is known that the ideal fungi growth condition is at a temperature of around 20 to 30 °C and a relative humidity of 80 to 100%. Therefore, apparently, the Mississippi weather is more suitable for the fungi to grow, thus accelerating the degradation process. From Fig. 3, southern pine species possesses better natural durability than other species, while that the heartwood

Fig. 2—Natural durability of Douglas-fir, southern pine, and Englemann spruce in Mississippi and Wisconsin.

Fig. 3—Natural durability heartwood of the indicated softwood species in Mississippi and Maine.

of white pine and eastern spruce is much less durable than that of other species.

Fig. 4 shows the results from the Florida stakes which installed in ground for about 32 years (Huffman 1996). As seen from these treated stakes, the heartwood is less durable than the sapwood. For example, after 32 years of exposure, almost all the heartwood was degraded. However, at higher retention levels, only a fraction of the sapwood was degraded. For sapwood, the percentage of decay decreased as the retention level

Fig. 4—Percentage of decay in heartwood and sapwood of southern pine after 32 years of field test (Huffman 1996)

increased. As the retention reached 7 pcf treating solution, the decay in sapwood is nearly zero while the decay in heartwood is about 100%. The retention in Fig. 4 reflects the uptake of the preservative solution, which was 1% of copper napthenate in mineral spirits (Moody 1952).

Conclusions

Heartwood treatability is much lower than the sapwood treatability. The factors attribute to the lowered heartwood treatability include the higher extractive content in heartwood, the increased rate of irreversible aspirated pits in heartwood, and, in addition, in hardwood, the tyloses formation in the heartwood. These anatomical factors result in the reduced apparent pit-pore sizes in heartwood and the associated low permeability in heartwood. As a result of these physiological reasons, the treatability of heartwood is much lower than that of the sapwood.

Besides the heartwood region, it seems that the transition zone in some species is also less permeable than the sapwood and less durable than heartwood. This perception makes the transition zone a more sensitive zone in terms of preservative treatment and decay behavior. However, there is no known research has been found which addresses this important anatomical region in terms of its durability and treatability. Therefore, further research in this area is warranted.

Untreated heartwood, with moderate or less natural decay resistance and exposed in ground contact anywhere within the temperate climatic zone, cannot be expected to last more than two decades. In environments of severe biological challenge, such as that of southern Mississippi, untreated heartwoods, exposed to ground contact, cannot be expected to last for a decade.

Climate is an important factor that affects the rate of decomposition of wood exposed in contact with the ground. It is of more importance in affecting the rate of decay in unprotected heartwood than is the degree of natural resistance for heartwoods with moderate or less resistance to decay.

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Wood Preservation--Preservative Treatment For Hardwood Glued-Laminated Bridges

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Abstract

Pressure-treating cycle guidelines were developed to obtain satisfactory penetration and retention of creosote preservatives in red oak, red maple and yellow poplar glued-laminated timbers. The creosote-treating cycle was used to treat a red oak glued-laminated timber bridge. A retention of 192.2 kg/m^3 (12.0 pcf) of creosote was obtained with penetration of more than 12.7 mm (0.5 inch). The treating cycle guidelines were slightly modified and used to treat a red maple glued-laminated timber bridge. Penetration ranged from 50.8 mm (2.0 inches) to completely through the cross section of the laminated beam with retention exceeding 240.3 kg/m^3 (15.0 pcf). Treatment of red maple bridge material also used a revised "post conditioning" cycle with a low-temperature/vacuum, post-creosote treatment cycle. The modified cycle resulted in an excellent creosote treatment of the red maple glued-laminated timber bridge and the post-treatment was successful in cleaning the surfaces and in controlling bleeding of the preservative.

Keywords: Bridges, Timber, Hardwood, Glued-Laminated, Preservative Retention, Preservative Penetration, Treating Cycle, Dimensional Stability

Background

Exterior use of hardwood glued-laminated timbers will require preservative treatment of the timbers. In general, hardwood species with low specific gravity values are usually treated at pressures and for durations similar to those treating cycles used for softwoods. Koch (1985) reports that hardwood species differ widely in their preservative treating cycle requirements. One caution in treating hardwoods, at least until the data are available, is that the effects of the treating process and/or the type of preservative may cause a reduction in the mechanical properties of the wood. Oil-type preservatives usually result in no appreciable reductions in mechanical properties after treatment since these preservatives apparently do not react with wood chemical constituents. Manbeck, et al. (1995) reports that the flexure properties are not reduced after creosote treatment of red oak, red maple and yellow poplar glued-laminated beams. However, reductions in mechanical properties may occur with oil-type preservatives if American Wood Preservers' Association (AWPA) recommended temperature-time combinations are exceeded during the preservative treatment cycle (Winandy, 1988; Barnes and Winandy, 1986).

The amount of preservative required to protect hardwoods depends on the wood species, service conditions, and type of preservative. Thompson and Koch (1981) have recommended 128.1 kg/m^3 (8.0 lbs/per cubic foot or pcf) retention of whole creosote in assay zones for protection of hardwoods placed in contact with the soil. While this level of retention is acceptable for railroad ties or other non-critical uses, AWPA recommends higher retention levels for bridge timbers and other structural members where failure or replacement is very impractical.

Data on the weatherability of preservative treated wood have been reported. Rietz (1961) reported that oil-type preservatives improved the weathering properties of red oak. The preservatives tested in his study included creosote and pentachlorophenol in various solutions. Baileys, et. al. (1994) reported on the gluebond and structural strength characteristics of hardwood glued-laminated beams treated with creosote. The creosote treated hardwood glued-laminated beams passed the American Institute of Timber Construction (1983, 1987) gluebond performance standards (Baileys, et. al., 1994).

Selbo (1967 and 1975) reported on the effects of eight preservatives, including creosote, on glueline strength for red oak, hard maple, southern pine, and Douglas fir glued-laminated specimens exposed for 20 years in an enclosed, unheated shed. The shear strengths of the resorcinol and phenol-resorcinol resins exposed to these eight preservatives used in the study, were similar to the strength of the wood. In another phase of this study, creosote provided the best protection for preventing checking and large amounts of shrinking and swelling for red oak after 20 years of exterior exposure. Water-borne preservative treatments provided little, if any improvement in protection against checking over the untreated wood. Douglas-fir, southern pine and red oak beams did not develop any appreciable delamination of glue joints after 20 years of exterior exposure. However, the hard maple untreated and water-borne preservative treated beams had appreciable glueline delamination. Study results identified creosote treatments, for glue-laminated beams subjected to exterior exposure, as providing enhanced weatherability along with desired protection against wood destroying organisms.

Data on the efficacy and service life of various hardwoods treated with several preservatives and exposed to the environment have been reviewed and published by Thompson and Koch (1981) and Koch (1985). All preservatives improved the service life over the untreated controls. However, creosote-treated hardwoods had the longest service life compared to the next best preservative treatment (usually pentachlorophenol). The waterborne metallic preservatives usually had the shortest service life compared to the other preservatives tested with hardwoods (Thompson and Koch, 1981).

Hardwoods are susceptible to degradation by soft rot fungi and, therefore, the preservative must be impregnated into the cell wall (Vick and Baechler, 1986; Gjovik and Gutzmer, 1989). The susceptibility to being degraded by soft rot fungi is directly related to the efficacy of wood preservatives. Certain salt solutions (e.g., copper sulfate, zinc sulfate, sodium chromate and arsenic acid) investigated as a wood preservative did not have a high efficacy in hardwoods. Zinc compounds are not effective in preventing soft rot in hardwoods. The lack of efficacy maybe associated with certain hardwood species that are difficult to treat. However, the exact reason zinc and copper compounds used as wood preservatives are ineffective in preventing soft rot in hardwoods is unknown (Vick and Baechler, 1986).

Based on the reports and a long history of success in treating railroad cross ties, creosote appeared to bean excellent choice as the preservative treatment for hardwoods. However, only a limited data base has been available on treatability and serviceability of hardwood glued-laminated timbers.

A number of researchers have summarized the effects of migration and mitigation of wood preservatives from treated wood products in the environment (Lamar and Kirk, 1994; Davis, Glasser, Evans and Lamar, 1993; Lamar, Evans and Glaser, 1993; Webb and Gjovik, 1988). Webb and Gjovik (1988) concluded that preservatives did not significantly migrate and/or accumulate in the environment after migration from the treated wood. Lamar and Kirk (1994) summarize the results of many research projects and conclude the microbiological treatments may be used as remediation of pentachlorophenol and creosote contaminated soils. Treated wood products and wood preservatives can be safely used with proper precautions (Lamar and Kirk, 1994; Davis, Glasser, Evans and Lamar, 1993 Lamar, Evans and Glaser, 1993; Webb and Gjovik 1988).

The purposes of this research were: 1) to determine the creosote treatability (penetration and retention) of three hardwood species glued-laminated timbers; 2) to develop guidelines for a pressure treating cycle to obtain satisfactory penetration and retention of three hardwood species glued-laminated beams; and 3) to demonstrate the applicability of the treating guidelines by treating red oak and red maple glued-laminated

timber bridges. The data for this research were obtained and analyzed for red oak *(Quercus rubra),* red maple *(Acer rubrum),* and yellow poplar *(Liriodendron tulipiferu)* glued-laminated beams.

Experimental

Laboratory preservative treatment cycle studies used red oak, red maple, and yellow poplar glued-laminated specimens treated at Koppers Industries, Inc. These studies were used as the basis for forming the guidelines for the treating cycles used to treat the hardwood glued-laminated test specimens. After the preliminary laboratory studies, additional red oak, red maple, and yellow poplar were manufactured into glued-laminated beams and subsequently treated with creosote to determine if glued-laminated hardwoods maybe treated to the levels recommended in AITC 109-84. Each glued-laminated hardwood species was treated at Koppers Industries, Inc., using creosote at a selected treatment cycle with a minimum of 6 replications on 6-ply laminated beams produced from 25.4 mm (1.0 inch) ply beams 152.4 x 152.4 x 1828.8 mm (6.0 inches x 6.0 inches x 6.0 feet). Average gross retention and assay retention in kilograms per cubic meter or kg/m^3 (pounds per cubic feet or pcf) of each beam was determined. The depth of penetration was measured on each lamination. The individual tests sampled the three wood species, six beam treatment replications, and each lamination.

The preliminary treatment tests were used to develop recommended empty cell treatment cycle guidelines. These guidelines formed the basis for the treatment of the red oak and red maple glued-laminated timber bridges.

Results and Discussion

The treatability test samples were 6-ply laminated beams produced from 25.4 x 152.4x 1828.8 mm (1.0 inch x 6.0 inches x 6.0 feet) red oak, red maple and yellow poplar lumber. Three beams of each species, along with three additional incised yellow poplar beams, were treated with creosote solution meeting American Wood-Preservers' Association (AWPA) P2 Standard. These were treated in a charge of oak crossties at the Koppers Industries, Inc. Susquehanna Treating Plant located near Wllliamsport, PA.

Retention was determined by gauge, weight difference and assay for all 12 test pieces. Penetration and assay assessments were made by cutting each beam at the midpoint and removing a cross-sectional wafer 12.7 mm (0.5 inch) in thickness. Retention data for these samples are presented in Table 1 as average weight difference and assay under the standard treating cycle. Assay extraction followed the AWPA A6 procedures on an assay zone of the outer 15.2 mm (0.6 inch) of wood.

Baileys, et. al., (1994) reported that retention of preservative was acceptable in these initial samples but due to variable penetration in the beams, additional glued-laminated specimens were prepared. The extra samples would be treated by varying the treating cycle parameters of temperature, time and pressures to determine a cycle that would achieve improved penetration on the edges of the laminates. As a result, the following empty cell cycle guidelines were recommended to treat red oak, red maple and yellow poplar glued-laminated materials with creosote or creosote solution (Baileys, et. al., 1994):

- Initial air pressure should be between 137.9-206.9 kPa (20-30 psi);
- Creosote should be introduced into chamber and pressurized to 1034 .3-1379.0 kPa (150-200 psi). Actual pressure is species dependent;
- Treatment temperature should be between 87.8-98.9°C (19O-21OºF);
- Treatment temperature and pressure should continue until 192.2 kg/m^3 (12.0 pcf) retention (by gauge) is achieved;
- Following pressure cycle, the pressure should be released in a slow step-down manner over a one-hour period;
- During the "slow press release," an expansion bath should be used with an increase in temperature in the treating cylinder of 5.6°C (10°F);
- Remove the creosote and apply a minimum vacuum of 74.3 kPa (22.0 inches) of mercury for two hours;
- Release vacuum and, if possible, steam-clean the surface of the glued-laminated members for one hour; and
- A final minimum vacuum of 74.3 kPa (22.0 inches) of mercury is applied to treated members for two hours. Actual vacuum varies with altitude.

As a means to verify this proposed cycle, six replications of 6-ply glued-laminated beams for each hardwood species were again treated in a commercial operation. The previous performance of yellow poplar was not improved by incising; therefore, this test did not include incised yellow poplar test samples, The results using this adjusted treating cycle are also included in Table 1 (Baileys, et. al., 1994). Penetration was improved in both red maple and yellow poplar using the adjusted cycle. Red oak had exceptionally good penetration using both the preliminary and adjusted cycles.

Gluebond performance data including cyclic delamination, shear strength, percent wood failure, and end-joint performance are presented in Labosky, et al. (1993), Janowiak, et al. (1993), and Baileys, et al. (1994). The effects of creosote treatment on the performance of red oak, yellow poplar and red maple are presented by Manbeck, et al. (1995).

Table 1—Creosote Treatment of Glued-laminated Test Beams (Baileys, et. al., 1994)

SPECIES	CREOSOTE RETENTION									
	PRELIMINARY TREATMENT CYCLE						ADJUSTED CYCLE			
	WEIGHT RETENTION		ASSAY Combined[a]		ASSAY Edge[b]		WEIGHT RETENTION		ASSAY Edge[b]	
	pcf	kg/m^3	pcf	kg/m^3	pcf	kg/m^3	pcf	kg/m^3	pcf	kg/m^3
Red Oak	5.9	94.5	6.95	111.3	5.92	94.8	11.2	179.4	9.77	156.5
Red Maple	12.5	200.2	13.79	220.9	13.64	218.5	18.3	293.1	22.95	367.6
Yellow Poplar	10.6	169.8	10.76	172.4	12.94	207.3	16.1	257.9	16.22	259.8
I-Yellow Poplar[c]	10.0	160.2	10.74	172.0	10.55	169.0	NOT INCLUDED			

[a]Combined sample assay values are weighted average of edge and face laminates.
[b]Assay zone 0-15.2m (0-0.6") from edge of laminates only.
[c]I-Yellow Poplar = Incised Yellow Poplar.

Red Oak Glued-Laminated Bridge Preservative Treatment Summary

The northern red oak lamination materials used to construct the hardwood glulam demonstration bridge in Ferguson Township, Centre County, Pennsylvania, were treated with creosote solution at the Koppers Industries Susquehanna Treating Plant in Muncy, PA, using pressure process methods and P-2 creosote solution meeting the American Wood Preservers' Association Standards. The treating took place in two separate charges due to lamination assembly difficulties involving the bridge rail and post components. The beams, back walls, deck panels and diaphragms were treated on September 18, with the post and rail material arriving on October 4, 1991.

The lumber used to fabricate the bridge components had been kiln-dried below 16% average moisture content prior to gluing and the assembled pieces were wrapped and covered with water repellent paper to maintain this dry condition. One beam had a distinct separation along a glue line before treatment on one end and a few other minor separations were observed, but none were extensive enough to cause concern. The beam with the separated glue line was repaired at the construction site.

Lamination and fabrication were performed prior to preservative treatment and the preassembled components were loaded onto tram cars with 19.1 mm (0.75 inch) cable used as stickers to separate the layers. This allowed for better heat transfer and flow of the treating solution around and into the wood during pressure and vacuum cycles inside the retort cylinder.

There were 33.9 m^3 (1197.0 cubic feet) of bridge material in the first charge loaded on September 18 with the balance of this cylinder charge material consisting of air-dry oak industrial grade crossties. The treating cycle used included the following parameters:

- Initial air pressure . . . . . 275.8 kPa (40 psi)
- Fi1l cylinder against initial air with preservative . . . 82.2°C (180°F)
- Ž Pressure period . . . . . 1275.6 kPa (185.0 psi) 4 hrs.
- Cylinder temperature raised from 82.2°C to 100°C (180°F to 212°F) during pressure
- Pressure release and maintain . 100°C (212°F) 2 hrs.
- Preservative pumped back . . . . . 50 minutes
- Final vacuum . . . . . 91.2 kPa (27.0 in. Hg) 3hrs.

Two core borings, 76.2 mm (3.0 inches) in length, were removed from each piece of bridge material to visually determine preservative penetration and provide samples for assay extraction to determine the level of retention.

Eighteen borings were taken from the beams with nine from face laminates and the other nine from inside laminates. All but two face laminate cores had 100% penetration of the annual rings in the 76.2 mm (3.0 inch) boring. Inner laminate cores obtained 51% (average) penetration of the annual rings in the 76.2 mm (3.0 inch) samples. Combining the penetration measurements of inner and face cores resulted in the beam samples having 83% of the annual ring count penetrated with preservative solution.

The deck panels were also bored and inspected for depth of penetration. Both face cores and inner laminate cores were again evaluated with face laminates exhibiting a more complete penetration of the wood by the creosote. The combined total ring count was 72% penetrated in the deck panel samples. Both the beam and deck panel borings meet the requirements for average penetration of 65% of annual rings in red oak as stated in the AWPA C2 Hardwood Lumber and C6 Crosstie Standards.

At the present time, hardwood retention of preservative using gross injection or gauge retention is determined by the total weight of preservative used to treat the volume of material. By this method, the retention of preservative in the deck and beam material was 205.0 kg/m^3 (12.8 pcf). However, an assay by solvent extraction was performed following the AWPA A6 Standard for determination of oil-type preservatives and water in wood. The deck and beam samples were assayed as two separate samples using the 0-25.4 mm (0-1.0 inch) zone from each boring. Results showed 262.7 kg/m^3 (16.4 pcf) in the beams and 241.9 kg/m^3 (15.1 pcf) in the deck panels. These levels easily meet the 192.2 kg/m^3 (12.0 pcf) soil contact retention requirements for softwood glulam structural members for highway construction specified in AWPA C14 Wood for Highway Construction Standard.

A second set of core borings were obtained from the post and bridge rail components treated in the later charge of material. Measured penetration was in a range of 19.1 to 63.5 mm (0.75 to 2.5 inches). The average was 38.1 mm (1.5 inches) for the 10 boring cores inspected. An assay was performed using the O-25.4 mm (0-1.0 inch) zone of wood resulting in a 145.8 kg/m^3 (9.1 pcf) retention, also meeting the specified creosote retention of 128.1 kg/m^3 (8.0 pcf) for sawn bridge (hand or guide) rails not in contact with ground or water as stated in C14.

The test bridge material of laminated red oak readily accepted preservative treatment with creosote solution in terms of both penetration and retention evaluations. Face lamination penetration was extremely good in all samples tested with acceptable but somewhat more variable penetration observed in the cores removed from inner laminates. Retention by gauge reading determination exceeded AWPA Standards for hardwood lumber in soil contact and using a 0-25.4 mm (0-1.0 inch) assay zone also gave results higher than specified for similar structural materials from softwood species.

The design of the red oak glued-laminated bridge included a 12.7 mm (0.5 inch) gap between deck panels. The gap would allow the deck panels to expand as they increased in moisture content from a average of 12% to approximately 20% during in-service. During treatment, the deck panels adsorbed moisture and expanded to between 3.2 and 19.1 mm (0.125 and 0.75 inches). The average expansion was 12.7 mm (0.5 inches).

Incidental preservative bleeding due to the kickback of the treating solution from the hardwood glulam was a cause of some concern at the construction site. This is common when treating kiln-dried materials which tend to trap initial air inside the wood cells and also retain a very heavy amount of preservative near the surface. Modification to the treatment cycle using additional cleanup cycle time with extended heat and vacuum to remove the initial air and clean excess creosote from the surface and/or use of a clean distillate creosote preservative may eliminate much of this problem.

Red Maple Glued-Laminated Bridge Preservative Treatment Summary

Red maple is considered an under-utilized source of hardwood lumber and timbers in many of the northeast states, especially Pennsylvania. The resource is plentiful, and one alternative to enable a utilization of this species is to manufacture glued-laminated materials. Although creosote preservatives are recognized as the primary treatment choice for hardwood timbers to be used in structural components, consideration must be given to alternative preservatives where there is human contact potential of the bridge (guide or hand) rail systems. Waterborne arsenical treatment chemicals are acceptable preservatives for softwood species when a clean, dry, nonoily surface is critical (such as backyard decks, porches, furniture and playground equipment). However, the experience in using these chemicals in hardwoods is limited because there is no water repellency provided to the treated hardwoods to prevent the development of moisture related defects. A recent development by Hickson Corp. and Koppers Industries, Inc. of an oil emulsion/waterborne system which injects an oil and wax combination into the outer 25.4 mm (1.0 inch) zone created the opportunity to test this new preservative system on the guide railings of this bridge.

The red maple bridge components were laminated from kiln-dried material (average 12% moisture content) prior to treatment. Creosote preservative treatment of the beams, deck panels, guide posts and diaphragms used as structural components 27.7 m^3 (977 cu ft) was conducted on June 13, 1995, at the Koppers Industries Susquehanna Plant located in Muncy, Pennsylvania. The glued-laminated bridge rail sections were shipped to the Koppers Industries treating plant in Montgomery, Alabama, to receive the two-step CCA/oil emulsion treatment available at that location.

Creosote Treatment — The treating cycle used with the red oak glued-laminated timber bridge was modified for the red maple girders and deck panels and included the following parameters:

- Initial air pressure . . . . . 172.4 kPa (25.0 psi) for 0.16 hrs.
- Fill the cylinder . . . . . 88.9°C (192°F) for 1.08 hrs.
- Pressure . . . . . target retention 192.2 kg/m^3 (12.0 pcf gauge) 1275.6 kPa @ 95°C (185.0 psi @ 203°F) for 4.00 hrs.
- Blowback preservative . . . . . 0.75 hrs.
- Vacuum . . . . . 87.8kPa (26.0 inches Hg) for 5.00 hrs.
- Pump drips & break to atmospheric pressure 0.16 hrs.
- Final Vacuum . . . . . 87.8kPa (26.0 inches Hg) for 2.00 hrs.
- Total time of treating . . . . . 13.15 hrs.

The balance of the charge treated with the bridge components were crossties, and the gauge retention injected during this treating cycle was 205.0 kg/m^3 (12.8 pcf) which includes the amount of treatment in the crossties.

Assay retention analysis is required on all commodities except crossties in the American Wood-Preservers' Association Standards; therefore, the bridge material was bored for penetration and creosote assay retention. The retention was determined on the 0-25.4 mm (0-1.0 inch) zone of wood for both the deck panels and beams as separate units using 10 borings from each group for the AWPA A6 Toluene Extraction Assay. The results of these analyses were 338.0 kg/m^3 (21.1 pcf) for the deck panel samples and 339.6 kg/m^3 (21.2 pcf) for the beams.

Retention by assay is based on the actual volume of wood sample analyzed; whereas, the gauge retention is calculated using the total volume of wood in the charge and the total weight of preservative injected during treatment. There can be as much as 40 to 60 percent of the wood in the center of an individual piece that does not contain preservative, depending upon the species and moisture content. Also, the treated portion of wood has a gradient of preservative which decreases as the depth of penetration toward the center increases, and the greatest amount of preservative is contained in the outer 6.4 to 12.7 mm (0.25 to 0.50 inch). Therefore, this outer 25.4 mm (1.0 inch) zone will have the highest concentration of creosote when retention by assay is determined.

Kiln-dried material very often creates a problem in treating with any oil-type preservative because the potential exists to retain an excessive amount of oil in the outer extremely dry wood cells. This problem was experienced in this charge and caused an oil residue to collect on the treated material; therefore, all of the bridge components were pressure washed to remove the excess preservative from the surface. A concern remained and was supported by the high assay retention, that the material may have a potential to bleed preservative when put into service unless some of the creosote could be extracted. To address this concern, a post-conditioning low-temperature/vacuum steaming, cycle was developed and used to "clean" the surface and improve the overall condition of the material.

The low-temperature/vacuum steam, cycle subjects the material to a closed system steaming which raises the temperature to 65.6-68.3°C (150-155°F) by use of steam produced by covering the heating coils inside the cylinder with water. This eliminates the exposure of the material to any live or high temperature steam directly from the boiler. After reaching this temperature, a vacuum was applied at 74.3 -81.1 kPa (22.0-24.0 inches of Hg) with the temperature adjusted to allow a small amount of water to be pulled from the condenser unit indicating some vapors were being removed. This condition was held for 24 hours, then the remaining water and creosote extracted during the cycle was pumped from the cylinder followed by a three-hour final vacuum period to reduce the vapors and cool the material.

The low-temperature/vacuum steaming, cleanup cycle resulted in a lower assay retention of the bridge deck and beam components. The consistent original assay results on the outer 25.4 mm (1.0 inch) zone of these two groups, performed separately, allowed the assay of twelve borings following the low-temperature/vacuum steaming, cleanup cycle to be combined into one extraction sample. The results of this retention assay were 269.1 kg/m^3 (16.8 pcf) in the outer 25.4 mm (1.0 inch) zone with a 9.4 percent moisture content, indicating the success in reducing preservative retention while not increasing the moisture level of the wood in these components. The vacuum, low-temperature steaming, cleanup cycle was very successful

in cleaning the surfaces of the red maple glued-laminated bridge components and removing the air trapped inside the components. The process eliminated all in-service bleeding of the creosote except for one component which had been accidentally dead-stacked preventing the cleanup of the surface.

Preservative penetration was determined using the requirements specified in *AWPA C28 Standard for Glued-Laminated Members.* Although the current Standard does not contain any hardwood species, proposals have been made to have red maple, red oak and yellow poplar included. All core borings of the deck panels and beams were taken from the edge of laminates in the center of the laminated component. After the original treatment, the twenty borings showed complete penetration of the creosote to a depth of 63.5 mm (2.50 inches) or more with some borings in excess of 76.2 mm (3 inches) in length.

The design of the red maple glued-laminated bridge included a 6.4 mm (0.25 inch) gap between deck panels. The purpose of the gap was to allow the panel to expand as the moisture content increased from an average of 12% to approximately 20% after installation of the bridge. During the creosote treatment process, the panels adsorbed moisture and expanded to between 3.2 and 6.4 mm (0.125 and 0.25 inches). Consequently, the edges of the deck panels were butted together during installation.

CCA/Oil Treatment — The bridge rail components to be treated with the CCA/oil emulsion system were shipped to the Koppers Industries plant located in Montgomery, Alabama. These four pieces were treated in a charge of southern pine poles with a target retention of 9.6 kg/m^3 (0.6 pcf) by assay with a follow-up treatment using the oil emulsion. Again, the AWPA C28 Standard was used as a guideline for the inspection after treatment.

Twenty borings were removed from the four units and used for both penetration and assay determinations. There were two borings taken from separate face laminates on each of the bridge rails with the remaining three from the inner laminates near the center of the piece. One boring of the twenty failed to meet the 12.7 mm (0.5 inch) penetration criteria, but two rejects are allowed in the Standard to accept the treatment. The reject boring had minimal surface penetration, but all other borings had 15.2 mm (0.6 inches) or deeper with the average penetration of the CCA treatment being 30.5 mm (1.2 inches).

Assay retention analysis was conducted using an LCA X-ray fluorescence analyzer to determine the amounts of the copper, chromium and arsenate in the wood. The assay zone used in CCA material was 0.0-15.2 mm (0.0-0.6 inch) as stipulated in the C28 Standard, and this material resulted in a 12.8 kg/m^3 (0.8 pcf) retention. This retention is higher than the amount recommended for in-soil contact of hardwoods treated with CCA in USDA (1981). Before continuing with the oil emulsion treatment, the materials were air dried for three weeks to remove excessive surface moisture and enhance penetration of the oil emulsion.

After surface checks began to develop, the oil emulsion treatment was performed. The decision had been made to avoid the larger diameter boring normally conducted to obtain the sample necessary to assess the retention of oil using this treatment system and to accept the material as satisfactory without an assay being conducted for the oil. However, the oil retention was determined to be 19.2 kg/m^3 (1.2 pcf) in the outer 12.7 mm (0.5 inch) on the southern pine poles treated in the emulsion charge with the red maple laminated bridge rails.

Recommended Guidelines Summary

The characteristics of treating cylinders vary and each treating plant may require slight modifications to the recommended guidelines. The ultimate objectives of these guidelines for treating hardwood glued-laminated timbers are to achieve a minimum retention of 192.2 kg/m^3 (12.0 pcf), to have a clean surface, and to eliminate in-service bleeding. The following empty cell treating guidelines are recommended to treat red oak, red maple and yellow poplar glued-laminated timbers with creosote or creosote solutions:

- Initial air pressure should be between 137.9-206.9 kPa (20-30 psi)
- Creosote should be introduced into chamber and pressurized to 1241.1 -1310.1 kPa (180-190 psi) at 82.2 - 87.8°C (180-190°F)
- Treatment temperature should be maintained at between 93.3 and 100°C (200 and 212°F)
- Treatment temperature and pressure should continue until 192.2 kg/m^3 (12 pcf) retention (by gauge) is achieved.
- Release pressure and maintain 100°C (212°F) for a minimum of 1 hour (expansion bath at 5.6°C (10°F) above treating temperature)
- Pump back preservative
- Vacuum at 87.8 -91.0 kPa (26-27 in Hg) for 4-5 hours
- Break to atmospheric pressure
- Vacuum at 87.8-91 kPa (26.0-27.0 in. Hg) for minimum of 2 hours

- Vacuum, low-temperature steam, treatment cycle:
  a) Cover the heating coils inside the cylinder with water
  b) Raise the temperature in the cylinder to 65.6 - 68.3°C (150-155°F)
  c) After reaching the temperature apply a vacuum of 74.3 -81.1 kPa (22.0 to 24.0 in. Hg) for a minimum of 24 hours
  d) Remove water and creosote from the cylinder
  e) Final vacuum at 74.3 -81.1 kpa (22.0-24.0 in. Hg) for 3 hours
- Total minimum time in treating cylinder - 40 hours.

All glued-laminated timbers should have wood stickers placed between the components and should be securely attached using nylon straps or steel bands to the tram cars in order to minimize flotation of the timbers in the creosote solution. If the possibility exists that the outside of the timbers may be damaged due to bumping an object, wood bumper guards or similar material should be placed on critical edges or areas. Use of the recommended treating cycle guidelines along with proper handling procedures will produce undamaged glued-laminated timbers that meet the desired penetration and retention requirements.

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Determining the Length of Timber Piles in Transportation Structures

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Abstract

Knowledge of pile length is a vital component for calculating the scour resistance of bridges. A nondestructive evaluation technique based on longitudinal stress wave propagation has been developed to determine the length of timber piles. A total of 33 piles from different bridge sites were selected for equipment evaluation and verification testing of the pile length technique. The piles were evaluated by the stress wave technique, then compared to construction records for verification of length. The stress wave technique has been proven to reliably estimate pile lengths between 20 and 60 ft with an accuracy of ± 15 percent. Additional field tests have been conducted on several bridges in Tennessee and Colorado. Based on these field tests and signal processing refinements, length estimates are possible for greater than 90 percent of piles tested.

Introduction

During the late 1980's, nationwide attention was directed toward bridge failures due to scour. Recognizing the need to uniformly evaluate bridge scour, the Federal Highway Administration (FHWA) published a Technical Advisory on the scour of bridges (Federal Highway Administration 1988). This Technical Advisory is used to address the effects of bridge scour in the design process and inspection of existing structures within the National Bridge Inspection Standards Program.

Knowledge of pile length is a vital component for calculating the scour resistance of a bridge. However, records of timber pile lengths may, in many cases, be nonexistent or incomplete due to the construction practices for timber piles. Piles are typically driven until they reach a predetermined resistance, then trimmed at the end to provide a solid level surface for substructure construction. Records that specify initial pile length, depth of driven pile or length of pile trim are often unavailable, especially in older structures. Thus, it is difficult to obtain timber pile length data for scour evaluations.

Research on Timber Pile Length

Limited research has been conducted on evaluating the length of embedded timber piles. Davis (1994) described use of sonic echo and parallel seismic methods for estimating pile length. The sonic echo method is based on impacting the head of the pile and measuring the time for the reflected stress wave to reach an accelerometer, also mounted on the head of the pile. Davis reported several difficulties using this method when a structure is resting on the pile: (a) damping of the stress wave and multiple reflected waves made determination of the echo from the pile tip difficult; (b) attaching the accelerometer

to the side of the pile was difficult and (c) delivering a direct impact to the side of the pile was difficult.

The parallel seismic method used by Davis (1994) avoids the difficulty of not having access to the head of the pile. An access hole is drilled parallel to the pile, lined with a plastic tube, and filled with water. The water acts as a couplant for a transducer (typically a geophone or hydrophone) submerged in the tube to detect the transit time of impacts delivered near the top of the pile. By lowering the transducer at known increments it is possible to determine the length of the pile from a plot of transducer depth versus transit time. Cost is a primary limitation to conducting the parallel seismic test on timber piles. The parallel seismic method has achieved acceptance for concrete structures and a modified sonic echo technique (termed ultraseismic) also shows promise for higher-value structures (Jalinoos and Olson, 1996).

Douglas and Holt (1993) used analysis of bending waves in timber piles to estimate length. A signal processing technique, termed the short kernal method, was developed to allow for processing of dispersive bending waves. Dispersive waves have frequency components which travel at different velocities and therefore, render signal processing more difficult particularly when dealing with unknown geometries. Nonetheless, these researchers were able to evaluate lengths of 26 piles with reported accuracies of approximately $\pm$ 10 percent.

Research on timber poles and piles (Anthony et al, 1989; 1992) resulted in nondestructive evaluation (NDE) techniques based on longitudinal stress wave propagation that provided the means to evaluate the length of timber piles. To adapt the technology for pile length determination modifications to existing impact methods and sensor attachments were necessary, coupled with further testing on piles of known lengths. Field testing of the technique was conducted to identy the accuracy, limitations and the means of applying the pile length determination technology. Details of this development are provided in the report to the Timber Bridge Information Resource Center (Engineering Data Management 1992). References to the technique are descrii below.

Technical Approach

The timber pile length determination technique is based on longitudinal stress wave propagation. Stress waves, produced from a hammer impact travel along the length of a pile and are reflected at boundaries until dissipation of the impact energy. The stress waves travel at a velocity that is dependent on the pile density, moisture content and material quality.

Pile length can be evaluated by measuring the time required for the stress wave to travel to the base of the pile and return. The reflection time is related to the resonant frequency of the pile. The measurement of reflection time or resonant frequency and stress wave velocity enable the calculation of pile length.

The three components of the data collection process for the pile length technique, shown in Figure 1, are the excitation source for inducing the stress wave (hammer), accelerometers for measuring the pile response and a data acquisition system for recording and processing the stress wave data. When determining pile length using this technique it is crucial that the induced stress wave propagate primarily along the longitudinal axis of the pile, avoiding transverse excitation of the pile as much as possible. Moreover, the induced stress wave must be of sufficient energy to travel the length of the pile and back.

After evaluating several approaches, it was determined that a direct impact to the pile induces the most desirable stress wave. However, since the end of the pile is generally not accessible, the impact must occur below the pile cap through an attachment on the side of the pile. The attachment (a lag screw inserted at 45 degrees) is positioned such that the operator can easily swing a hammer without the interference of the bridge stringers or other components near the pile cap.

Stress waves induced from the hammer impact travel through the pile and are reflected at the ends of the pile. From the information recorded by the two sensors, located at a fixed distance apart an estimate of stress wave velocity is obtained. Pile length is calculated from stress wave velocity and the reflection time. Reflection time is the time taken for the stress wave to travel from the impact point to the bottom of the pile and back.

The resonant frequency of the pile is the inverse of the reflection time required for the stress wave to travel twice the length of the pile. The resonant frequency of a pile is, therefore, related to pile length and stress wave velocity. Thus, the length of the pile can also be estimated using the resonant frequency of the piles.

Evaluation of the Pile Length Technique

A total of 33 piles from different bridge sites were selected for equipment evaluation and verification testing of the pile length technique. The bridge sites were in four

different states. Table 1 lists the number of piles tested, the age and condition of the piles for the different sites. The Colorado piles were used during the initial stages of the project to evaluate multiple sensor configurations and hammer impact methods. The Louisiana, Tennessee and Minnesota bridge sites each contained piles of various lengths which had been recorded during construction. The piles were evaluated by the stress wave technique, then compared to construction records for verification of length.

Pile lengths from Colorado and Louisiana were estimated using only the resonant frequency method whereas piles from Tennessee and Minnesota were evaluated using both the resonant frequency method and the reflection time method.

The advantage of using both the resonant frequency and the reflection time is that it is possible to evaluate and predict lengths for a greater percentage of piles tested. There are some piles for which an estimate of the length cannot be obtained using resonant frequency because of difficulties in interpreting the frequency record. In many such cases, it is possible to obtain an estimate of the length using the reflection time. In general, the reflection time method provides a more accurate estimate for the length of piles.

As an example, a frequency record for which it is difficult to determine frequency spacing is shown in Figure 2. This frequency record has only one dominant peak (at approximately 500 Hz). It is difficult to obtain an estimate of the resonant frequency from a single peak because spacing between peaks is used to calculate the resonant frequency. For the same pile, a time domain record of the stress wave and its representation after filtering are shown in Figure 3. From Figure 3(b), an estimate of the reflection time is obtained as the time between the two marked points. Using reflection time, the length of this pile was estimated to be 30 ft The actual length obtained from the construction records, was 31 ft.

Figure 4 depicts the relationship between the actual and estimated pile length for the 33 piles listed in Table 1. Points on the figure marked with a filled circle represent the piles from Tennessee and Minnesota. These piles were evaluated using both resonant frequency and reflection time. As shown in Figure 4, the range of pile length estimates fall within $\pm$ 15 percent of the actual pile length as defined by the two lines.

Figure 1—Configuration for conducting pile length evaluation.

Table 1—Number, age and condition of piles located at specific test sites.

Location of Test Site	Number of Piles	Age of Piles (Yrs.)	Condition of Piles
Colorado	13	46 to 59	Decayed to Good
Louisiana	9	0 or Unknown	Good to New
Tennessee	7	0	New
Minnesota	4	Unknown	Good

Figure 2—Frequency record representing a 30 ft pile.

(a)

(b)

Figure 3—Time-domain representing data for a 30 ft. pile, including (a) raw data and (b) filtered data.

Figure 4—Plot of actual and estimated pile lengths for 33 piles of known length listed in Table 1.

Application of the Pile Length Technique

In addition to the piles discussed above, tests have been performed on several other bridge sites in Colorado and Tennessee. NDE data on 26 piles from three bridge sites in Tennessee were collected for predicting unknown pile lengths. These piles were older with conditions ranging from good to badly weathered. Using the pile length technique, it was possible to obtain an estimate of length for most of the piles tested.

The Colorado Department of Transportation (CDOT) proposed to use the technique to evaluate five scour-critical bridges supported by timber piles. Further, the CDOT desired to define the accuracy of the nondestructive technique in rocky and sandy soils which were not evaluated in previous research.

The pile length technique was evaluated and refined on the bridge over South Platte River near the town of Snyder. This bridge consisted of several timber pile bents with some being in water. Data were collected on 60 piles from 12 bents from the bridge, including some of the bents that were in water. Digital filtering techniques were used to smooth the wave propagation data. The data were analyzed using both time domain and frequency domain analyses. From the predicted lengths, an average length for the piles in a bent was calculated.

One task of the Colorado project was to further define the accuracy of the technique in a Colorado soil. This was to be done by extracting piles from the Snyder bridge and comparing the actual lengths to the predicted lengths. Unfortunately, the pile extraction process was extremely

difficult. In spite of CDOT's efforts, the pile extraction had to be abandoned without obtaining any actual lengths for the piles. Therefore, the technique's accuracy could not be further verified.

Task 2 of the Colorado project involved evaluation of five bridges that were identified to be in scour-critical regions with unknown timber pile lengths. These bridges were dispersed throughout southern Colorado and represented typical Colorado soils. Data were analyzed for each pile and an average length for the piles in a bent was calculated. The technique was successful in determining the length of all of the 68 piles tested. The use of the two analysis methods for determining length, one in the time domain and the other in the frequency domain, makes the technique more robust in predicting the unknown depth.

Summary

A reliable NDE method to evaluate the length of timber piles has been developed. Field-rugged impact methods and sensor attachments are available which are simple to install and use. The stress wave technique has been proven to reliably estimate pile lengths between 20 and 60 ft with an accuracy of $\pm$ 15 percent. Use of both resonant frequency and reflection time makes the technique more robust in predicting unknown length. Length estimates were possible on greater than 90 percent of the piles tested in field projects in Tennessee and Colorado. Engineers and maintenance personnel who require knowledge of pile length to evaluate the effects of scour on pile capacity will find this technique invaluable for acquiring previously unavailable information.

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Determining In-Place Modulus of Elasticity of Stress-Laminated Timber Decks Using NDE

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Abstract

The objective of this study was to examine use of stress wave nondestructive evaluation (NDE) techniques to evaluate the in-place modulus of elasticity (MOE) of the laminations in stress-laminated timber bridge decks. Prior to bridge deck construction, longitudinal stress wave and transverse vibration NDE techniques were used to determine the MOE of 40 Southern Pine lumber specimens. A strong correlative relationship was found to exist between the MOE values obtained by these two techniques. Then, several bridge decks were constructed from these specimens, and several types of instrumentation were used to measure the MOE of individual specimens within the decks prior to transverse stressing and at maximum transverse stress. Excellent agreement was found between the MOE values prior to bridge deck construction and those observed after deck construction. More importantly, average MOE values obtained from measurements of the individual laminations showed strong agreement with bridge deck MOE.

Keywords: stress wave, nondestructive evaluation, timber bridge

Introduction

Stress-laminated timber bridge decks are constructed by placing lumber laminations on edge and compressing them together with high strength steel bars (Fig. 1). The compression develops friction and load distribution between the laminations so that they act together as a large orthotropic plate. The concept of stress lamination was first developed in Canada in the rnid-1970s and was introduced in the United States in the mid-1980s. Currently, more than 300 stress-laminated decks exist in the United States, many of which were constructed through demonstration timber bridge programs of the USDA Forest Service and Federal Highway Administration (FHWA) (USDA 1995). These bridges are popular for low-volume road applications because they can be constructed using locally available wood species and labor.

With the introduction of stress-laminated timber bridges in the United States, the USDA Forest Service, Forest Products Laboratory (FPL), initiated a program to evaluate the field performance of representative bridges (Ritter and others 1991). A key element of this

Figure 1—Cross section of a typical stress-laminated timber deck system.

program is testing and analysis of the bridges to evaluate performance and formulate design requirements for vehicle load distribution. Based on data obtained from field testing, an orthotropic plate analysis program was developed that accurately models bridge behavior and load distribution under various field conditions. To verify and further refine the model, accurate information is needed on the bridge configuration, interlaminar compression, and longitudinal modulus of elasticity (MOE) at the time of the load testing. Of these bridge variables, the most difficult to assess is the MOE of the lumber laminations. Because of the variability of published MOE values within a lumber species and grade, field measurement of the actual lamination MOE is considered necessary for accurate modeling.

For several bridges in the FPL performance monitoring program, lamination MOE is measured just prior to bridge construction. This is generally accomplished by using the transverse vibration technique that provides reliable, accurate MOE data (Wacker and Ritter 1992; Ritter and others 1995). However, this type of field measurement proved expensive and difficult to schedule around bridge construction. It was subsequently determined that an in-place nondestructive evaluation (NDE) method was needed for MOE measurement. It was further determined that such a method should also be adaptable to the long-term evaluation of the bridge's stiffness and condition. As a result of these requirements, a cooperative project was initiated between the FPL and FHWA to develop the needed equipment and procedures.

The primary objective of this study was to develop the equipment and procedures required to measure MOE of lumber laminations in stress-laminated timber bridge decks. A secondary objective was to determine if the average MOE obtained from these measurements was affected by transverse stressing, and hence, could be used to estimate the MOE of a deck.

Background

Significant research and development efforts have been devoted to the use of NDE techniques for wood members. A recent review of the literature revealed that these efforts have resulted in a variety of tools for the in-place assessment of wood structural components (Ross and Pellerin 1994). The review also showed that previous and current applications of NDE have been with single members having relatively simple boundary conditions.

Figure 2—Ultrasonic measurement system used to measure stress-wave transmission time in wood products (top). Measurement system used to measure impact-induced stress wave propagation time in wood products (bottom).

One of the most widely used NDE techniques for in-place assessment is based on transmission characteristics of stress waves in the wood. Several such techniques utilize speed of stress wave transmission through the wood as an indicator of the amount of degradation contained in a particular section. Two commonly used adaptations of this concept are illustrated in Figure 2. Both use simple time-of-flight-type measurement systems to determine speed of wave propagation. In these measurement systems, a mechanical or ultrasonic impact is used to impart a wave into a member. Piezoelectric sensors are placed at two points on the member and used to sense passing of the wave. The time it takes for the wave to travel between sensors is measured and used to compute wave propagation speed. Times in excess of baseline values for wood that has not been degraded by decay fungi or other biological agents are indicative of a weakened member.

Although significant effort has been devoted to the use of NDE technology for locating degraded regions in structural members, no work has been conducted using NDE to determine the MOE of bridge decks.

Figure 3—Setup used to measure free vibration characteristics of individual laminations.

Materials and Methods

Forty Southern Pine lumber specimens were obtained from a group of 300 laminations. These specimens were to be used in the construction of several stress-laminated timber bridge deck sections. The deck sections were to be tested to failure in flexure as part of a laboratory study. Prior to construction of the bridge decks, the MOE of each lumber specimen was determined using two nondestructive testing techniques. A transverse vibration NDT technique was used to determine the flatwise MOE of each piece (Fig. 3) (Ross and others 1991).

Speed of stress wave propagation through each lumber specimen was then measured by utilizing an adaptation of the previously described technique (Fig. 4). This adaptation was developed in an attempt to arrive at a measurement technique that could be used in field applications. Note that two types of equipment were used to measure stress wave transmission times: a digital storage oscilloscope and a commercially available stress wave timer. An oscilloscope was utilized so that it would be possible to observe the actual magnitude of the stress wave. We anticipated that there would be significant interaction between individual laminations within the deck; hence, we decided it would be beneficial to examine the wave as it traveled in the specimens.

Figure 4—Technique utilized to measure impact-induced stress wave propagation times in individual laminations.

Several bridge decks were then constructed from the lumber specimens. After construction, the MOE of each lamination was measured using stress wave NDE procedures.

Results and Discussion

Prior to Deck Construction

The relationships between transverse vibration and stress wave MOE values, obtained from using both the oscilloscope and stress wave timing unit, for the individual specimens prior to bridge deck construction are shown in Figures 5 and 6. Results of a linear regression analysis comparing individual values are summarized in Table 1. Note that strong relationships were found between the MOE values for the lumber specimens.

After Deck Construction

The relationships between transverse vibration and stress wave MOE values for the laminations after the timber bridge deck construction are illustrated in Figures 7–9. Results of linear regression analyses comparing various MOE values are summarized in Table 2. It is important to note that a poor relationship was found between the stress wave MOE values measured using the commercial timing device and transverse vibration MOE values, whereas a strong relationship was observed when the oscilloscope was

Figure 5—Relationship between stress wave and transverse vibration MOE for individual laminations prior to deck construction. Note that stress wave MOE was determined by using a commercially available stress wave timing device.

Figure 6—Relationship between stress wave and transverse vibration MOE for individual laminations prior to deck construction. Note that stress wave MOE was determined by an oscilloscope.

Table 1—Results of linear regression analysis capacity stress wave (SW) and transverse vibration (TV) MOE values of lumber specimens prior to deck construction.

Equipment used	Regression equation	Correlation coefficient, R
Nicolet Model 310 digital storage oscilloscope	$MOE_{TV} = 1.014(MOE_{SW}) + 1.331$	0.93
Metriguard Model 239A stress wave timer	$MOE_{TV} = 1.015(MOE_{SW}) + 1.077$	0.93

Figure 7—Relationship between stress wave and transverse vibration MOE for individual laminations. Note that the stress wave MOE was determined for the laminations after deck construction by using a commercially available stress wave timing device.

Figure 8—Relationship between stress wave and transverse vibration MOE for individual laminations. Note that the stress wave MOE was determined for the laminations after deck construction by a digital storage oscilloscope.

Figure 9—Relationship between stress wave and transverse vibration MOE for individual laminations. Note that the stress wave MOE was determined for the laminations after deck construction (under full transverse stressing) by using a digital storage oscilloscope.

Table 2—Results of linear regression analyses comparing stress wave (SW) and transverse vibration (TV) MOE values of laminations. Stress wave MOE values were determined after deck construction.

Equipment used	Regression equation	Correlation coefficient, R
Metriguard Model 239A stress wave timer	$MOE_{TV} = 0.274(MOE_{SW}) + 12.60$	0.33
Nicolet Model 310 digital storage oscilloscope		
Unstressed	$MOE_{TV} = 0.936(MOE_{SW}) + 0.678$	0.78
Stressed	$MOE_{TV} = 0.903(MOE_{SW}) + 1.520$	0.68

Table 3—Average MOE values obtained from the various test setups.

Deck	Transverse vibration MOE (GPa)	Stress wave MOE (stressed) (GPa)	Stress wave MOE (unstressed) (GPa)	
			Oscillo-scope	Timer
1	16.20	16.41	16.20	13.65
2	16.20	16.13	17.03	11.65

utilized. Significant interaction was observed between the individual members when they were placed in the deck section. Although this interaction had little effect on the speed at which the wave propagated in a lamination, it did significantly alter the rate at which the wave lost energy. As a consequence, the signal observed by the stop transducer was lower in amplitude than what was observed in the specimen alone. This would yield longer observed transmission times, because these types of units require that the signal be of a magnitude sufficient to cross a fixed threshold voltage. This would yield a slower transmission speed and MOE value.

Average MOE Values

Average MOE values for the two decks are summarized in Table 3. Note that the average values obtained from the stress wave measurement are nearly identical to those obtained from transverse vibration testing of the individual laminations.

Conclusions

Based on the results of this study, we conclude the following:

- Stress wave MOE values of individual laminations are strongly correlated to those obtained from transverse vibration testing methods.
- Stress wave nondestructive testing techniques can be used to assess the MOE of individual laminations in a stress-laminated deck. However, commonly used stress wave timing devices may yield MOE values that are lower than expected.
- Prestressing has little effect on MOE as measured by using stress wave techniques.
- The average MOE of laminations, as measured in situ, corresponds with the average MOE of a deck.

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In-Place Detection of Decay in Timber Bridges-An Application of Stress Wave Technology

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Abstract

A study was recently conducted to assess the technical feasibility of using stress wave nondestructive evaluation methods to locate decayed members in timber bridges.

Stress wave nondestructive evaluation techniques were used to locate decayed components in several timber bridges in eastern Oregon. Various stress wave techniques were used to conduct an in-situ evaluation of stringers, decking, and compression members Components suspected as having decay were identified and evaluated in a laboratory after dismantling. Both visual evaluation of the members and subsequent laboratory testing indicated that the stress wave techniques were able to locate decayed components with a high degree of accuracy.

Keywords: NDE, stress wave, residual strength, timber bridge, MOE, compression, decay, properties

Introduction

Evaluation and upgrading of timber structures is an important area of professional practice for a structural consultant. Design strength parameters for timber are known at the time of construction. The National Design Supplement tabulates various strength parameters for different types of wood (NDS 1991). Size, grade, and species are addressed in various tables in the supplement. Once the structure is built, there is no guarantee on its life span. If properly maintained and monitored, wood structures can last a long time. With the introduction of nondestructive evaluation (NDE), problem areas can be identified and removed, increasing the life of the structure.

There are many methods of determiniug the strength characteristics of the timber components, which are categorized into two classes, destructive testing and nondestructive testing. Destructive testing can only be performed in a laboratory. Nondestructive materials evaluation is the science of identifying the mechanical properties of a piece of material without altering its end-use capabilities (Pellerin 1994). Many wood structures, such as buildings and bridges, rely on the strength properties of the timber from which they are constructed. Testing procedures used in NDE can be utilized to recognize potential problem areas in all structures. Due to the natural origins of wood, the potential is there for decay. With the introduction of NDE, it is possible to locate problem areas and fix them before they become hazardous. Through the use of stress wave technology, the progression of decay can be identified and the component evaluated for removal. Replacing components, rather than the structure, is time and cost efficient (Hoyle and Rutherford 1987).

Wood structures, generally, are subject to degradation of their structural integrity due to decay. Periodic inspection of wood structures is necessary to insure continued performance. Stress wave technology has proven to be a successful method of inspecting wood structures (Ross and Pellerin 1991). If decay is present in the member being tested, the attenuation and propagation time passing through the member is increased. Several techniques that utilize stress wave propagation time have been developed for the in-situ evaluation of wood structures. The need for NDE is expanding due to the increasing amount of resources that are being devoted to repair and rehabilitation of existing structures rather than to new construction (Ross and Pellerin 1991). Raising costs of new construction have been attributed to the increase in prices for wood. These increases had engineers looking for ways to cut their costs. The answer is to repair existing problem areas instead of replacing the entire structure. As more resources are devoted to repair, the

need for in-situ assessment of structures is evident. Stress wave technology is a cost efficient solution to the problem of in-situ evaluation.

There are three main goals in this project. The main goal in this research was to be able to validate an in-situ NDE stress wave evaluation with actual destructive and nondestructive laboratory results. The NDE is to show where decay is in the bridge and the extent of the damage caused.

The second goal of this research is to verify in-situ data with laboratory data. The goal was to correlate the two sets of data and prove that an in-situ evaluation gives accurate results. These results, in turn, would show that a timber bridge can be assessed in-situ and the results would not be subject to question.

The third goal of this research is to establish the feasibility of accurately predicting residual strength from stress wave evaluation. Correlating the stress wave evaluation with column capacity is a major part of the third goal. Using destructive compression testing, the capacity of each column can be determined. By comparing the capacity to the tabulated National Design Specifications value for sound wood, the residual capacity can be determined.

Case Studies

Two timber bridge inspection case studies are presented. Both bridges are rural bridges found in Umatilla County, Oregon. Inspection methods and procedures include sounding, stress wave velocity, and moisture measurement. These two bridges were classified by the Umatilla County Bridge Department as being deficient and in need of repair. The opportunity arose for a research team from WSU to perform a nondestructive evaluation (NDE) on these bridges to detemined their deficient components. A complete analysis was to be performed on the bridges before and after removal. After removal, the components of the bridge were taken to the Wood Materials and Engineering Laboratory (WMEL) at WSU for verification of results.

There were two main goals in this project. The first goal was to perform stress wave NDE on a bridge in place. The NDE was to show where decay was in the bridge and the extent of the damage that it had caused. NDE is more thorough and scientific than the visual inspection that most timber bridges get for evaluation. Where visual inspection will spot a good portion of the decayed material, it can also miss possible decayed sections and condemn sound sections. Eliminating the guesswork involved in a simple visual analysis is the first step in proceeding to a more scientific approach to bridge analysis. Avoiding condemnation of a structure with limited amouts of decay can minimize the extent and cost of unnecessary replacement.

The second goal of this research was to verify in-situ data with laboratory data. The goal was to correlate the two sets of data and prove that an in-situ evaluation gives accurate results. These results, in turn would show that a timber bridge can be assessed in-situ and the results would not be subject to question.

The list of deficient bridges presented by the Umatilla County Bridge Department contained two bridges that were scheduled for immediate removal. These two bridges were ideal for the proposed research, since there immediate removal would allow the laboratory evaluation to be performed soon after the in-situ evaluation.

Bridge 529

Bridge 529 was not found on the County's original list of deficient bridges. It was placed on their list and scheduled for immediate removal when, on a routine inspection, the stringers were found to be decayed on the ends.

Bridge 529 consisted of steel side trusses, concrete abutment wall, and wood decking and stringers. The stringers were untreated and sat on the abutment wall (Figure 1).

Bridge 529 spanned a creek that flowed only in the spring. During high water, it was not uncommon for water to be splashed up onto the abutment wall ledge. Having no way of draining, the water was absorbed by the untreated stringers. This additional moisture created

Figure 1—Drawing of wall and stringer.

an environment where decay could flourish in every one of the stringers. The stringers' ends decayed at approximately a 45° angle and were starting to slip off of the wall.

Inspection of Bridge 529

The inspection of Bridge 529 started with a visual inspection and evaluation. By picking out the obvious decayed material, the damage done by the high water could be determined. Through NDE, the evaluation could be taken further to determine the extent of decay.

The visual inspection yielded that the stringers had extensive decay in the ends that sat on the abutment wall. This determination matched Umatilla County's bridge inspectors evaluation. The condition of the tops of the stringers, where the decking was nailed to them, was uncertain. The decking appeared to be in good shape. Since the decking had no wearing course, water was free to fall between the deck planks onto the tops of the stringers below. The potential for decay existed in the tops of the stringers since water was able to penetrate to the them through the spaces between the deck pkanks.

The second phase of NDE was to start testing the components. The stringers were marked at two foot intervals lengthwise, two inches from each edge, and on centerline transverse. The stringers were measured to be 3.75 inches by 11.75 inches by 20 feet long. Transverse stress wave evaluation was completed on the stringers at the above intervals. Longitudinal stress wave times were taken for a few select stringers. The stop accelerometer was clamped to the decayed end due to that end being inaccessible. The accelerometer was clamped as close to the decayed end as possible.

The inspection of Bridge 529 was completed using a Nicolet 410 oscilloscope and Metriguard 239A stress wave timer. Longitudinal analysis of the decking was completed using the oscilloscope. Each deck member was evaluated twice for accuracy. The stringers were evaluated by means of transverse stress wave evaluation.

Analysis of Field Data— The stringer's modulus of elasticity (MOE) was calculated from the longitudinal stress wave velocity and mass density, which was derived from the weight and dimensions of the member. Using a template, stress wave times were recorded and probable decay areas marked out.

Testing at the Wood Materials Engineering Laboratory— Six stringers and five deck planks were brought back to the WMEL at WSU and reanalyzed. All of the stringers were again visually inspected for decay. The ends of the stringers had excessive decay where they sat on the abutment wall. It was observed that a couple of the stringers had decay pockets along the top where the deck planks were secured. The stringers were sketched and the visual decay marked on the drawings (Figure 2).

The stringers were evaluated in the laboratory the same way they were tested in the field. The stringers underwent longitudinal and transverse stress wave evaluation. In addition to the longitudinal stress wave evaluation with the oscilloscope, additional procedures were used to evaluate the stringers. Using the Dolch PC, pulse-echo times were recorded and their corresponding MOE calculated. The Metriguard stress wave timer was also used to record stress wave times.

The deck members of Bridge 529 were evaluated using longitudinal stress wave evaluation and compared to the results that we obtained in the field. The stress wave times were converted to MOE values and then graphed against each other.

A regression coefficient (R^2) value of 0.79 was obtained from the in-situ versus laboratory longitudinal stress wave times. The regression coefficient predicts the variability in the data, with an R^2 of 1 being perfect. Figure 3 shows the data of in-situ longitudinal stress wave time graphed versus laboratory longitudinal stress wave time.

Figure 2—Sketchs of Bridge 529 stringers and decayed areas.

Figure 3--Laboratory stress wave time versus in-situ stress wave time for Bridge 529.

In summary the results for Bridge 529 showed that there was severe decay damage to the ends of the stringers and no decay damage to the decking. Longitudinal stress wave times are often analyzed as microseconds per foot (ps/ft) rather than inches per millisecond. Values for sound wood, moderate decay, and severe decay can all be derived from the values given in Table 1. Generally, longitudinal stress waves in sound wood travel at a rate of approximately 60 µs/ft and transverse waves travel at a rate of approximately 200 µs/ft in untreated sound wood.

The stress wave travel rates in the stringers varied from 56 µs/ft to 64 µs/ft using a Nicolet 410 oscilloscope. In-situ evaluation of the stringers was used with a clamped accelerometer due an end being restricted. Having the end that was decayed bypassed, the longitudinal stress wave times were not representative of the entire length of the stringer. There is obvious decay in all of the stringer ends. However, looking at the members stress wave transit rate, no decay would be assumed. If the Dolch PC pulse-echo technique were used, an analysis could be done from end to end. The Dolch analysis produced travel rates between 56 µs/ft to 74 µs/ft. The variation is small between the Dolch and Nicolet, but noticeable. The difference in times is attributed to the decayed end.

The decking yielded results that the members were structurally sound without any noticeable decay.

The NDE of Bridge 529 produced similar results for the in-situ evaluation and the laboratory evaluation. The variability between the tests that were performed was small. The evaluation of the deck members yielded a 0.79 R^2 regression coefficient. The quality of the deck members was high. There was no visible or detectable decay in any of the deck members. The treated decking had held up to the weather and abuse sustained by traffic. However, the stringers did not fair as well as the decking. The untreated stringers were very susceptible to decay. They were exposed to excessive moisture caused by overspray from the creek at high water. Improper drainage capacity on the abutment allowed the water to sit on the sill until the stringers absorbed it. The combination of being untreated and sitting in water led to the excessive decay encountered in the NDE of the stringers.

Bridge 117

Bridge 117 was on the Bridge Department's list for immediate replacement. The inspector for the county had assessed the bridge and determined that the columns that supported the bridge were deficient.

Inspection of Bridge 117— The NDE of Bridge 117 started with a visual inspection. Overall, the bridge was in good shape except for the columns and wings. The columns (Figures 4 and 5) had a few visual signs of decay. Water had leaked through the deck and had introduced moisture to the stringers, caps, and columns. Bridge 117 spans a runoff creek bed that was dry at the time of our evaluation. The bridge was on a rural road that was subjected to loaded grain trucks and combines.

A complete NDE of Bridge 117 was accomplished in four hours time. Longitudinal stress wave evaluation was performed on all of the deck members using the Nicolet 410 Oscilloscope. The transverse evaluation of the caps, columns and stringers was performed with the Metriguard 239A stress wave timer. No signs of decay

Figure 4—Columns 1-5

Figure 5—Columns 7-11

were evident in the visual inspection of the stringers. Two stringers were marked and evaluated with transverse times at every two feet intervals at two inches from the top and bottom, and at the centerline. The outside two stringers were picked for laboratory for evaluation since they were the most susceptible to weathering and decay. The columns were marked every two feet and transverse stress wave times were taken at the centerline of each column. Column 11 showed advance signs of decay due to high stress wave times and the transverse stress wave spacing was reduced to six inches. The caps were tested at the location where every column was connected.

Analysis of Field Data— The field analysis was broke down into three groups: decking, caps and stringer, and columns. The first group of data that was analyzed was the decking. The longitudinal stress wave evaluation did not show any deficient members. All 19 members were tested twice for accuracy. From the NDE results, it was determined that the decking was in good shape and was not in need of replacement.

The NDE of the bridge proved that the caps and stringers were in excellent shape. Questionable areas on the stringers were marked to be verified by the laboratory testing, but generally the analysis proved the stringers to be in excellent condition. The caps had very consistent times throughout the respective lengths of each member. The end of the south cap did have a higher stress wave time than the rest of the timber and, therefore, was marked for laboratory verification.

The main reason the bridge was on the deficient list was due to the columns. Visual inspection revealed that only a couple of the columns had obvious areas of decay. The stress wave evaluation was conducted by inking readings at the bottom, top, and middle of the columns along the centerline. Through NDE, all of the columns were determined to have some decay.

Laboratory Evaluation of Bridge 117— The laboratory evaluation at the WMEL began with a visual inspection of all the components. The stringers, decking and caps appeared to be structurally sound. Column inspection revealed excessive decay to most of the bottoms and a few of the tops. Figure 6 shows the tops of the columns and the decay damage; Figure 7 shows the bottoms of the columns.

Figure 6—Tops of Columns

Figure 7—Bottoms of Columns

The second phase of the evaluation in the laboratory was to verify the results we tabulated with our in-situ assessment. The testing methods in the laboratory would follow, as closely as possible, to the in-situ testing that took place. Further laboratory testing would expand the testing done in the field in order to map out the decayed sections in the components of the bridge.

Caps And Stringers— The caps and stringers were the first to undergo the stress wave evaluation. The stringers were determined to be structurally sound with no decay. Due the good quality of the timbers, only two stringers were tested in the field. Both outside stringers were tested since they would be the most probable of being deficient due to being the only two exposed to weather. Field results and conclusions were verified by the laboratory results.

In-situ, the caps had been tested immediately next to where the columns came into them at the centerline of the cap. Stress wave times at the WMEL were taken on the centerline as well as three inches off centerline each way. Field results were again verified by the laboratory results. The questionable end of the south cap that raised concern in the analysis of the field data was verified by the laboratory results showing high stress wave times. Visual inspection of the end showed a split that caused the high stress wave times. Overall, the caps were in excellent shape except for a few splits. Rebar had been driven into the cups where the columns were connected to the caps. This rebar had initiated and amplified some of these splits.

Columns— The columns underwent an extensive stress wave evaluation to determine the extent of decay in each one. Values for sound wood, moderate decay, and severe decay for this project were configured and tabulated in Table 1.

Table 1—Laboratory conclusions for decay detection.

Column Condition	Stress wave time (µs/ft)
Sound wood	250-300
Incipient decay	300-500
Moderate decay	500-800
Severe decay	800-2000
Extreme decay (shell only)	2000+

In the laboratory the columns were evaluated from side to side and front to back. The field times were side to side transmission. Front to back times were taken to help map out possible decay areas. Many of the bottom ends of the columns were so badly degraded that it was not possible to get stress wave times.

The field evaluation correlated well to the laboratory results, both identifying decayed areas. The direct correlation of times between the laboratory and in-situ testing was not a goal of this research. The interest was more in the fact that both procedures were uble to spot problem areas.

All but four of the columns were loaded to failure in compression. The setup for the compression test can be seen in Figure 8. Figures 9 and 10 show the top and bottom, respectively, of column 5 before testing. Before the compression test was performed, the ends were squared. The top of the column can be seen to have extreme rot in the center and only a shell of good wood holding the column together. Creosote penetration of the column top was minimal. The bottom of the column did not show any visual signs of severe decay. However, the stress wave analysis showed that the entire column was decayed.

Column 12 was a corner column, as was column 5, that did not have a large amount of decay, compared to column 5. After the ends were squared the column was visually inspected for internal decay. The visual inspection showed that there was no decay present. However, the stress wave analysis showed that there was a pocket of decay in the bottom end of the column. Figures 11 and 12 show the top and bottom of column 12.

Figure 8—Column compression test

Figure 9—Top of column five

Figure 10—Bottom of column five

The columns were loaded until failure. Column 5 failed abruptly (Figure l3) at an ultimate load of 37.5 kips. The National Design Specifications (NDS 1991) Supplement tabulates compression parallel-to-grain values for Douglas-fir Larch as 700 psi for No. 2 grade in Table 4D. Adjusting for loading rate and changing the tabulated fifth percentile to the median value, the capacity should be 258 kips. The load duration factor can be calculated from Figure 14 (National Design Specifications 1991). The columns' low capacity is a direct result of the extensive decay in the column. Decay had completely deteriorated the interior of the column. Figure 15 shows column 5 after the compression test.

Column 12 was loaded to an ultimate load of 172.0 kips. The column failed by a compression failure where the end was decayed (Figure 16). The visual inspection failed to identify the decayed area. However, the stress wave evaluation of column 12 showed a high stress wave time at the point that it failed in compression. The decay pocket can be seen in Figure 16 on the upper left corner of the picture.

Figure 11—Top of column 12

Figure 12—Bottom of column 12

Figure 13—Column #5 at failure in the compression test

The rest of the columns, except three which exceeded the machine capacity and one that fell apart before testing, were tested and their capacities compared to the NDS (1991) value for sound wood. Figure 17 is a graph of the columns' capacity divided by the NDS (1991) value for sound wood versus stress wave time. The capacity is in terms of percentages of the NDS

Figure 14—Load duration factors for various load duration (NDS 1991)

(1991) value. The worst cross section of the column from the transverse stress wave evaluation was used in the graph. It was assumed that failure would occur at the weakest cross section of the column. An R^2 regression coefficient of 0.91 was determined through least squares linear regression. The high regression coefficient shows that the variability in the data is low and that using stress wave evaluation to predict residual compressive strength is a strong possibility.

Figure 15—Column #5 after failure

Decking— Longitudinal stress wave evaluation of the decking produced results that showed no decay throughout the members. The Nicolet 410 Oscilloscope was used to obtain the longitudinal stress wave transit times in the laboratory. Two readings were taken in the field for accuracy. Five readings were taken at the WMEL for increased accuracy. The

Figure 16—Column #12 after failure

Figure 17—Graph of column capacity/NDS median capacity versus transverse stress wave time

Figure 18—Graph of field longitudinal stress wave times versus laboratory longitudinal stress wave times

averages of each were plotted against each other and can be seen in Figure 18.

In summary, Bridge 117 turned out to be a perfect case study. The in-situ NDE of Bridge 117 was verified by the laboratory NDE. The bridge was placed in 1965, giving it a 30 year life span at the time of removal. The bridge, given the life span, was in remarkable shape, except for a few components. Deterioration due to decay in the columns and backwalls had made the replacement of the bridge necessary.

The decking was in excellent shape with no visible or detectable decay. Longitudinal transit rates for the stress waves averaged 64 µs/ft for the laboratory evaluation and 63 µs/ft for in-situ evaluation. The regression coefficient between in-situ stress wave times and laboratory times produced an R^2 value of 0.83. The high regression coefficient, and almost exact transit rate average, lead to the conclusion that results obtained in the field could be reproduced easily in the laboratory for longitudinal stress wave evaluation.

The caps and stringers were evaluated at the same locations as those in the field. Longitudinal stress wave evaluation was not performed on either component. Instead, a more detailed transverse stress wave evaluation was performed on the members to determine the presence of decay. Initial in-situ evaluation of the stringers and caps yielded results of showing no decay in the members. The caps were evaluated where the columns were attached. The cap ends were the only places that higher stress wave times were recorded. The south cap had one particular place on the end that produced a high stress wave time. Decay was not a factor in the high time obtained in the field. This was verified by the laboratory results in that the high stress wave time was caused by a split. In general, the stringers all looked to be in excellent shape. Two were brought back to the WMEL for verification of results. The NDE of the stringers in the laboratory verified the in-situ analysis. The stringers were in excellent shape and the results for both tests verified this.

Reasons for the deterioration of the columns were obvious when the laboratory evaluation was completed. As can be seen in Figures 9 and 11, the penetration of the creosote was poor in the tops of the columns when compared to the penetration in the bottoms (Figures 10 and 12). All of the columns had both ends squared for the compression test. After squaring the ends, it was hypothesized that the columns had been 'cut to fit' when they were being placed. The tops were cut so they would fit the bridge at the time of construction. Additional treatment had been applied to the tops after they were cut, but the penetration was poor. Essentially, this created a treated column with the top being penetrated more easily by moisture. The treated shell helped keep the moisture in, accelerating the decay process. The hypothesis is backed by the fact that all of the bottoms were decayed in the columns (Figure 7) where the moisture was locked in. With the exception of column 5, the tops of the columns showed little visible decay. Both the stress wave evaluation in the field and in the laboratory spotted excessive decay in all of the columns.

All of the columns except four were loaded in compression until failure. Three of the columns

exceeded the capability of the testing machine, and one fell apart before it could be tested. For a 12 inch x 12 inch Douglasfir timber post, The capacity is 258 kips when adjusted for load duration and median value. The values obtained in the compression test are considerably lower than the median value specified by the NDS (1991), The high regression coefficient (0.91) showed that the variability in the data was low. Using the stress wave times to predict the column capacity can be performed by using the graph.

Conclusions

Through the use of stress wave technology, researchers and engineers have been able to evaluate structures for deterioration without harming the structure. Testing procedures used in NDE can be utilized to recognize potential problem areas in structures. Rather than removing the structure itself, these problem areas can be removed, saving time and money.

The two bridge case studies showed the application of nondestructive testing on in-situ structures. The field data can be reduced and the structure evaluated for decay. By being able to reproduce the field results with destructive and nondestructive laboratory results, confidence in future field evaluations without laboratory verification grew. In-situ assessment is relatively easy when applying the nondestructive stress wave evaluation techniques.

Stress wave velocity technology is valuable in determining the soundness of timber structures. It has been shown to be a valid and accurate method for mapping the extent of unsoundness due to decay in timber structures. Considerable effort has been devoted to developing NDE techniques for assessing the performance of timber structures.

The use of stress wave velocity to detect rot, other defects, and their extent in timber structures is practical. It is time and cost efficient to identify and remove individual components, rather then the entire structure. Stress wave techniques have proven to be reproducible and an accurate way of mapping decay.

In years past, researchers have been unable to predict residual strength of various components. In part, this is attributed to the lack of a correlation between residual capacity and stress wave times. With the additional destructive compression testing of the columns, a correlation between residual capacity and stress wave time has been established. Initial results from the tests that were ran show that it is feasible to use stress wave evaluation to predict residual capacity.

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Development of Limit States Design Procedures for Timber Bridges

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Abstract

Recent work in North America, Europe, and Australia has resulted in the development of Limit States or Load and Resistance Factor Design (LRFD) procedures for timber bridges. This new approach makes significant departures from the current allowable (working) stress design procedures, which have often been based on the design of domestic building structures. This paper discusses the development of limit states design procedures for timber bridges in Australia and the United States, with a particular focus on the design of stress-laminated timber bridge decks and girder bridges. Issues such as characterisation of material properties, reliability, strength, and serviceability are also presented.

Keywords: Timber, Bridges, Design, Limit States, Load and resistance factor design, procedures.

Introduction

With any structural material, successful design is a function of the adherence to realistic and safe design criteria. In most situations, these criteria are set in design standards. Historically, design standards have focused solely on strength considerations based on allowable stress design criteria which limit the stresses induced by loads to values less than or equal to permissible stresses defined in the standards.

The current edition of the Australian Timber Structures Design Code (AS 1720.1-1988) is written in an allowable stress format and is one of the few structural design codes used by designers in Australia which is not yet in a limit states format (Standards Australia 1988). A soft conversion to limit states format was published in July 1994 as DR 94276, a draft document for public comment. This document has undergone several significant revisions and it is anticipated that the final version will be published in late 1996. A similar development process has occurred for timber bridge design in the United States where a load and resistance factor design (LRFD) code, *LRFD Bridge Design Specifications*, was recently published by the American Association of State Highway and Transportation Officials (AASHTO).

Timber Design Codes

Historical Developments

Most timber codes that were used throughout the world until the 1980s were in an allowable stress format. The permissible stresses for these codes were based on extensive research into the properties of clear wood samples tested in the first half of this century. Clear wood samples are small pieces of wood that are free from any sign of obvious strength-reducing characteristics. The absence of strength-reducing characteristics means that the properties are essentially those of the wood fibres aligned

in the most favourable orientation. In general, the clear wood samples provide an indication of the upper limit of the expected performance of realistic timber pieces.

Derivation of Strength Properties

Design standards correct the clear wood sample data by reducing the basic species strength data to allow for the expected weakening caused by such characteristics as slope of grain, knots, checks, and splits. Over the years, these adjustments varied to ensure that various grades of some species showed satisfactory performance in service. However for most species, no data was available to relate the performance of the small clear samples on which the design properties were based to the performance of in-grade timber. In some respects, this approach represents the opposite that is required for design purposes, because the critical design criteria is not how strong a piece of timber is, but rather how weak it could be.

Part of the move towards reliability-based design was the need to have a clear understanding of the actual properties of the timber used in service. This led to the implementation of in-grade testing programs which assign the properties of timber samples randomly obtained from stocks of commercially available timber. Because commercially available timber has characteristics that affect the strength of the timber, and these characteristics can be randomly positioned in service spans, the in-grade test requirements in Australia call for random positioning of the test pieces within the test span. In-grade testing in other countries biases the location of defects, which generally results in lower characteristic strengths for pieces of small end section timber (less than 150 mm (6 in.) deep), but results in similar strength results for larger sizes as a result of the larger test span required for deeper sections.

Data from in-grade testing show considerable variation, which represents the true variability in properties of commercially available timber. These data can be used to determine either allowable stresses or characteristic strengths of the tested grade of timber; however, the sample size required is very large if the data are to be statistically significant.

Reliability of Material Properties

In-grade testing gives a measure of both the properties and the variance of the properties of commercially available timber. This more accurately represents actual component properties compared to the small, clear wood sample data. The current Australian Timber Structures code (AS 1720.1) recognises this in engineering applications through the use of a strength modification factor known as the k_2 factor (Amendment 1 - AS 1720.1)(Standards Australia 1993). This factor reduces strength data for timber to be used in places with severe consequences of failure, but strength properties derived from in-grade data receive a lower penalty than the properties derived from small, clear sample data. This effect is also reflected in the modification factors specified in the forthcoming limit states version of AS 1720.1-1996.

One of the driving forces for the incorporation of such a "reliability" strength modification factor in an allowable stress code was the fact that in Australia, there are no design procedures available for timber bridges other than the general structural provisions of AS 1720.1. The one exception to this is for the design of stress-laminated timber bridge decks, which is discussed later in this paper. The lack of suitable design code provisions for timber bridges has presented bridge designers with a number of problems, not the least of which is a general ignorance and lack of understanding of timber engineering among the design professions.

Limit States Design Procedures

Why "Limit States"?

Limit states design reflects a global trend. Plans are in place for the use of limit states design codes for all engineering materials in Europe, North America, Australia, New Zealand, and more recently in Asia. This global trend has a sound rational basis, as variations in capacity reduction (ø) factors can be used to account more accurately for variability in properties or capacity models. This results in a more uniform and quantifiable level of reliability than can be achieved with allowable stress design.

Limit states codes separate loads expected for the serviceability limit state from those expected for the strength limit state. This is the reality of design. The parts of any timber design code that pertain to deflection will change very little in a limit states code, and so will produce similar results to those of the current allowable stress code at similar loads to the current working loads. This is significant, as many structural elements are controlled by the serviceability limit state.

A number of limit states design codes for timber structures are currently in use throughout the world. Of special note is the Canadian code which has made substantial use of Canadian in-grade data to produce a reliability-based limit states code (Canadian Standards Association 1984). The draft European code, *EuroCode 5* (1992), makes use of a wealth of data on the behaviour of European softwood timber and connections but differs markedly from the Canadian standard in format. This code has no official status as yet, but will serve as a

model for national European codes to be developed in the future. New Zealand's timber design code (Standards New Zealand 1993) has also used in-grade data for their principal structural species, radiate pine, to produce a soft conversion of a permissible stress format code to a limit states code. The USA has developed and implemented a limit states timber structures code which is also quite different to other existing limit states codes and drafts (Gramala et al. 1994; ASCE 1995).

Limit States Philosophy

Traditionally, allowable stress design codes have focused on providing adequate strength and achieving a level of safety acceptable to the general community. However, limit states philosophy recognises that the community in general, and clients in particular, are also interested in other design considerations such as serviceability, safety, and stability. If performance by any of these criteria is unsatisfactory, the "limits" in one or more of the limit states are exceeded. There are others as well. The fire limit state is a special case, where the loads that must be carried by a structure that has been partially damaged by fire are of primary consideration. Another is the fatigue limit state in which a structure or component that is loaded and unloaded repeatedly throughout its life must perform satisfactorily with acceptable fatigue damage.

Each of the performance limit states is different, representing a different loading scenario in the life of a structure. Violation of the limits produces different consequences for each of the performance limits mentioned. Design for a limit state aims to produce satisfactory performance to the limit prescribed in each of the performance states (Boughton and Crews 1996). In the design of timber structures, it is primarily the serviceability and strength limit states that will be the first consideration, with checking required to ensure adequate performance in other limit states.

Serviceability Limit States

Under prescribed loads, there may be limits on deflection (all materials), cracking (concrete, composite structures), and vibration (all materials). If performance is beyond these limits, the client may find the performance of the structure unacceptable. In many cases, these limits are not prescribed in codes, but are left to the discretion of the designer and client. Where the serviceability limits are violated, there may be inconvenience in the operation of the structure or some damage to non-structural elements, such as the asphalt wearing surface on timber bridges. In general, the damage can be repaired as part of routine maintenance, but rectifying the fault that caused the damage is often very difficult as it involves modification of the basic structure.

Violations of the serviceability limit states are not catastrophes but produce annoyances requiring extra work and expense for the client. If the violations occur less frequently than typical routine maintenance, they are generally considered as a separate maintenance issue. However, if the violations occur more frequently than typical routine maintenance, the required repair work becomes an additional burden and the client will have justifiable cause for complaint.

The serviceability load combinations specified in most limit states design codes are reasonable estimates of combinations of loads likely to cause maintenance problems or affect the performance of the structure. Under most codes, the probability of exceeding the serviceability load combinations is approximately 5% per year.

Strength Limit States

Violations of the strength limit state imply structural failure. Thus, it is important that there be a very low probability that the strength limit state will be exceeded over the life of the structure. Failure may be due to overloading and/or under-performance. Overloading is addressed by the use of appropriate load factors and under-performance is addressed by both a capacity reduction factor and the use of conservative strength capacities derived from the lower 5th percentile of test data for timber.

Figure 1 shows a classic probability distribution model for loads. Load effects can be approximated by normal distributions, log-normal distributions, or Gumbel distributions. In viewing the probability distribution, the probability of exceeding a given load (z) is given by the area to the right of the load (shaded in Figure 1) as a fractile of the entire area under the distribution curve. The AUSTROADS bridge design code defines the limit states probabilities as shown in Table 1.

Figure 1— Probability distribution model for typical loads on a structure.

Table 1—Definition of Limit States probabilities (AUSTROADS).

Limit State	Return interval of load	Probability of exceedance in any 1 year
Servicability	20 years	0.05
Ultimate (strength)	2,000 years	0.0005

In the same way that there is a probability distribution for loads, there is also a probability distribution model for strength resulting from material variation, failure mechanics, and variation in workmanship. The probability distribution model for strength is illustrated in Figure 2 and is similar to the curve for loading. The principle concern in this diagram is the chance of not attaining a given strength. The area under the curve to the left of a value expressed as a fractile of the total area under the curve gives the probability of not attaining that strength.

By overlaying the load distribution curve and the strength distribution curve, it can be seen that the strength curve lies to the right of the load curve (Figure 3). As the two curves come closer together, the probability of failure increases. Therefore, the distributions must be separated as necessary to reflect an acceptable low probability of failure. Factors in design codes are chosen to give a probability of failure that is calibrated to acceptable design performance and practice. In designing or checking for the strength limit state, designers ensure that the probability that the factored capacity is less than the factored loads is very small.

Capacity Design Equations

The design equation for the strength limit state is:

$$R_d \geq S^* \qquad \textbf{(1)}$$

where R_d is the design ultimate capacity of the member and S* is the design action effect due to the factored design loads imposed on the particular element. The design ultimate capacity is derived from the nominal capacity of the section multiplied by the material capacity reduction factor as shown in the following:

$$R_d = \phi R_u \qquad (2)$$

In this case, R_u is the nominal capacity and ø is the material capacity reduction factor, defined by the code, to give an appropriate level of safety for the type of structural element under consideration. The nominal capacity is stated in terms of a tabulated characteristic value, multiplied by the relevant strength modification factors, which are designated as "k" factors in the Australian code. This is defined in the following:

$$R_u = k_{\mathbf{mod}}\, R_k \qquad (3)$$

where K_{mod} is the total effect of the strength modification factors that are relevant to the design action (e.g., duration of load, load sharing, size, stability), and R_k is the basic characteristic strength of the structural member. Details of the AASHTO LRFD equations have a similar format and are discussed in detail elsewhere (Ritter and Nowak 1994).

The principal differences between an allowable stress design code and a limit states code are in the design philosophy, the loads, and the basic design stresses. By comparison with Equations (1) to (3), allowable stress codes generally present the same design equations in the

Figure 2—Probability distribution model for strength.

Figure 3—Relative positions of load and strength probability distributions.

form given in the following:

$$f'_k \leq k_{\mathbf{mod}} F'_k \qquad (4)$$

where f'_k is the actual stress F'_k is the basic allowable stress for the design action under consideration. In limit states design, the design effects for live load, dead load, and combinations of loads must be compared with the factored design capacities obtained from the previous equations. In all cases the design capacity must be greater than or equal to the load effect if the strength limit states are to be satisfied. As the design loads are factored to give those for the very rarely occurring limit state "event", the ultimate loads are substantially larger than the actual working loads. The strength side of the equation must also correspond to the capacity of the member under the type of loading anticipated in the strength limit state event. The characteristic stress given in the limit states code must therefore be a stress which could reasonably be expected to be achieved just prior to the failure of the member. These characteristic stresses are substantially greater than the basic allowable stresses, recognizing that in limit states format, the factor of safety is principally applied to the loads, rather than the stresses.

Interim Methods for Improving Reliability

Most of the timber structures which formed the experience base for development of AS 1720.1 and similar allowable stress codes were essentially "domestic structures" which contain many parallel elements. If failure of one member occurred, it would normally lead to load redistribution to adjacent elements. The magnitude and nature of loads in these essentially low rise residential structures are also such that in many cases, a "failure" may go unnoticed. However, many modern timber structures, such as glulam or structural composite lumber (SCL) portal frames and bridges, have little redundancy so that redistribution of load is difficult or not possible. Failure in these circumstances may prove catastrophic. It was only as in-grade data started to become available that the code committees could properly evaluate the level of reliability of timber structures.

It was clear that where parallel systems were available and where the cost and risk of failure were low (e.g., in domestic structures), the level of reliability of the entire structure was adequate. However, where such load sharing was not available or where the loads were of greater magnitude and much more concentrated, the level of reliability in timber structures needed to be increased, particularly for timber bridges. The "material and application" factor, k_2, was introduced in an amendment to AS 1720.1 in 1993. The factor was intended to correct the overall reliability of timber structures, which were still being designed using a allowable stress code, to bring it into line with steel and concrete structures which were already being designed using reliability-based, limit states design codes. The k_2 factor as it currently stands affects only those structural elements where load redistribution is not possible and/or where failure of the element may prove catastrophic; i.e., it is concerned with the effect of failure of an element on the "system" behaviour. Because an in-grade evaluation of structural properties gives a higher level of reliability in the properties than those inferred from the testing of small clear samples, the k_2 factor for material with in-grade properties is greater than that with properties found by other methods.

Application of Limit States to Timber Bridges

Historical Developments

In Australia, design of all bridges is defined by the AUSTROADS Bridge Design Code, which was published in a Limit States format in 1992. This code has a reliability basis for determining loads for strength and serviceability limit states and the design of all aspects of bridges constructed from concrete and/or steel. Although the AUSTROADS code is specific to road and pedestrian bridges, the format and technical provisions are similar to the Standards Australia Limit States structural design codes, which have been in use since the late 1980s. However, there are no AUSTROADS provisions for timber bridges and engineers assessing or designing such structures have been forced to use the existing allowable stress version of AS 1720.1 - 1988. For most engineers, this is difficult since they are unfamiliar with allowable stress timber design criteria. Additionally, timber design is outside of the experience of most practicing engineers.

For bridge designers in the United States, many of these problems have been essentially addressed by the AASHTO LRFD provisions for design of timber bridges. The development of similar procedures in Australia is still underway, although most issues will be addressed with the publication of the Limit States version of AS 1720.1 at the end of 1996.

The first Australian version of a limit states design code for timber structures was published in April 1995 (Crews 1995). Although this code is specific to stress-laminated timber (SLT) bridge decks, it contains principles which are generally applicable to the design of engineered timber structures. This design specification references the loading provisions of Section 2 of the AUSTROADS code.

Defining Appropriate Reliability Levels

One of the problems facing development of the limit states format for timber bridges is the fact that most material properties used throughout the world have been derived on the basis of building loads rather than the greater or more concentrated loads associated with bridge structures. As noted previously, this necessitates higher levels of safety and reliability. This has been partially addressed in Australia for both the existing allowable stress design timber structures code and the limit states design code for SLT bridges, by use of the k_2 strength modification factor.

Bridge structures also require a high factor of safety which must be addressed not only for loads but also in the characteristic material strength properties. The effect of this in a limit states format is that the material properties or characteristic strengths specified by such codes may in fact have a lower than desirable β safety index. For example, the derivation of fifth percentile modulus of rupture (MOR) values, which form the basis of the characteristic strength properties of timber, has been based on a 75'% tolerance level which is acceptable for normal building applications. However, in order to achieve the higher degree of reliability which is necessitated by a bridge structure, the tolerance level for prediction of fifth percentile values should possibly be as high as a 95% confidence level. This would effectively lower the fifth percentile estimate of MOR or the characteristic strengths in order to obtain suitable design properties for bridges. Similarly, a first percentile rather than a fifth percentile value could be used, using the existing statistical methods for characterising material properties, with much the same effect.

Modification Factors

Possibly the simplest and most desirable approach would be to adjust the current industry values by a "bridge" or "consequence of failure" adjustment factor applied by the designer. This is the method which has been adopted in most design codes through selection of appropriate values for the capacity reduction factor. This is an alternative to independently developing a specific set of characteristic strength properties for heavily loaded, engineered timber structures such as bridges.

In the current allowable stress edition of AS1720.1, it is not possible to incorporate a material capacity reduction factor (ø), so an increase to the β safety index has been effected for engineered timber structures by including the k_2 material and application factor. For the purpose of the first edition of the limit states design procedure for SLT bridges, the k_2 factor has been used as the modifier for the material properties rather than changing the basic characteristic strength properties or ø.

The current values of k_2 in AS 1720.1 are 0.9 and 0.7 for characteristic properties derived by in-grade and small clear samples, respectively. Initial research by Crews indicated that these values should be reduced to 0.85 and 0.65 respectively, for bridges (Crews 1994). Although this approach appears to increase the safety index to an acceptable level for bridge structures, further research needs to be done in this area. Similar work is being undertaken in the United States and has been previously presented (Ritter and Williamson 1991).

For the design of SLT bridge decks, k_2 will equal 0.85 if the material properties have been obtained using an in-grade testing program, or will equal 0.65 if the material properties have been derived by any other method (e.g., from testing of small clear samples).

Development of a "True" Reliability

The intention of the k_2 factor in the Australian timber codes has been to ensure that the level of reliability of timber in higher risk/consequence of failure "engineered" structures is consistent with that for similar structures designed using other engineering materials, such as steel or reinforced concrete. Although the k_2 factor is a responsible and convenient way to increase reliability for an allowable stress design code, it is not really appropriate for use in a limit states code where the reliability of material properties or the capacity of the structural element is generally modified by a capacity reduction (ø) factor.

Recent work by Crews (1966) proposed that k_2 be removed from the limit states code (draft) and that it be replaced by a table of appropriate derived ø factors. This will simplify use of the code by eliminating a "k" factor and "doubling up" on reliability modifiers (i.e., currently a designer must use both k_2 and a ø factor). It will also permit the use of ø factors which have a logical connection with a specific safety index and level of reliability for the timber product being designed. The research underlying this proposal, which has been accepted for inclusion in AS 1720.1-1996, focuses on a detailed probabilistic analysis of load effects and material variability in the determination of safety indices and levels of reliability for timber structures, which are appropriate to their end use and the consequence of failure (Leicester 1985; Foschi et al. 1989). The outcome of this research program is a table of ø factors, which are reproduced in Table 2.

These capacity reduction factors effectively decrease the probability of failure, due to a combination of excessive overload with grossly substandard timber strength, to levels appropriate for the type of structure being designed.

Table 2—Proposed capacity reduction (ø) factors for AS 1720.1.

Properties group		Consequence of failure Category 3	Consequence of failure Category 2	Consequence of failure Category 1
Group	**Typical examples**	**Minor significance**	**Normal significance**	**High significance**
A	SCL & plywood	0.9	0.85	0.80
B	MGP pine & glulam (ingrade with quality control and ongoing verification)	0.9	0.8	0.75
C	F graded timber with some verification	0.85	0.7	0.65
D	F graded timber	0.8	0.65	0.55

Note: Consequence of failure Category 3 corresponds to $\beta \approx 3.0$, Category 2 corresponds to $\beta \approx 3.5$-4.0, Category 1 corresponds to $\beta \approx 4.0$ - 4.5

For bridges, secondary components would be classed under Category 2, while all primary members and critical elements would be classed under Category 1. Webs and laminates in SLT decks would be classified under Category 1, noting that the Australian limit states design procedures for plate decks already permit significant increases (up to 50% for sawn timber) in the basic characteristic strength as a result of load sharing distribution effects.

The original calibrations were done for $\beta \cong 3.0$, 3.5, and 4.0, respectively, but due to rounding of the ø factors, a range of β values occurs in the table. The ø factors for connection systems are still being addressed, but it is considered that a value of β=5 to 6 is appropriate for connection systems in bridges. Depending on the timber material in the connection, this will produce values of ø, ranging from 0.75 to 0.4. Combining this approach with the existing limit states provisions for ultimate limit state (ULS) design loads means that the probability of failure during a 100 year design life is about 10^{-4} for Category 2 members and about 10^{-5} for Category 1 members.

The AASHTO LFRD specification is similar and combines a factor which is relevant to the type of load effect, with a table of "base resistance" characteristic strengths for species and grades of timber commonly used in bridge design. For species and grades not included in the LRFD tables, a series of conversion factors is provided to determine values for the base resistance appropriate for bridge design. The nominal resistance is then determined using modification factors in a form similar to that shown in Equation (3).

Defining Failure at the Strength Limit State

An important consideration that also needs to be addressed in determining appropriate levels of reliability for bridge design is the definition of what constitutes failure. For example, in a single lane girder bridge, the failure of one girder as a result of excessive load combined with weak material is likely to be catastrophic for the entire bridge. However, a single girder failure in a wider bridge with 7 or 8 girders may not be quite as disastrous, depending on the location of the girder in the bridge.

For SLT decks constructed as either plates, box, or T-beam sections, the deck system has exceptionally high "post critical" load capacity, even after failure at the strength limit state has occurred. It should be noted that full-scale laboratory testing of SLT deck systems has confirmed that the ultimate load capacity of such decks is generally considerably in excess of the ULS requirements specified in the AASHTO LFRD and AUSTROADS limit states loading codes. Not only are such deck systems remarkably ductile, but they inherently redistribute loads around any localised area of member failure (due to rupture), and behave in a linear elastic manner in the serviceability load range even after failure has occurred in the critical web members of cellular/box decks. Unless prestress has decreased to less than 50% of the initial

level, these deck systems do not "fail" catastrophically in a manner which would be observed in steel or concrete deck systems. T-beam SLT bridges have enhanced load sharing capacity when compared with girder bridges, but still tend to be limited by the capacity of the webs, particularly on the tension side of the beams. Details of ultimate and post critical behaviour of SLT decks systems are presented elsewhere (Crews and Bakoss 1993; Crews and Bakoss 1996).

Defining Load Events

The preceding discussion has focused mainly on the problems associated with defining reliability with respect to the material capacity or resistance side of Equation (l). Considerable research work has also been undertaken with respect to the loading allowances for limit states design codes. Both the AASHTO and AUSTROADS codes specify numerous load events based on the effects of vehicular and environmental loads (predominately due to earthquake, wind, and water) on the bridge structure.

Generally, three vehicular load events are defined for consideration in a limit states code. These are based on "standard" design vehicles, with specified axle loads and geometry. The loads are factored and load effect envelopes for flexure, shear, and reactions are determined. The three vehicular load events normally considered are (1) strength (2) serviceability for a "normal" design truck, and (3) strength for an "abnormal" heavy load design vehicle. For example, AUSTROADS specifies that the strength limit state be determined for the load effects of

- dead load with a ULS load factor of 1.2,
- a T44 vehicle with a ULS load factor of 2.0, and
- a Heavy Load Platform (HLP) vehicle with a ULS load factor of 1.5.

The serviceability limit state (SLS) is determined for load factors of 1.0 for all load events. In all cases for vehicle loads, the dynamic load allowance (DLA) must be applied as detailed in the next section.

Dynamic Load Allowance

In the past, the effects of dynamic loading have usually been ignored for timber bridges. The AASHTO LRFD specification requires that the load effects from the static design vehicle loads be increased by 75% for deck joints and 33% for all other components, while the AUSTROADS code requires a 20 to 40% increase for the T44 vehicle loads (depending on the first natural flexural frequency of the superstructure) and 10% for heavy load platform (HLP) loads. It is generally recognised that timber bridges have excellent inherent damping under dynamic load and because of this fact, both the AASHTO and Australian SLT provisions permit a reduction in the dynamic load allowance, of 50% and 30%, respectively.

Serviceability Limit States

Serviceability design for timber bridges is mainly concerned with deflection limits. Neither the previous U.S. nor Australian allowable stress design codes contain provisions for checking deflection criteria. However, this has now been addressed in the AASHTO LFRD specification and the Australian SLT Code. Deflection limits are generally based on the unfactored vehicle live load, including the DLA. Timber bridges tend to be much more tolerant of deflection than those of concrete or steel and this is reflected in the deflection limits specified in codes. The AASHTO LFRD specification presents a deflection limit of bridge span divided by 425, which is left to the discretion of the designer to apply. The Australian SLT provisions specify span divided by 380. This latter limit is applicable only to SLT decks with a flexible bituminous seal wearing course, and a limit of span divided by 450 is more appropriate for other bridge types. These limits are based mainly on the need to avoid rotation at the supports, and more stringent limits may be appropriate if a rigid asphaltic concrete wearing course is used on the deck to avoid cracking over the supports.

Concluding Remarks

The AASHTO LRFD specifications provide structural engineers with a suitable code for limit states design of timber bridges. The reliability approach incorporated in the forthcoming limit states timber structures code (AS 1720.1-1996) in Australia will provide the technical basis for development of a specific timber bridge design code. Preliminary work on an Australian code for timber bridges has commenced for Standards Australia, which has now taken over the codification role of AUSTROADS. The current SLT code is being revised in the light of work currently being done by the authors, and it is envisaged that this updated design specification will be incorporated into this general timber bridge code. The end result is that engineers in both the United States and Australia can have the same level of confidence in the design and reliability of timber bridges as they currently enjoy for similar structures designed in concrete and steel.

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Design of Timber Foundation Piling for Highway Bridges and Other Structures

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Abstract

Presented information on structures supported by treated timber piling, including details of the East Side (FDR) Drive in New York City. Determined the average and mean Factor of Safety of the Engineering News Formula for 43 site-specific cases. Showed examples of present-day design for timber piling supported by friction, end bearing, or a combination of both. Considered practical and theoretical geotechnical design criteria. Furnished timber piling and preservative specifications. Reviewed durability for foundation, fresh water, and marine piling. Addressed environmental considerations. Concluded that local Geotechnical engineers apply empirical experience to design treated round timber foundation piling.

Keywords: design, durability, Engineering News Formula, environmental, factor of safety, preservative, specifications, timber piling.

Introduction

Starting in 1865, treated round timber piles supplied most of the deep foundation support for highway bridges and other structures in the United States. The first treated piles installed were for the Taunton River Bridge, Taunton, Massachusetts. The U.S. Army Corps of Engineers alone used over six million timber piles to construct the Locks and Dams for the Inland Waterway System.

The purpose of this paper is to introduce highway engineers to the subject, Design of Timber Foundation Piling for Highway Bridges and Other Structures.

East River Drive, New York, NY

An example of the extensive use of treated timber piling in the past was Manhattan's 11,265.41 meter (7 mile) long East River Drive in New York City, constructed between circa 1935 and May 25, 1942.

Figure 1—Timber Pile Relieving Platform, East River Drive, New York, NY

According to Ralph Mann, (Mann, 1942) "For 6920.18 meters (4.3 miles) a relieving platform supported on creosoted piles was built on the bulkhead line established by the U.S. Engineer Corps, which is usually from 30.48 to 76.2 m (100 to 250 ft.) out from the former shore line, in water from 6.10 to 9.14 m (20 to 30 ft.) deep." The piles were 355.60 mm (14 inch) diameter, 0.91 m (3 ft.) from the Butt, treated with 256.32 to 288.36 kg/m^3 (16 to 18 pcf) (gage) creosote for the Southern Yellow Pine and 192.24 to 224.28 kg/m^3 (12 to 14 pcf) (gage) for the longer Douglas Firs. Installed in bents was a total of about 77,000 treated timber piles, as shown on Figure No. 1. Spacing of the bents were from 1.22 to 1.684 m(4 to 5-1/2 ft.)apart.

The East River Drive is still very much in heavy use today, some 60 years after original construction. Recent surveys on the 60 year timber piling show about 1.60 mm (1/16 inch) attack by limnoria tripunctata at creosote retention levels less than 112.14 kg/m^3 (7 pcf). (Altiero, 1996). Attack was greater south of 45th street where the East River has higher salinity levels.

Recently rebuilt on dual-treated Class B Southern Pine piles 19.81 m (65 ft.) long is a section near 14th Street. New York State D.O.T. and TAMS Consultants, Inc. decided to use the product again since it lasted 60 years in such a harsh environment as the East River. They improved the expected life by specifying a dual treatment of 16.02 kg/m^3 (1.0 pcf) Chromated Copper Arsenate (CCA) and 320.40kg/m^3 (20 pcf) creosote, in accordance with American Wood Preservers' Association (AWPA) Standard C 3, Piles, Preservative Treatment by Pressure Process. (AWPA, 1995)

Experienced New York City engineers specify relatively lightly loaded timber piles for the docks surrounding Manhattan by driving the piles to a pre-determined depth, not to a bearing capacity formula. The river muds regain their strength after driving, that is, the pore-water pressure dissipates. But for the East River Drive, the heavy relieving platform required installing timber piling through, "3.05 to 6.10 m (10 to 20 ft.) of soft mud which overlies an equal thickness of more stable material that develops hard driving where the piles "take-up" either in sand or on rock." (Mann, 1942)

Engineering News Formula

Nearly every timber piling job specifies the Engineering News (EN) Formula for driving or capacity criteria (1). Wellington developed the formula in 1893 for timber piling driven with a drop hammer, as published in Engineering News magazine. (Chellis, 1961) However, no publications since then address the actual safety factor using the EN Formula.

Engineering News Formula

$$\text{Design Capacity, } N(\text{lbs}) = \frac{2\ WH}{S + 0.1} \qquad (1)$$

Where: W = weight of the hammer, N(lbs)
H = height of drop, m(ft)
S = final set of pile, mm(inch) per blow

Always present the EN Formula as a bearing graph for site-specific hammer and capacity conditions. A good rule-of-thumb for the commonly used Vulcan No. 1, 20,325 Nm (15,000 ft. lb.) hammer, is design capacity (tons) equals hammer strokes per 0.3 m (ft), as shown on the bearing graph on Figure 2. Some common rules for installing timber piles are as follows:

Figure 2—EN Formula Bearing Graph for 20,325 Nm (15,000 ft. lb.) hammer

1. Drive a timber pile no more than 60 strokes per 0.3 m (ft.) [5 strokes per 25.4 mm (inch)] with a 20,325 Nm (15,000 ft. lbs) hammer.

2. The normal capacity of a timber foundation pile is 266.9 kN (30 ton). Occasionally engineers specify 355.8 kN (40 ton) capacity where soil conditions permit. Beach front house piles often use only 89.0 kN (10 ton) capacity.

Research indicates 43 examples where driving resistance correlates load tests to failure to determine the actual safety factor of the Engineering News Formula. This research, presented in Table No. 1, includes job sites, soil conditions, pile sizes and lengths, hammer types, driving records, and load test capacities at failure.

The theoretical factor of safety of the Engineering News Formula is 6.0, but analysis of these 43 sites indicates the average factor of safety is about 4.0 and the median about 3.4. The lowest value, 1.6, was for a highway bridge over Charlemont Creek in Portage County, Ohio, constructed in 1942 during WW II.

Recent Examples of Timber Foundation Piling

Following are three recent examples of treated timber foundation piling: J.F. Kennedy International Airport, New York, NY (friction piles in sand); Comfort Inn, Exton, PA (end-bearing piles on rock); and Atlantic City High School, Atlantic City, NJ (friction and end-bearing):

J.F. Kennedy International Airport, New York, NY (friction piles in sand)

A portion of Jamaica Bay Marshlands, located on the southern shore of Long Island, was filled in the early 1940's with hydraulically transported sand, to elevation 3.66 m (12 ft.) MSL to construct J.F. Kennedy International Airport (formerly Idlewild). The 2.44 to 4.88 m (8 to 16 ft.) thick sand fill was placed over organic deposits, 0.61 to 3.35 m (2 to 11 ft.) thick, which were over a 30.48 to 38.10 m (100 to 125 ft.) natural sand deposit, that is, glacial outwash upper Pleistocene deposits of the Wisconsin Age, as shown on Figure 3. (Graham, 1990)

To minimize differential settlements for a 1990 Cargo Terminal building at Kennedy Airport in New York, some 1000 ASTM D-25, Class B, Southern Pine, 12.19 m (40 feet) long Timber Foundation piles were pressure-treated with creosote per AWPA C 3 specifications to a retention of 411.41 kg/m^3 (12 pcf) in the outer 50.80 mm (2 inches) and installed as the deep foundation. The timber piles were supplied by NTPC member companies in accordance with the New York City Building Code, i.e., ASTM D-25, 8274 kPa (1200 psi) allowable design stress, minimum 203.20 mm (8 inch) tip uniform taper, for 266.88 kN (30 ton) capacity friction piles.

The pile driving contractor used Vulcan No. 1 hammers with a rated energy of 20,325 Nm (15,000 ft. lbs.), and drove the timber piles to a resistance of 35 blows per 0.30m (foot) to allow extra capacity for dragdown, etc. The piles were installed into 254.00 to 304.80 mm (10 to 12 inch) diameter, 3.05 m (10 foot) deep, pre-augured holes. Lima 703 and 803 cranes were used, with fixed leads. No expensive load tests were required for the 266.88 kN (30 ton) capacity piles per the New York City Building Code. (At the present time, almost 8000 treated Class B's with 203.20 mm (8 inch) diameter tip timber piles are being installed for the new Terminal One project at Kennedy.)

Figure 3—Generalized Soil Profile J.F. Kennedy International Airport

Comfort Inn, Exton,PA (end-bearing on rock)

In 1983, to support a new Comfort Inn in Exton, PA, a 355.84 kN (40 ton) capacity timber pile foundation was installed to end-bearing on rock. (Graham, 1989) Test borings at the site indicated the top of rock was about 6.10 m (20 feet) beneath the ground surface.

Overlaying the rock was 1.52 m (5 feet) of coarse, 47 blows per 0.3 m(f) sand and a 1.52 m (5 foot) layer of 22 to 33 blows per 0.3 m(f) fine sand, over which lay a 3.05 m (10 foot) mixture of 8 to 14 blows per 0.3 m(f) sand, silt and clay. Conditions were considered ideal for driving timber piles, especially since the overburden materials contained no boulders or other obstructions. Almost 120 timber piles, 6.10 to 7.32 m (20 to 24 ft) long, ASTM Class B, 12-3-8 were pressure-treated with creosote to 192.24 kg/m^3 (12 pcf) per AWPA Standard C 3. The contractor drove the timber piles with an MKT 9B3 double-acting air hammer. The 9B3 is rated at 11,856.25 Nm (8,750 ft. lbs.) of energy per blow. A Manitowoc 2900 truck crane was used to handle 14.63 m (48 foot) leads, the hammer and pile driving. An Ingersoll-Rand, Corp Mem, Roanoke, VA, 1050 air compressor was used to power the MKT 9B3 hammer.

Atlantic City High School, Atlantic City, NJ (friction and end-bearing)

The Great Island site of the Atlantic City High School is an island in Lakes Bay, located between Atlantic City and the mainland. (Graham, 1993) According to the Soils & Foundation Report, "... Atlantic City is underlain by stratified granular soils of alluvial origin, possibly interspersed with recent organic alluvium, overlying sands and gravels of the Cape May Formation, as shown on Table No. 4. Bedrock exists at very substantial depths, many hundreds of meters (feet)...", as shown on Table No. 4.

Approximately 3000 Southern Pine timber piles, 15.24 m (50 ft.) long, for 311.36 kN (35 ton) capacity, ASTM D-25 and sized to a minimum tip and butt diameters of 203.20 and 304.80 mm (8 and 12 inches) (12-3-8), respectively. Treatment was specified to be creosote in accordance with AWPA C 3 specifications. The butt cut-offs were coated with two coats of hot creosote, followed by one coat of coal tar pitch, allowing sufficient time between applications permitting absorption of each coat, as described in AWPA M 4.

Using a Manitowoc 2900 Crane, the contractor installed the 311.36 kN (35 ton) capacity timber piles with a Vulcan No. 1 hammer rated at 20,325 Nm (15,000 ft. lbs) of energy powered by an Ingersol Rand 900 compressor. The piles were driven to 37 blows per last 0.3 m (ft.) in accordance with the Engineering News Formula with the tips penetrating into the very dense basal SAND stratum or very stiff to hard CLAY or clayey SILT. An ICE 1250 auger was used to predrill approximately 9.75 m (32 ft.) below working grade; the lower 6.10 m (20 ft.) of auger was 304.80 mm (12 inch) diameter while the upper portion was 406.40 mm (16 inch) diameter. The auger was mounted on the side of the "fixed leader." An A-frame moon beam controlled the vertical and horizontal alignment of the fixed leader and "spotted" the auger and the piles.

El 6.10 to 7.92 m (20 to 26 ft)
SURCHARGED in 1992

El 3.81 m (12.5 ft) finished floor slab
3.05 m (10 ft)
GWL top of ground El 1.52 to 3.05 m (5 to 10 ft)

0 hydraulic SAND FILL prior to 1978

-3.05 m (-10 ft) very soft organic SILT with layers of PEAT
-6.10 m (-20 ft) loose silty fine SAND trace shells

-9.14 m (-30 ft) compact to very dense basal SAND

-12.19m (-40 ft)

-15.24 m (-50 ft) very stiff to hard silty CLAY or clayey SILT
-18.29 m (-60 ft)
compact to very dense basal SAND
-21.34 m (-70 ft)

Figure 4—Generalized Geologic Profile Atlantic City High School

Geotechnical Design

Design a round treated timber pile job by first obtaining information on adjacent structures and records of nearby test boring data.

Timber piling primarily are end bearing piles driven into dense sand or hard clay layers with some friction in the upper layers of softer soils. When evaluating the test boring data, look for a bearing strata some 9.14 to 18.29 m (30 to 60 ft.) deep with split spoon hammer blows of 30 per 0.3 m (ft.) or more. Stronger sub-surface soil bearing layers exist in most site-specific projects.

Perform geotechnical analysis, such as given in the Navy's DM 7.2 (NAVFAC, 1982), shown on Figure 5. The FHWA has a very complex manual on Design and Construction of Driven Pile Foundations (FHWA, 1985) and a Soils and Foundations Workshop Manual. (FHWA, 1982) Also, the Corps Design of Pile Foundations has been made available by ASCE. (ASCE, 1993) However these government manuals should be used only as a guide. Experience with using timber piling and empirical design are the most often used criteria.

Specifications for Round Timber Piling and Preservatives

Treated Round Timber Piling Specifications were presented in 1992 at the Deep Foundations Institute (DFI) in New Orleans (Graham, 1992), and are summarized below to include timber and preservatives:

Timber- Longleaf, Shortleaf, Loblolly, or Slash species of Southern Pine or Coastal Douglas Fir supplied per ASTM D 25 (ASTM, 1995).
Sizes- Class A, Class B, 203.20 mm (8 inch) tip natural taper, or use the Tables in ASTM D-25.
Treatment- AWPA C 3, Piles; C 14 Highway Construction; C 18 Marine Construction. See Table 3. (AWPA, 1995)
Quality Control- Notarized certificates furnished to the engineer with each delivery.
Field Treatment- AWPA M 4 (AWPA, 1995)
Design Stress- ASTM D-2899, 8274 kPa (1200 psi). (ASTM, 1995)

For complete timber piling installation specifications, please refer to the DFI's Treated Round Wood Piling Specifications. (Graham, 1992)

Sizes

The most common sizes of Round Timber Piling are Class B, as shown in Table 2, and 203.20 mm (8 inch) diameter tip natural taper, given on Figure 6.

Class B piles were first proposed in circa 1915, then formally adopted by ASTM D 25 in 1937 (ASTM, 1937). About 50% of timber piles supplied today are Class B.

The 203.20 mm (8 inch) diameter tip natural piles are called the New York City pile, because they are specified in the Building Code. Often used up to 12.19 to 13.72 m (40 to 45 ft) long, they are the most economical 266.88 kN (30 ton) capacity pile.

$$Qult = P_T N_q A_T + \sum_{H=H_O}^{H=H_O+D} (K_{HC})(P_O)(TAN\ \delta)(S)$$

a)GRANULAR SOIL

$$Qult = C(N_{CS})\pi R^2 + C_A\, 2\pi RZ$$

b) COHESIVE SOIL

Figure 5—Ultimate Load Capacity Equations for (a) Granular, and (b) Cohesive Soil (NAVFAC, 1982)

Table 2— Class B sizes per ASTM D 25-37 to 58

	Class B					
	0.91 m (3 ft.) from Butt				At Tip, min.	
Length, m (ft)	Min.		Max.			
	C*	D**	C	D	C	D
Under 12.2 Under (40)	965 (38)	305 (12)	1600 (63)	508 (20)	635 (25)	203 (8)
12.2 to 16.5 (40 to 54)	965 (38)	305 (12)	1600 (63)	508 (20)	559 (22)	178 (7)
16.8 to 22.6 (55 to 74)	1041 (41)	330 (13)	1600 (63)	508 (20)	559 (22)	178 (7)
22.9 to 27.4 (75 to 90)	1041 (41)	330 (13)	1600 (63)	508 (20)	483 (19)	152 (6)
Over 27.4 Over (90)	1041 (41)	330 (13)	1600 (63)	508 (20)	406 (16)	127 (5)

* C = Circumference ** D = Diameter

203.2 mm (8 Inch) Diameter Tip
Natural Taper per ASTM D 25-91

Figure 6—Natural Taper Round Timber Piling

Durability

Round timber foundation piling installed in the ground below the ground water table will last for centuries. Examples are the Companile Tower in Venice, Italy, built in 900, and an Erie Canal structure in Waterford, NY, where 80 year old untreated timber piles, extracted in February, 1996, are as good as new.

Treatment is required for Round Timber Piling installed in the ground, where a portion is above the water table. Properly treated and installed in a concrete pile cap, the piles will probably last for centuries, according to an industry study conducted until 1955. The American Wood Preservers Association (AWPA) regularly surveyed some 40 pressure-treated timber foundation pile installations throughout the country. Several of the structures were installed as early as 1922. Included in the AWPA study were grain elevators, public schools, breweries, apartment buildings, veteran's hospitals, viaducts, generating plants, bridges, oil refineries, plants, theatres, post offices, court houses, and train stations. The piles were cut-off well above the ground water level. The study was terminated because there was no reason to continue, there had been no failures.

Installed in the ground, but extending up into the air, such as at a beach house, timber piles will last as long as utility poles, about 75 years in the North and 50 in the South. Decay will occur about 0.61 m (2 ft.) below the ground surface.

Table 3—AWPA Specified Preservative Retention

	Round Southern Pine Piling	
	Creosote kg/m^3 (pcf)	CCA kg/m^3 (pcf)
Foundation	192.2	12.8
	(12.0)	(0.8)
Land & Fresh Water	192.2	12.8
	(12.0)	(.08)
Marine*	320.4/256.3	40.1/24.0
	(20.0/16.0)	(2.5/1.5)
Dual Treatment	320.4	16.0
	(20.0)	(1.0)

- 256.3 kg/m^3 (16.0 pcf) creosote or 24.0 kg/m^3 (1.5 pcf) CCA for marine use, from New Jersey and northward on the East coast of the United States.
- In those areas where Teredo and pholad attack are expected or known and where Limnoria tripunctata attack is not prevalent, creosote or creosote solution treatment will provide adequate protection.
- In those areas where Teredo and Limnoria tripunctata attack is expected or known and where pholad attack is not prevalent, either dual treatment or high retention of ACA, ACZA or CCA treatment will provide adequate protection.
- In those areas where Limnoria tripunctata and pholad attack is expected or known, the dual treatment provides the maximum protection known at present.
- A map showing areas of marine borer hazards in U.S. waters may be found in Figure 2 at the end of AWPA Standard C 3.

Round marine piling, subject to marine borer attack, will be a function of the salinity, current, and treatment. (Chellis, 1961) "Salinity ranges from 30

to 35 parts of salt per 1000 in the ocean..." The danger point for marine borer attack starts around 15 parts per thousand. Current is also a factor. Toredo will not attack above 0.72 m/s (1.4 knots) and limnoria over 0.93 m/s (1.8 knots). Creosote marine treatment lasts for about 50 years in the North and in the South about the same number of years as pounds per cubic foot of treatment. The expected life of round CCA treated piles is still unknown, but, with few exceptions, the results look excellent. One exception is the severe attack by Sphaeroma in Tarpon Springs, FL. (Johnson, 1987) Marine treatment of CCA causes brittleness, and Southern Pine treated to 40.05 kg/m^3 (2.5 pcf) should not be used for fender piling. (AFPA, 1991)

Environmental Considerations

According to the Environmental Protection Agency, round timber piling, properly treated with creosote or CCA, or dual treated with both, has no harmful effect on the environment. (Webb, 1988) Creosote is biodegradable and CCA, when fixed to the wood, does not leach out. (Graham, 1991)

National Geographic (Grall, 1992) published an article titled, Pillar of Life, showing how marine life attaches to timber piling to help the environment. The following is a quote from the Grail article:

"When a piling is driven into the bottom of the bay, life takes up residence almost immediately. Bacteria, algae, and protozoans cover the submerged surface. This "slime" provides a foothold for larger creatures to attach themselves in succession. In the summer young ivory barnacles are among the first to appear. With them come sun sponges and mosslike bryozoans, which spread over the piling by budding. Bright patches of algae such as sea lettuce soon arrive, followed by hydroids and bulbous sea squirts. Mussels use byssus threads to anchor themselves. Tubebuilder amphipods construct tunnels of mud and detritus for protection and for a niche on the crowded piling. Still others affix themselves to the shells of animals already attached. Almost every underwater part of the piling is covered with sessile species, each looking for food, shelter, and a place to propagate."

Conclusions

Treated round timber piling have been successful for the past 130 years in the United States as deep foundations for highway bridges and other structures. Based on the Engineering News formula driving criteria, timber piles were installed to a safety factor averaging 4.0, with a mean of 3.5. Design has been based on the local Geotechnical Engineers knowledge and experience of site-specific soil and rock conditions.

Engineers refer to ASTM D 25 for timber and AWPA C 3 for preservative specifications. Piling sizes are usually Class B or 203.2 mm (8 inch) diameter tip natural taper. Durability of treated timber piles has been determined for foundation, land and fresh water, and marine conditions. Creosote and CCA were approved for use by the Environmental Protection Agency in 1986. Creosote is biodegradable and CCA, when fixed to the wood, does not leach. National Geographic published a paper showing how treated round timber piles help the marine environment.

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Table 1—Example Projects

LOCATION	PROJECT	SOIL CONDITIONS	REFERENCE	DIAMETER mm (in.)		LENGTH m (ft.)	HAMMER	EN FORMULA kN (Ton)	LOAD TEST kN (Ton)	FACTOR OF SAFETY
				butt	tip					
New Orleans, LA (1954)	Bridge over Inner Harbor	Sandy Silty CLAY	Peck 1961	393.7 (15.5)	190.5 (7.5)	18.8976 (62)	Vulcan No. 2	88.96 (10)	400.32 (45)	4.5
SAME				381 (15)	177.8 (7)	18.5928 (61)	Vulcan No. 2	133.44 (15)	489.28 (55)	3.7
SAME				355.6 (14)	203.2 (8)	18.288 (60)	Vulcan No. 2	88.96 (10)	373.632 (42)	4.2
Bonner County, ID (1952)	Sandpoint Bridge	CLAY	Peck 1961		152.4 (6)	21.9456 (72)	Vulcan No. 2	62.272 (7)	266.88 (30)	4.3
Council Bluffs, IA	Broadway Viaduct	CLAY clayey SAND SAND	Peck 1961	304.8 (12)		5.7912 (19)	Drop Hammer	88.96 (10)	355.84 (40)	4.0
Portage County, OH (1942)	Bridge Over Ravonna Ordinance Plant	silty CLAY	Peck 1961	381 (15)	177.8 (7)	13.1064 (43)	Vulcan No. 2	195.712 (22)	382.528 (43)	2.0
Lorain County, OH (1943)	Bridge over Charlemont Creek	silty CLAY	Peck 1961	322.58 (12.7)	185.42 (7.3)		Vulcan No. 2	257.984 (29)	400.32 (45)	1.6
New Orleans, LA (1956)	Greater New Orleans Bridge	silty CLAY	Peck 1961	406.4 (16)	152.4 (6)	18.288 (60)	Vulcan No. 1	35.584 (4)	222.4 (25)	6.3
SAME				406.4 (16)	152.4 (6)	21.336 (70)	Vulcan No. 1	62.272 (7)	400.32 (45)	6.4
SAME				406.4 (16)	152.4 (6)	18.288 (60)	Vulcan No. 1	26.688 (3)	266.88 (30)	10.0
New Orleans, LA (1954)			Peck 1961	406.4 (16)	152.4 (6)	24.384 (80)	Vulcan No. 1	533.76 (60)	711.68 (80)	13.3

Table 1—Continued

LOCATION	PROJECT	SOIL CONDITIONS	REFERENCE	DIAMETER mm (in.)		LENGTH m (ft.)	HAMMER	EN FORMULA kN (Ton)	LOAD TEST kN (Ton)	FACTOR OF SAFETY
				butt	tip					
Chicago, IL (1960)		HARDPAN (end-bearing)	McClurg 1960	355.6 (14)	228.6 (9)	12.8016 (42)	Vulcan No. 1	418.112 (47)	1085.312 (122)	2.6
SAME				304.8 (12)	177.8 (7)	12.8016 (42)	Vulcan No. 1	400.32 (45)	1343.296 (151)	3.3
SAME				355.6 (14)	228.6 (9)	12.8016 (42)	Vulcan No. 1	373.632 (42)	1263.232 (142)	3.4
SAME				355.6 (14)	177.8 (9)	12.8016 (42)	Vulcan No. 1	489.28 (55)	2090.56 (235)	4.3
SAME				304.8 (12)	177.8 (7)	12.8016 (42)	Vulcan No. 1	462.592 (52)	1085.312 (122)	2.3
Norfolk, VA	Waterfront	CLAY	Chellis 1961	342.9 (13.5)	203.2 (8)	17.6784 (58)	Vulcan No. 1	115.648 (13)	391.424 (44)	3.4
SAME				431.8 (17)	177.8 (7)	20.4216 (67)	Vulcan No. 1	133.44 (15)	355.84 (40)	2.7
Pike County, OH	PeePee Creek Bridge	SAND & GRAVEL	Chellis 1961	355.6 (14)	177.8 (7)	12.8016 (42)	Vulcan No. 1	231.296 (26)	631.66 (71)	2.7
Monroe County, OH	Kokasing River Bridge	SAND & GRAVEL	Chellis 1961	304.8 (12)	177.8 (7)	8.8392 (29)	Vulcan No. 2	151.232 (17)	329.152 (37)	2.2
Pike County, OH	Crooked Creek Bridge	SAND & GRAVEL	Chellis 1961			7.62 (25)	Vulcan No. 2	160.128 (18)	400.32 (45)	2.5
Burnside, LA (1956)	Hwy Research Board, Spec. Report 36	CLAY	Peck 1958	330.2 (13)	177.8 (7)	12.192 (40)	Vulcan No. 1	97.856 (11)	444.8 (50)	4.5
SAME				381 (15)	203.2 (8)	15.24 (50)	Vulcan No. 1	142.336 (16)	667.2 (75)	4.7
SAME				355.6 (14)	177.8 (7)	15.24 (50)	Vulcan No. 1	142.336 (16)	756.16 (85)	5.3

Table 1—Continued

LOCATION	PROJECT	SOIL CONDITIONS	REFERENCE	DIAMETER mm (in.)		LENGTH m (ft.)	HAMMER	EN FORMULA kN (Ton)	LOAD TEST kN (Ton)	FACTOR OF SAFETY
				butt	tip					
New Orleans, LA (prior to 1958)	S. Broad Ave. Overpass	Sandy Silty CLAY	Peck (1958)	355.6 (14)	177.8 (7)	15.8496 (52)	Vulcan No. 1	186.816 (21)	596.032 (67)	3.1
SAME				355.6 (14)	177.8 (7)	15.8496 (52)	Vulcan No. 1	266.88 (30)	640.512 (72)	2.4
New Orleans, LA (1948)	Elysian Fields Ave. Overpass	Sandy Silty CLAY	Peck (1958)	457.2 (18)	203.2 (8)	17.6784 (58)	MKT 9B3	133.44 (15)	444.8 (50)	3.3
SAME				457.2 (18)	203.2 (8)	16.1544 (53)	MKT 9B3	177.92 (20)	578.24 (65)	3.3
SAME				457.2 (18)	177.8 (7)	16.1544 (53)	MKT 9B3	133.44 (15)	489.28 (55)	3.7
New Orleans, LA	Gentilly Blvd. Overpass	SILT CLAY	Peck (1958)	393.7 (15.5)	184.15 (7.25)	17.9832 (59)	MKT 9B3	195.712 (22)	889.6 (100)	4.5
SAME				393.7 (15.5)	184.15 (7.25)	18.5928 (61)	MKT 9B3	195.712 (22)	711.68 (80)	3.6
SAME				406.4 (16)	203.2 (8)	17.0688 (56)	MKT 9B3	88.96 (10)	622.72 (70)	7.0
Monticello, MN (1971)	Bridge	fine silty SAND	Goble (1971)	312.42 (12.3)	218.44 (8l.6)	7.3152 (24)	Link Belt 440	240.192 (27)	1174.272 (132)	4.8
Winnemuca, NV (1980)	I-80 Viaduct	SAND & GRAVEL	Goble (1980)	355.6 (14)	254 (10)	13.716 (45)	Delmag D-30	658.304 (74)	1245.44 (140)	1.9
Philadelphia PA	Intn'l Airport Baggage Claim B		Earle (not dated)	361.95 (14 .25)	215.9 (8.5)	7.9248 (26)	Vulcan No. 1	444.8 (50)	845.12 (95)	1.9

Table 1—Continued

LOCATION	PROJECT	SOIL CONDITIONS	REFERENCE	DIAMETER mm (in.)		LENGTH m (ft.)	HAMMER	EN FORMULA kN (Ton)	LOAD TEST kN (Ton)	FACTOR OF SAFETY
				butt	tip					
Freeport, TX (1958)	Dow Chemical Plant	CLAY & Silty SAND	AWPI (1958)	330.2 (13)	203.2 (8)	9.7536 (32)	Vulcan No. 1	391.424 (44)	711.68 (80)	1.8
SAME				307.34 (12.1)	210.82 (8.3)	9.144 (30)	Vulcan No. 1	409.216 (46)	800.64 (90)	2.0
Charleston, SC (1962)	Cooper River Bridge	CLAY (Marl)	Pittman (1962)	342.9 (13.5)	203.2 (8)	17.6784 (58)	Vulcan No. 1	160.128 (18)	1049.728 (118)	6.5
Harrison, NJ (1987)	PATH Main Repair Facility	SAND & GRAVEL	PA,NY & NJ (1987)	330.2 (13)	177.8 (7)	14.0208 (46)	Vulcan No. 1	186.816 (21)	711.68 (80)	3.8
Philadelphia PA (1982)	Toys "R" Us 3rd St. & Oregon Ave.	organic SILT over dense SAND	McClymont (1982)	355.6 (14)	203.2 (8)	10.3632 (34)	FEC 1500	231.296 (26)	640.512 (72)	2.8
Mobile, AL (1964)	I-10 @ Dauphin Island Pkwy.	CLAY & SAND over dense SAND (end bearing)	AWPI (1967)	349.25 (13.75)	241.3 (9.5)	17.0688 (56)	Vulcan No. 1	569.344 (64)	1156.48 (130)	2.0
New Orleans, LA (1965)	I-10 interchange near Lafayette-Duson	Sandy Silty CLAY	AWPI (1967)	342.9 (13.5)	203.2 (8)	10.3632 (34)	Vulcan No. 1	177.92 (20)	889.6 (100)	5.0
Port Arthur, TX (1968)	S.H.73 25th St. Overpass	Layer CLAY SAND SILT	Harper (1968)	384.175 (15.125)	222.25 (8.75)		Delmag D-12	569.344 (64)	1467.84 (165)	2.6

Ontario's Experience With Composite Wood/Steel Bridges

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Longitudinally laminated prestressed decks for bridges gained prominence in the late 1970's and have evolved into an effective structural system. It was similarly recognized that combining such a deck with steel girders could offer further advantages for longer span bridges. The concept of a composite wood and steel bridge was first reported upon by the Ontario Ministry of Transportation in 1986. Using shear bulkheads a longitudinally laminated deck was made composite with steel girders. Ontario has now constructed three full scale bridges using this technique. A number of unique details and improvements to the original concept have been developed.

In 1986 MT0 developed and tested a steel-wood composite bridge configuration [1]. The concept consisted of a longitudinally laminated prestressed wood deck made composite with steel girders through the use of shear bulkheads. These bulkheads were created by coring holes through the stressed wood deck directly over top of girders, welding shear studs to the girders and then filling the holes with fibre reinforced cementitious grout. (fig 1).

Having confirmed the functionality of this concept it was desirable to construct full scale applications. Between 1991 and 1994 three steel/wood composite bridges were built in Ontario--North Pagwachuan R., Aubinadong R. and Hoiles Cr.-- each with unique characteristics.

Although the original research dealt effectively with the theory of composite steel/wood design many other design details unique to this structural form had to be developed and a number of significant improvements were made through the course of construction.

Figure 1 The concept of composite steel/wood design

North Pagwachuan River Bridge

Description

The North Pagwachuan River Bridge is located on Highway 11 in Northern Ontario approximately 400 km from the nearest major urban centre, Thunder Bay. The existing bridge had been scheduled for replacement by the Ministry of Transportation (MTO) due to its advanced state of deterioration. A problem encountered in the structural planning was that the fairly remote location of this bridge dictated that an alternative to traditional concrete and steel options be used.

It was decided that this project offered an excellent opportunity to employ the methods developed in the MT0 research [1] in a full scale prototype application for Ontario's first composite wood/steel bridge.

Soil conditions at the site precluded the use of intermediate piers resulting in a fairly long 50 m clear span. The bridge had a two lane cross section 12.4 m wide with 4% superelevation. The deck was 286 mm

thick supported on 5--1800 mm deep girders. (Fig.2). It was constructed on the existing highway alignment necessitating a temporary detour and bridge to carry traffic during construction.

Superstructure Design

MTO's research [1] concluded that it was reasonable to assume that 50% of the deck acted compositely with the girders. This was of course dependant upon a number of factors including girder spacing, deck thickness and prestressing forces but was felt to be a reasonable assumption.

Because the system was a prototype and its long term effectiveness not proven, the superstructure was designed such that the girders acted compositely for serviceability loading only. At ultimate limit state however, the naked girder sections were designed to carry all loads irrespective of the deck. Conversely, as long as composite action existed the superstructure would be considerably over designed. It was recognized therefore that the true economies of the design could not be fully realized.

The bridge was designed using the methodology suggested in the research [l] and complied with the Ontario Highway Bridge Design Code (OHBDC) [2]. It was analyzed using the simplified methods of the code by idealizing the configuration as a two dimensional orthotropic plate. Plate properties were approximated based upon the material properties of the wood.

Figure 2 Cross section, North Pagwachuan River Bridge.

Decking

The deck itself was designed to span in the longitudinal direction supported on transverse diaphragms at 7.25 m spacings. No allowance was made for the support provided along the longitudinal girder lines because of the substantially lower transverse stiffness of the deck.

The material used for the deck laminates was 38 mm x 286 mm Douglas Fir, #2 or better. It originated in Oregon, was milled, fabricated and treated in Montreal, Quebec and shipped to Thunder Bay, Ontario. All sections were precut to lengths of up to 6 m and all stressing holes were predrilled prior to treatment.

It was necessary to minimize to number of butt splices in the deck and also to ensure that stress rod holes lined up after the deck was assembled. Therefore tolerances were built into all dimensions and the entire assembly pattern was laid out in plan form to assist the contractor during construction.

The deck was assembled on pre-erected girders by establishing a start point along the centre girder and attaching laminates in all directions from there. (The size of the bridge precluded pre-assembly.) The intention was to minimize the affects of dimensional tolerances at the edges since the final position of the deck relative to the girders was important. Still the deck width ended up being about 450 mm wider on each side than the theoretical width after it was assembled. This had been anticipated, and in order to further ensure that the deck ended up in the correct final location over the girders, one line of shear bulkheads was installed along the centre girder to act as an anchor prior to stressing (Fig 3). Also temporary support bracket extensions were attached to the permanent brackets along the edges to support the 450 mm overhangs (Fig. 4). All laminates were nailed together with 100 mm air driven nails at 600 mm spacings in order to assist construction.

In specifying a treatment for the wood it was decided that traditional creosote treatments were unacceptable in that creosote exudation which normally occurs would not be tolerated by environmental regulatory agencies. As a result it was decided to employ an experimental severe Reuping process to treat the wood. The objective was to obtain adequate penetration of the preservative but to minimize final retention.

At the time of treatment the process appeared to have had the desired effect [3]. However a delay in awarding the contract by one year necessitated storing the wood for one winter outside. The intense freezing weather had the effect of causing creosote to in fact exude the following year, when it was needed for construction. This made handling extremely difficult and uncomfortable in warm weather, thereby necessitating night time assembly of the deck.

Figure 4 Overhang support brackets (after stressing).

A potential problem that became evident after the deck was assembled was that the longitudinal dimensional tolerance of the deck resulted in gaps of about 6 mm between all laminate butt splices. This was a concern because salt water would eventually percolate through the asphalt surface and flow between these gaps and onto the unprotected steel girders. A “quick fix” and effective solution was to have the contractor fabricate wood wedges out of the scrap material which could be drive into the gaps in order to plug them.

Shear Bulkheads

As had been anticipated the task of field core drilling 470--250 mm diameter holes for the shear bulkheads turned out to be arduous. A specially fabricated carbide tooth core bit was used but was found to generate considerable heat and slowed the process. The contractor found that the drilling time could be lessened and the bit life extended by drilling a serious of 25 mm diameter holes around the circumference of the bulkhead.

After coring The wood in the holes was subsequently treated with sprayed applications of creosote. While helpful, the long term effectiveness of this approach is uncertain. To further guard against ingress of water into these bulkheads a rubber membrane was bonded to the deck surface along each line of bulkheads.

Core drilling of the holes through the laminated deck had to take place after the first stage stressing (except along the centre girder) in order to ensure that they were located directly above the girders. Since there was already considerable transverse compressive stress in the deck

prior to drilling of the holes the stress increase which resulted when material was removed at the holes caused some distortion and local crushing as well as unknown prestress loss.

After completing this process if became evident that the concept of field drilling the shear bulkhead holes was an area where considerable improvement could be made.

Deck Stressing

The OHBDC specifies demanding stressing cycles in order to mitigate prestress loss. This was however felt to be overly restrictive and costly for this project in that the stressing would have extended over a period of at least 5 weeks prior to which the final shear bulkheads could not be fabricated.

This requirement exists in order to mitigate prestress losses which occur shortly after initial stressing. These losses in turn decrease interlaminate shear in the deck which affects deck strength. Prestress losses are dependant upon a number of factors including wood creep related to the wood species and moisture content [4]. Notwithstanding, the sequence was felt to pose an unfair economic penalty on this system and would have to be relaxed were it to be developed into a competitive configuration. It was decided to alter the sequence such that only two stressing cycles were carried out--the initial (including a restress after 12 hours) and a final stress just ten days after the initial stressing. To counter the potential for prestress loss the initial force specified was higher than required and the wood was supplied with a very low moisture content. It was also recognized that should this approach not be effective the rods could be restressed in the future if necessary.

The presence of rigidly connected steel girders did complicate the potential restressability of the deck. Because the deck is rigidly fixed to the girders any compression of the deck due to restressing would be resisted by the diaphragms between the girders. Therefore to facilitate the level of movement necessary the diaphragm-to-girder bolted connections were made with slotted holes. In the event of the need to re-stress, the bolts could be loosened first.

A problem often encountered in stressed wood decks is the tendency of the deck to warp when stressed, due to the high transverse compression in the deck. This can be resisted by anchoring the deck to its support system. This potential problem was anticipated and anchors consisting of small clip angles brackets were included in the design. However, because of the higher than normal prestress forces, the problem turned out to be more severe than anticipated. Also, because the deck moved during stressing some hold downs rotated and therefor could not engage the diaphragm flanges properly. Some quick fix anchor designs had to be developed to overcome the problem. As well, at the approach ramps, where anchoring was difficult there was an even greater tendency to warp. The problem was eventually overcome by forcing the deck back down with a heavy weight and then securing it in place.

Railing System

Developing a railing system substantial enough to satisfy OHBDC [2] requirements proved challenging because of the relatively weak transverse strength of the deck. The code required the railing to sustain a static point load of 80 kn (performance level (PL) specifications did not come into effect in Ontario until later). It was again desirable to avoid the use of concrete. A wood railing was considered but rejected as a potential maintenance concern viz snow plows. A galvanized steel box beam railing was finally selected. It was mounted to steel W section posts which were fastened to the deck with a somewhat sophisticated arrangement of timbers, bolts and shear connectors modelled after a PL-1 wood railing connection [5]. A new innovation was the use of Ekki, a tropical hardwood, to both act as a snow plough “rub rail” curb and also anchor the railing posts and distribute forces throughout the deck.

The combination of potential out of plane transverse bending in the deck and the relative weak transverse strength necessitated special cantilever brackets on the outsides of the girders to support the 1.2 m deck overhang. (Fig. 4)

Substructure and Approaches

For the same reasons that concrete was being avoided in the superstructure it was also desirable to keep it out of the substructure. While the use of a crib type of substructure would have been preferred , very unstable soil necessitated the use of pile foundations. A fairly simple configuration of steel piles and timber lagging was developed (fig. 5).

Figure 5 H-piles and timber lagging substructure.

The foundation piles were designed to function as a rigid A-frame which meant that thermal movement of the superstructure had to be accommodated. The girders were placed on elastomeric bearing pads and the wood deck was extended 6.8 m beyond the abutments to function as an approach ramp. Simple rubberized asphalt cycle control joints were placed at the end of the deck.

Aubinadong River Bridge

The Aubinadong River Bridge is located about 300 km north of Sault Ste Marie, Ontario and like North Pagwachuan is not close to a reliable source of concrete. The new bridge was located some 500 m downstream of the existing bridge at a location that required a clear span of 52 m.

Once again this presented an opportunity to use the composite wood/steel configuration. Having constructed North Pagwachuan the year before, a number of improvements were made to the design.

The basic concept remained unchanged, however significant modifications were made to a number of elements including the shear bulkheads, deck hold down system and stressing requirements.

Wood Deck

Instead of using Douglas Fir it was decided that this system would gain favour if it made use of a local species of wood, namely one that is native to Northern Ontario. The local treatable species with strength characteristics suitable for this kind of deck and in significant dimensions was Jack Pine. To that end a deck was designed using 38 mm x 235 mm laminates. The wood was harvested, fabricated and treated in Northern Ontario and shipped to the site. This time the material was treated in the same year as construction using the same reduced retention method that was employed at North Pagwachuan R. The end product was highly satisfactory in that there were no surface residues.

Shear Bulkheads

Some disadvantages of the drilled shear bulkhead included, concerns over long term durability, delays due to the slow field coring process and crushing of wood due to removal of large amounts of timber. It was felt that if the bulkhead openings were prefabricated then the shape, orientation and size could be designed to mitigate these problems. The experiences of North Pagwachuan showed that the amount of transverse deck movement that occurred due to elastic compression during stressing was very predictable. Of course a far greater amount of movement took place due to closing of spaces between laminates however since this totally disappeared during stressing it turned out to be irrelevant in determining where to locate the bulkheads. In theory therefore the bulkheads could be offset from the girders during deck assembly such that they were located directly over the girders, after stressing.

Since there were no longer any restrictions on bulkhead size and shape they could now be made short and wide in order to accommodate more shear studs while maximizing material between bulkheads. To further increase fabrication efficiency large dimension lumber was used between bulkheads instead of standard width laminations (fig. 6).

The new prefabricated bulkheads were now rectangular in shape measuring 380 mm wide by 250 mm long with up to 500 mm of material between them. There were also 12 shear studs in each bulkhead instead of 5 as at North Pagwachuan.

The new shear bulkhead design performed very well during construction. No crushing of wood occurred and the openings that were offset from the girders slid over into their predicted final positions during stressing. Finally, because the openings and therefore the wood along the bulkhead lines was all prefabricated, it had been creosoted in the plant and its longevity was no longer a concern.

Deck Hold Downs

Deck uplift had been a significant problem at North Pagwachuan and it was clear that more robust deck hold downs were required. The new ones were based on the same principals as had been used at North Pagwachuan however with the ability to grip into the wood in order to prevent twisting under movement. They were still attached to the diaphragms and facilitated lateral deck movement. These new hold downs performed well in that they seated properly and did not rotate during deck

movement.

Serious uplift occurred at North Pagwachuan on the approach ramps where there was no simple way of anchoring the deck down during stressing. Since the uplift that occurred was largely a result of the high prestress forces (no uplift was observed until approaching final prestress force) it then occurred that a simple answer was simply to lower the forces at the bridge ends. This was possible because the approach ramps were actually supported by the granular subgrade underneath and therefore did not require a significant level of prestress.

The only drawback was a visual one in that because the wood did not compress as much there was a visual sweep along the edge of the deck. This essentially became hidden by the curb and guard rail system.

Deck Stressing

The stressing cycle was further modified in that the contractor was encouraged to monitor the stress level in the rods after stage one and undertake stage two once the level had dropped to about 50%. This reduced the 10 day wait period by a couple of days and was felt to not adversely affect the ultimate prestress loss.

It was still necessary to accommodate possible future restressing of the deck. However the use of slotted holes in all diaphragm connections was felt to be too costly and excessive given the minor elastic movements that were expected. It was decided that oversized holes in the diaphragms would allow adequate movement at a much lower cost.

Deck Drains

North Pagwachuan did not have deck drains and because of the cross fall and longitudinal profile this has not been a problem. Aubinadong R was constructed on a horizontal tangent with little vertical slope and as a result it was determined that deck drains were needed. A simple design was used which involve drilling an oversize hole through the deck and grouting in a steel pipe. The one drawback that is seen is the potential for premature decay due to the field drilling. A prefabricated opening would have been more desirable.

Figure 7 Completed Aubinadong River Bridge.

Hoiles Creek Bridge

The Hoiles Creek Bridge is located about 25 km east of the North Pagwachuan R. Bridge on Highway 11 and therefore once again it was desirable to avoid the use of concrete. It was replaced in 1994 at the same time as the Aubinadong R. Br. This structure was also a single span but was considerably smaller than the other two at 30 m. The cross section dimensions were the same as at North Pagwachuan.

Most of the modifications that were incorporated into the Aubinadong R. Br. design were also made to the Hoiles Cr. A major difference was that this site presented an opportunity to stage construct the bridge and determine the effectiveness of this system in avoiding a separate detour. It was estimated that savings of up to $250,000.00 could be realised.

Staged construction for plain stress laminated decks was first carried out by MT0 in 1980 at the Sioux Narrows Bridge [6] as well as a number of times since. The primary difference from traditional stressed deck construction is that a bulkhead is required at the centre of the bridge against which the first half of the bridge is stressed. This bulkhead must then be incorporated into the second half of the structure. Of course the added complication at Hoiles Cr. was that the deck had to be composite with girders after the first stage.

In previous designs a steel channel stressing bulkhead was used at the stage point, necessitating complicated and costly detailing. Ekki timber was felt to offer the same rigidity as the steel channels while allowing for

simpler detailing (fig. 8,9).

Figure 8 Stressing bulkhead details.

Figure 9 First Stage of deck prior to stressing. Note Ekki stressing bulkhead on right and rectangular shear bulkheads in deck.

Summary

Composite steel/wood was proven to be an effective configuration particularly at remote locations. Costs were found to be comparable with steel/concrete construction.

The design must be made more cfficicnt in order to achieve maximum benefit from the composite action and make the system competitive with more traditional bridge configurations. Full composite action at ULS must be accepted.

Changes made in the design of the shear bulkheads by prefabricating rectangular shapes have gone a long way towards improving their longevity and simplifying construction.

The system has proven itself as particularly adaptable to staged construction.

Improvements in stressing systems which minimize prestress loss would be of particular benefit here in that allowance for future re-stressing could be dispensed with.

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The Aubinadong River Bridge was designed by M. M. Dillon Ltd. and the Hoiles Creek Bridge by Ontario North Engineering Inc. again with valuable input from many persons within MTO.

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Durability and detail design - the result of 15 years of systematic improvements

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Abstract

Most weather exposed wooden structures do not fail due to structural errors but due to decay of essential parts of the carrying system. To avoid further substitution by concrete and steel, wooden structures must be built for a service life of 25-50 years. Chemical wood preservation being increasingly criticized, it was necessary to rethink environmentally less harmful possibilities. By following old and proven design principles, a number of essential constructive details were developed and tested. Service life of exposed wooden structures can be comparable to other building materials. With a minimum of maintenance, there is no quick failure mechanism to destroy a covered bridge. As wood reacts quickly to adverse influences, shorter maintenance intervals are recommended for exposed wood structures to keep repairs, if needed, easy and cheap - a feature substitute materials cannot offer. The quintessence of fifteen years of supervision of pilot structures is being presented to enhance the awareness of designers as well as craftsmen for the importance of proper detail design for the durability.

Keywords: bridge, durability, design, detail, exposure, preservation, wood, weathering

Introduction

Exposed wooden structures have a long tradition in all countries where timber has been readily available as construction material. In Europe the oldest exposed wood constructions still in service today date back to the 11th century (1). In Switzerland alone about 250 covered bridges are still in good working condition, and many more would be around, had they not been carried away by floods or burned down for strategic reasons during the innumerous wars of the last 500 years.

Of all the wooden structures that have disappeared with time, only very few failed due to structural errors, but most of them degraded progressively as a consequence of decay by fungi. (Termites are beyond the considerations of this paper.) Nevertheless much engineering effort is still being invested to optimize the structure itself, but only little research work goes into finding better ways to prevent the decay of structural parts.

For the last 15 years a project team of EMPA has concentrated its efforts on extending the service life of weather exposed wood structures. The first pilot structure of this program was built in 1979. For each of the following projects, the experiences gained on the previous ones were systematically included in the planning phase, until approx. 40 structures could be included in the supervision program. Every structure is visited periodically by an experienced specialist, and developments are being recorded during its entire service life.

Failure mechanism of weather exposed wood

The following (known) sequence leads ultimately to the failure of weather exposed wood members.

Cause	Effect
sunshine —>	wood shrinking
internal stress—>	longitudinal cracks
rain —>	water into cracks
swelling —>	cracks get deeper
etc.	etc.
deep cracks —>	moisture stays
moisture stays—>	fungi develop
fungi action —>	internal decay
strong decay —>	loss of strength

The speed of decay depends on exposure and a number of climatic factors. Often damage go unnoticed until major repairs are necessary.

Measures to extend service life

To extend service life, the above chain of events must be interrupted at some point, either by

- *preventing the development of cracks and delaminations:* Protecting the essential members of the structure from heating up, by providing sunshields and possibly a surface treatment of an appropriate color (not too dark), or

- *stopping water from entering the fissures:* Covering the cracked surfaces where rain could enter, or

- *poisoning the fungi:* Applying pressure treatment all the way down to the bottom of the deepest potential cracks. Considering the difficult penetration in the usual construction wood (softwoods), one cannot count on it for larger cross sections.

Consequences for design

The major risk factors for the supporting elements are rain and direct sunshine, but not the changes of ambient moisture.

The most effective weather protection is a good roof which covers the entire structure. Such a roof is costly and reduces the design load, and many bridges were built without. In our climate it is then a matter of 5-15 years until decay makes such structures unsafe for their design load and they must be repaired. Such short service lives are normally not acceptable and have brought wood as bridge material into disrepute for a long time in many parts of the world.

As steel and concrete became cheap and readily available, these became the materials of choice for bridge construction, even for minor crossings where wood would have performed perfectly well. For many decades new wooden bridges were therefore out of discussion, except for temporary crossings or in remote areas with difficult access.

Rediscovering wood for exposed engineering structures

In the aftermath of the first oil crisis of 1973 public awareness for "ecological thinking" and "green ideas" made it attractive for decision makers (politicians) to resort to "natural materials" for all kinds of applications - and wood took much benefit of this trend.

In Europe, Switzerland has been on the forefront of this development and started to combine wood with the new technologies available, in order to obtain performing engineering structures. Such new technologies were f.ex.:

Moisture resistant (resorcinol) glues

However, it took only a few years to discover that moisture resistant glueing did not mean weather resistant structures. The wood was still subject to decay and depending how good or poor the detail design was, it took from 2-8 years for structural damages to become apparent.

This spoke once more against wood, unless decay could be prevented effectively.

Pressure treatment of sawmill stock - the weatherproof glulam

In a next step the phenolic glues were used to assemble pressure treated boards. The main soft-wood in Switzerland being spruce, the penetration was not sufficient, unless it was improved by incising the timber before treatment - a process quite common in North America.

Earlier attempts to assemble creosote treated boards had been successful (2), but nowadays the only accepted treatments were waterborne CCF- and CCB-salts. (For environmental reasons Switzer-land had banned CCA and PCB already much earlier.) A number of manufacturing problems still had to be resolved, such as glue application rates, temperature, pressing time, wood MC etc., but eventually a "*weatherproof glulam*" was being manufactured, which opened again competitive chances for wood as weather exposed structural material.

Now the durability of the wood was given by the chemical preservation of the entire cross section, and design became occasionally negligent again. In some cases a few years of exposure were enough for first symptoms of decay to appear. Obviously preservative concentrations were not always sufficient to prevent the growth of wood destroying fungi.

The practical conclusion was that chemical preservation in such an application could only *postpone* the decay of sawn timber for quite some time, *but not prevent it for sure.*

Environmental considerations

For proper gluelines the boards have to be planed shortly before glueing. On pressure treated boards, this additional manufacturing step removes the best treated layers. As environmental concerns grew further, the industrial production of these weatherproof glulams was increasingly handicapped by the growing pile of chemically soaked shavings to be disposed off.

Today, in view of the other pressures against chemical treatments, the use of these weatherproof glulams must be restricted to special cases where no other method of preservation will suffice - or their manufacturing will be banned by regulations.

It is still our conviction that *chemical preservation must be maintained* as a back-up or auxiliary measure at critical spots and where no other means of preservation will work, but it *should never be a substitute for poor design.*

Detail design - an essential feature of durability

Wood constructions have the great advantage that practically any element can be repaired or replaced - a feature unthinkable of concrete structures. However, such an exchange may be a demanding task. Therefore avoiding a replacement by protecting major structural parts from adverse weather conditions is by far preferable. This is what proper detail design is all about.

The principle of preservation by design is quite simple - and has been known for long: *Keep water away from structural wood members.*

Traditionally the result was a large roof. On the other hand wood may get temporarily wet without taking harm. Therefore covering only the spots where water gets trapped and cannot dry, i.e joints, cracks, endgrain wood etc., should work as well.

The design of construction details to achieve this is an essential part of the planning work. For a articulate designer with a good material knowledge much of it is simple common sense. (Unfortunately) there is nothing spectacular to it, but the return on the investment is excellent.

Conception of structures

Bridges, where the deck is placed above and is protecting the supporting structure from weathering, have demonstrated a better state of preservation than those where the deck was between or below the carrying structure.

The conception of a bridge depends on many factors and is often given by topography, required waterway clearance, load, appearance etc. The decision is taken already at a very early planning stage, but can have - other factors being equal - a decisive influence on the long-term behaviour of the structure. Ideally the advice of a wood expert should be already included at that early stage.

The less the structure protects itself, the more effort must be invested in protecting the individual endangered parts. Much of this can be done at the drawing board, assuming the engineer is responsive to the needs and limits of the construction material - and keeps in mind that full sun exposure is at least as dangerous as direct rain.

Execution of structures

Other detail measures cannot be drawn up on a blueprint, but are simply part of good craftmanship. Accordingly, they must be worked out, in the workshop or on site, by craftsmen knowing their working material and how to handle it.

Wood may get wet, as long as it can dry afterwards. The the so-called “intelligent details” must

- provide protection from rain and sun,
- channel liquid water away quickly
- allow wet areas to dry out more easily

As trivial as this may sound, it is not always easy to achieve these features in practice and they must usually be custom-fitted to each new project.

Future possibilities

The development task goes. Much work is going into combining wooden carrying structures with concrete bridge decks, or taking advantage of high strength fiber reinforcements, etc.

Regarding chemical treatment, the efforts are geared at using *lesser quantities of chemicals,* by applying the preservative only where needed. This demands new application techniques, which are still in the experimental stage. Another pathway is being

followed using *less toxic chemicals,* like boron, and finding ways to cope with their disadvantages.

Unfortunately, wood is always a few steps behind concrete and steel in taking advantage of new technical possibilities!

Proven effective construction details

It was a learning by trial and error. Fifteen years of periodic supervision and improvements on approximately fourty pilot structures have shown which details are most critical and how they could be protected without using chemicals. The quintessence of this work is the following set of recommendations. It applies to our moist, temperate climate, but will be helpful elsewhere as well.

Exposed horizontal surfaces - All horizontal (or slightly slanting) surfaces of structural elements should be protected from weathering, or they will invariably develop cracks where water can accumulate. Such surfaces must be protected with sheet metal covers or replaceable wood boards, which are properly attached, see fig. 1.

① air space
② spacer rods
③ weather protected attachment
④ dripping groove

Fig.1: Covering horizontal top faces

Covers and their attachment - In order not to puncture the top faces of wooden elements with screwholes, the fastening of planks and boards for bridge decks, handrails etc., (see fig.1).

- should always be from the protected side
- must permit unimpaired deformations
- provide an air gap

For a simple umbrella function, sheet metal or flexible rooting material can be very effective, but only until it is damaged or punctured. Properly installed wooden covers of sufficient width are just as effective, usually much cheaper, and less subject to vandalism.

Wooden covers will last 10-20 years - and with pressure treatment even longer. Replacement, if needed, is easy and cheap.

Horizontal wood interfaces - A waterproof barrier of tar paper or sheet metal must be installed to prevent capillary penetration of water into the lower members, Be careful not to puncture the separation layer, see fig.2.

① sheet metal or tar paper (approx. 3 cm oversize on either side)
② weather protected attachment

Fig. 2: Waterproof separation between horizontal contact surfaces

Vertical Ventilation slots and Drainage holes - To be effective, they must be at least approx. 2cm (3/4 inch) wide - or they will plug up fast, particularly in forested areas, see fig.3.

① minimum width 2 cm (~1")
② recommended width 1cm (~ 1/2")
③ self cleaning slots
④ gutter piece (metal)

Fig.3: Quick draining and easy cleaning

Dripping grooves - They are very effective on the bottom faces of rain exposed elements, such as large glulam trusses, but also handrails and cover boards, etc., see fig. 1 and 4.

Exposed vertical surfaces - of not replaceable elements, e.g. main faces of glulam trusses, should have at least a water repellent surface treatment or, for longer lasting, a protective cladding, particularly on the sunny side.

Tightening of screws and bolts- Quite often the lumber is used in a freshly sawn state and will shrink in service; glulams one the other hand are delivered dry (l0-12 % MC) and will normally swell up to a service MC of up to 20 %.

Therefore, the fastening bolts of glulam elements should only be *tightened lightly* at the time of erection, or the swelling will cause crushings of the wood surfaces, which become capillary entry ways for water into the wood member.

Vertical wood interfaces - As a rule, unavoidable contact areas should always be kept as small as possible to limit capillary penetration and encourage a rapid drying. To minimize the wood to wood contact, separate the surfaces with metal (or plastic) washers, see fig.4.

① spacer (thick washer)
② weather protected attachment
③ slanted cut

Fig. 4: Minimizing vertical contact

Footings/Abutements - see fig.5. Provide plenty of space around them. Prevent accumulation of dirt and leaves.
Wood pieces in ground contact and in the splashing zone should be decay resistent or pressure treated, or made of galvanized steel. Minimize their size and number.

Endgrain wood - of lumber and particularly of glulam must be protected from rain and mainly sunshine to prevent fissuring, see fig.5. Similarly exposed pile tops should be covered as well.

1 spacer
2 cover board
3 air space
4 weather protected attachment
5 tar paper
6 treated, or decay resistent wood

Fig.5 Endgrain protection and minimizing ground contact

Implementation

To include these performing details in a project, some additional efforts are required during planning and execution. The major problem is to develop an awareness for the decisive importance of these features. Their implementation is often part of the finishing work and should be the trade mark of a well built project.

The additional costs involved are *negligible,* and the return translates into years of extension of service life.

Surface treatments

Adequate surface treatments are an excellent supplemental way to protect exposed wood effectively. One purpose of a coating is to retard moisture exchange between the wood and the ambient surroundings. By dampening the natural moisture movements in the wood, the opening of cracks where liquid water can accumulate, is greatly reduced.

The subject of surface coating of wood cannot be treated exhaustively in this paper. (Further information can be taken from the extensive literature on the subject, f.ex. ref. 3).

Practical hints

- For a good dimensional stability, water repellent (WRP-) surface coatings of a certain film thickness are needed. In literature at least 50 microns are recommended. The optimum thickness of a coating is also product dependent. The protection given by thicker coatings is better, but they demand more work for refinishing: Thin coatings can mostly be brushed off, whereas thick coatings must normally be scraped off or even sandblasted.

- In practice the following *formula* has demonstrated the longest renovation periods:
  * One of two applications of a penetrating stain
  * plus a good top coat of 50-80 microns, using a film forming product of good quality

- The color, mainly for sun exposed members, should not be too dark to avoid undue heating up of the wood, but still have enough pigmentation to prevent photochemical degradation of the wood surface below the coating.

- An intact surface coating works two ways: In one direction it is as a physical shield preventing liquid water from reaching the wood surface, and in the other direction acts as vapor barrier. As soon as the coating film is cracked, liquid water can seep below the coating and is then trapped in the wood by the vapor barrier. The MC can build up with time and eventually lead to decay.
A cracked coating can be worse than no coating at all, which should explain the need for repainting when cracks start to show.

- Pressure treated wood *does not need* an additional surface treatment. The natural graying of the wood is slower than for untreated wood, but leads eventually to the same appearance. However, surface coatings do reduce water uptake and moisture movements of the wood and thus can increase the service life of a weather exposed structure still further.

- Apart from esthetic considerations, the additional expense of a good surface treatment is offset by longer maintenance period. Like pressure treatment, it is an excellent supplemental measure to prolong the durability of exposed structures, but (once more) *should never be a substitute for poor design.*

Maintenance and repair

No planning or execution is perfect and weather exposed wood reacts faster than steel or concrete to adverse influences. Therefore it is a good policy to check wooden structures already after a few years of service. Potential problems are then still small and can be easily remedied (5).

We cannot emphasize strongly enough the value of periodic checking and maintenance for the durability of exposed wooden structures. The work required will mainly consist of

- unplug and clean drainage paths for water
- repair covers and minor damages
- refinishing surface coatings before they have lost their effectiveness
- add covers where needed

Some of the details proposed are also very adequate for retrofitting existing structures when signs of decay can be noticed. Once water is prevented from getting into the endangered zones, the fungi will die off and the damage will be stopped.

Most of this maintenance work can be done by regular highways crews, occasionally assisted by a local carpenter or contractor.

Unfortunately, this aspect of control, maintenance and early repair, is often being neglected, postponed or not done knowledgeably. This may save some money in the short run, but end up costing a lot more with time. More important, however, is the fact that it is poor publicity for wood as bridge material.

Conclusions and outlook

The best way to guarantee in the future the use of wood for weather exposed structures is by designing them for the longest possible service life.

Retrofitting existing structures before decay has done extensive structural damage can be an efficient way to extend service life at low cost.

By combining good detail design with the effective supplemental measures described, i. e. water repellent surface coating and chemical treatment where needed - but only there! - it is possible to equip weather exposed wooden structures for a service life comparable to other construction materials - and still keep the advantage of wood as ecological material without disposal problems.

Following these guidelines wood has a great potential for weather exposed structures. Its main competitors, i.e. steel or concrete take a few years longer to show damages, but then, rehabilitation is usually difficult, expensive or not worth it at all.

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The Modern Timber Bridge Program in the State of Maine: A Five-Year Report

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Abstract

According to the most recent US Forest Service survey, Maine is the most heavily forested state in the US. However less than 3% of Maine's bridges are made out of timber. In 1989, the University of Maine led a state effort to develop a plan that would increase the use of Maine timber in bridge construction. The plan, called the Maine Timber Bridge and Infrastructure Initiative (MTBI) called for research at the University of Maine, construction of sixteen demonstration bridges over 10 years, and technology transfer. This paper describes the goals of the MTBI, the accomplishments of the program in the first five years, and outlines some obstacles and opportunities.

Keywords: Timber, bridges, research, design, cost, durability

Introduction

According to the most recent US Forest Service survey conducted in 1982, and now being repeated in 1996, Maine is the most heavily forested state in the US with nearly 89% of its land covered by forests. In recent years, Maine has also ranked fourth in the nation behind Oregon, Idaho, and Washington in the percentage contribution of the lumber/solid wood products industry to the total gross state product.

While the Forest Products Industry is a cornerstone of Maine's economy, less than 3% of Maine's bridges are made from wood. Maine is very typical of the New England states where wood is rarely used as a structural material in bridges. In comparison, other states have a significantly larger percentage of their bridge inventory made of wood. Arkansas for example has nearly one third of its bridges made of wood and Minnesota has nearly 15% of its inventory made of wood. Therefore, there appear to be some opportunities to utilize more wood on Maine's bridges.

The lack of use of timber in New England's bridges is also true for other aspects of heavy construction in the region. As opposed to the pacific Northwest, the New England region seldom uses heavy timber (including engineered wood products such as glued-laminated timber) framing in applications such as commercial and institutional buildings, large condominiums, hotels, manufacturing facilities, warehouses, shopping centers and the like.

In 1989, the University of Maine led a state effort to develop a plan that would increase the use of Maine timber in bridge construction. In the same year, a coalition was organized which included the Maine Department of Transportation, the Maine Resource Conservation and Development Areas (RC&Ds), the

Maine Forest Service, representatives of Maine's structural wood products industries, and representatives of Maine's congressional delegation. In 1990, the coalition issued a report entitled the Maine Timber Bridge and Infrastructure Initiative (MTBI). The report outlined a 10-year plan to increase the use of modern timber bridges in the state of Maine.

This paper describes the goals of the MTBI, the accomplishments of the program in the first five years, and outlines some obstacles and opportunities.

The 10-Year Maine Timber Bridge and Infrastructure Initiative Plan

Increasing the use of Maine timber in bridge applications was the ultimate goal of the MTBI. Timber bridges were seen to add value to the State's largest natural resource, possibly creating a new export potential for the state. The 1990 MTBI plan called for:

(1) Research at the University of Maine to develop economical timber bridge designs using Maine wood species

(2) The construction of sixteen demonstration bridges in the state over the 10-year period to demonstrate the new technology

(3) A technology-transfer effort to disseminate the new information through publications, seminars, conferences and videotapes.

Now that a plan was developed, it was necessary to secure resources so that the plan could be carried out. To help move the timber bridge technology forward both at the state and national levels, the MTBI coalition worked with Maine Senator George Mitchell. Maine Senator Mitchell then spearheaded the creation of a timber bridge program through the Federal Highway Administration (FHWA). This FHWA timber bridge program, made possible though the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, provided federal funding on a competitive basis for states to encourage the construction of innovative timber bridges using regional wood species. The five-year program also provided federal funding on a competitive basis for research in timber bridge technology.

The ISTEA legislation enabled the MTBI and the Maine DOT to build and monitor new types of modern timber bridges that would have been financially difficult to construct or monitor without the support of the program. To conduct the research, the University of Maine worked with the USDA Forest Products Laboratory Timber Bridge Team in Madison, Wisconsin on a number of cooperative programs. These cooperative research programs allowed the state of Maine to benefit from on-going research at the national level and to obtain better data on its own wood species. The University of Maine also obtained funding to carry out the research and technology transfer activities called for in the MTBI plan from various sources including the National Science Foundation, the Maine DOT, the Maine Science and Technology Foundation, and the USDA Timber Bridge Initiative.

Activities of the Maine Timber Bridge Initiative in the First 5 Years

Since the MTBI plan was developed in 1990, considerable progress has been achieved on accomplishing the three objectives outlined in the MTBI plan: (1) research, (2) demonstration bridges and (3) technology transfer.

In the research area, a number of projects have been conducted and some are still on-going. In general, the technical challenges result from the fact that many Maine wood species have relatively low mechanical properties and are only available in smaller dimension-size lumber. Also, many of Maine's wood species are refractory and difficult to properly treat with preservative chemicals. The situation is compounded by the fact that Maine has no glue-laminating facility, the closest being in Unadilla, New York. The state has no facility for treating wood with either creosote or pentachlorophenol. Up until very recently, the only local preservative treatment available has been Chromated Copper Arsenate (CCA).

Examples of Research Projects Conducted as Part of the MTBI

The following are examples of the research projects that have been conducted in association with the MTBI

1. Survey of Maine Timber Resources for bridge construction. One of the first projects that was completed was a critical survey of Maine wood resources to select species that would be most suitable for bridge construction. In evaluating each species' suitability, the following criteria were used: Adequate

stiffness and strength, good treatability with wood preservatives, availability in Maine forests, availability in lumber mills in the appropriate sizes and grades, and under-utilized species. Using these criteria, eastern hemlock, red maple and red pine appeared to be promising candidates for bridge construction in Maine.

2. *Use of CCA-treatment in stress-laminated decks.* CCA has been the only wood treatment available in the state. Not using CCA would have meant shipping Maine lumber out of state for treatment then back to the state, a situation which significantly reduces the economic viability of timber bridges. As a result, the Maine DOT has constructed three CCA treated decks; the oldest was constructed in 1991. The University of Maine has been monitoring these projects which have performed well so far.

3. *Use of Metal Plate Connected (MPC) wood trusses in bridge applications.* In Maine, as in many parts of the United States, the highest-capacity wood structural member fabricated locally is the metal plate connected (MPC) truss. MPC trusses are lightweight wood trusses assembled using dimension lumber and metal connector plates. They are commonly used for framing roofs and floors in both commercial and residential buildings. MPC trusses are cost-effective because they are constructed using widely available dimension lumber, are simple to fabricate, structurally efficient, and easy to handle. MPC trusses also offer high stiffness, which is particularly important in bridge construction. Until recently, however, these trusses have not been used for constructing timber bridges.

In 1991, the University of Maine and the USDA Forest Products Laboratory initiated a cooperative study to investigate the use of MPC trusses in low-volume rural bridge applications. The study showed that MPC trusses can be used in bridge applications provided that proper consideration for fatigue of MPC joints, corrosion protection and details to prevent connector plate 'back-out' are incorporated into the design. Using fatigue test results of 172 individual MPC joints and 33 full-scale trusses, recommendations for fatigue design were developed. Two MPC truss bridges, 46 ft and 39 ft long, were built in Maine in 1993 and 1994.

4. *Use of eastern hemlock and red maple in Glulam Bridges.* Eastern hemlock and red maple are abundant wood species in Maine, but are not used in commercial glulam construction. Glulam members in building and bridge construction in the state of Maine are currently imported from other states. This project, conducted cooperatively with the USDA Forest Products Laboratory and the Maine DOT, carried out MOE and knot-size testing of eastern hemlock and red maple that are needed for developing glulam combinations for these species. 31,000 lf of red maple and 1,000 lf of eastern hemlock were surveyed. Lay-ups for $24F_b$-1.8E red maple glulams were developed and a 52 ft span red maple girder bridge was constructed in Maine in 1995.

5. *FRP Reinforced eastern hemlock glulams.* The benefits of reinforcing glulam beams made with eastern hemlock, a relatively weak and under-utilized wood species in the state of Maine, are being studied. As part of one study, nine beams reinforced with fiber-reinforced plastics (FRP) on the tension side and three unreinforced controls were instrumented and tested to failure in four-point bending. Low, medium, and high quality wood were used in the experimental study. FRP reinforcement ratios ranged from 0.3% to 3.1 %. A nonlinear numerical model that predicts the performance of the FRP-glulam beams through the entire load range was developed and its predictions were compared with the test results. The reinforced beams showed substantial gains in strength (up to 56%) and stiffness (up to 37%) by the addition of less than 2.1 % FRP reinforcement. The reinforced beams also showed a significant increase in ductility, as the failure mode was shifted to a compression of the top wood fibers.

Despite the encouraging results, further research is necessary before FRP-reinforced wood beams are widely used in bridge applications. One major concern is the long-term durability of the FRP-wood interface in a bridge environment. The hygro-thermal-mechanical stresses that will develop at the wood-FRP interface in service need to be evaluated carefully. In addition, the interaction between moisture, temperature, fatigue and their effect on bond strength and creep behavior of the system are not entirely understood. Fundamental research at the University of Maine is on-going to address these and other related issues. To start collecting field data on durability, a 124ft long FRP-reinforced glulam pedestrian pier was constructed on the Maine coast in 1995. The pier is being monitored with particular attention to the FRP-wood interface and to creep. Two more FRP-reinforced glulam vehicular bridges are planned for construction in Maine in 1997.

Demonstration Bridges Constructed in Maine

The 1990 MTBI plan called for the construction of 16 demonstration timber bridges in the state of Maine over 10 years. In 1996, the demonstration bridge effort is on target in terms of the number of bridges built. Table 1 gives a list of the projects completed and those planned.

The technology transfer activities over the past five years have included over thirty presentations and conferences to various groups within the state and the preparation of videotapes on timber bridge technology. The MTBI activities have been featured in over forty news articles and television segments in and outside the state of Maine.

Concluding Remarks: Obstacles and Opportunities

Following over five years of research and the construction of numerous demonstration bridges, much has been learned by the MTBI participants. The following summarizes some of the important obstacles and opportunities:

1. Cost of timber bridges. This is probably the most significant barrier to the increased use of timber bridges, at least in the state of Maine.

Unfortunately, often times the ISTEA demonstration bridges are demonstrating to DOTs how expensive timber bridges can be. As one would expect, the cost of a one-of-a-kind demonstration bridge is high. In Maine, the most cost-effective demonstration projects were generally constructed outside the ISTEA program, often when town personnel or small local contractors were responsible for the construction. An opportunity might be for future demonstration programs to increase focus on non-DOT bridges.

2. Durability of timber bridges. This, along with the cost issue, are possibly the two most important barriers to the increased use of timber bridges in the state of Maine. The durability issue is always on the top of the list of concerns expressed by DOT engineers in MTBI meetings. On one occasion, an experienced DOT bridge maintenance engineer stated in a meeting "I have spent a career removing deteriorated timber bridges. Why do we want to build more of them?" However, many of the bridges that the engineer is referring to were not properly designed and some were not even treated with preservative chemicals. In any case, the DOT perception that timber does not last in a bridge environment has been expressed at the national level by recent surveys of DOT engineers where wood was ranked last for durability when compared with steel or concrete. Maine DOT experience is that treated wood piles have a useful life of only 20 to 30 years, whereas steel and concrete bridges are often assumed to last between 70 and 90 years.

To answer legitimate concerns of DOT engineers, it is

Table 1 - Maine Demonstration Bridges

Bridge Location	Length (ft)	Structure type	Wood species	Treat-ment	Funding	Year built
Gray	22	Stresslam deck	eastern hemlock	CCA	USDA	1991
Byron	46	Stresslam truss	southern pine	CCA	MDOT[a]	1993
N. Yarmouth	39	Stresslam truss	southern pine	CCA	MDOT	1993
Bangor	70	Covered bridge	eastern hemlock	CCA	USDA	1994
Sangerville	54	glulam	red maple	penta	MDOT	1995
Bar Harbor	124	FRP-glulam	red maple	penta	USDA	1995
Crowley Island	192	glulam	SP & e. hemlock	penta	Private	1995
Garland	30	stresslam/glulam	red maple	penta	MDOT	1996
Milbridge	16	FRP-stresslam	eastern hemlock	CCA	Town	1996
Milbridge Pier	185	FRP-stresslam	southern pine	CCA	Town	1997
Sebois	36	FRP-glulam	red pine	penta	USDA	1997
Otisfield	25	glulam	red maple	penta	MDOT	1997
Biddeford	28	glulam	red maple	penta	MDOT	1997
Malone	190	glulam	southern pine	penta	Private	1997
Medway	52	FRP-glulam	red maple	penta	MDOT	1997

[a] Most MDOT timber bridges were constructed under the ISTEA timber bridge demonstration program

critical to collect proper information on timber bridge durability. There is a lack of data on the durability of properly designed, properly treated timber in a bridge environment. This is mainly because 'modern timber bridges" have only been constructed in recent years. The required data may be best obtained by conducting on-site condition surveys of timber bridges in the US. The surveys should cover different environments, wood species, preservative types, structural systems and should attempt to identify design or construction related causes of wood deterioration.

3. Cost and availability of "under-utilized" local wood species. On many demonstration projects, it has been difficult to secure wood at reasonable prices in the grades or quantities or sizes required, particularly when 'under-utilized" native species were called for. For example red maple is the second most abundant species in the state of Maine. It is considered a 'weed' by many and it has very few uses. Only hardwood mills carry it and it is not graded to softwood standards.

As a result, demonstration projects in Maine have had to pay up to $1,200/thousand bf to obtain #2 or better, 2x6 red maple. On the other hand, imported southern pine is available in Maine at almost half the cost of native red maple. Contractors preparing bids for ISTEA projects quickly learn that there might only be one supplier for the required native species, that there might be significant delays in wood delivery, and that the native species must be transported out of state to be fabricated and treated. These barriers have caused bid prices to increase.

4. Expanding the scope of the timber bridge initiative. In recent years, the state of Maine DOT has been constructing at most thirty vehicular bridges/year. If 20% of these were made out of wood, at most six timber bridges would be built every year in Maine. This is hardly a number to sustain an industry or to create opportunities for a sustained supply of under-utilized wood species. Therefore, a good approach may be to expand the two national timber bridge programs to a general wood-in-construction utilization program. This is the direction that the MTBI has taken. A glulam plant that supplies girders for an industrial facility can also supply girders for bridges. By creating more demand for engineered wood products in heavy construction in general, rather than just bridges, there will be better opportunities to develop sustained value-added markets for under-utilized local species.

The Federal Highway Administration Timber Bridge Program

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Abstract

The Federal Highway Administration (FHWA), as directed by the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, Section 1039, is addressing the Highway Timber Bridge Research and Demonstration Program. Funding for the program is being made available for six years. The program, in its sixth year of funding, is being coordinated between the FHWA's Office of Engineering, the Office of Engineering Research and Development, and the Office of Technology Applications. This paper presents an overview of the program.

Keywords: Federal Highway Administration, Timber bridge program, ISTEA, Duwadi, Wood.

Introduction

The principle role of the FHWA is administering the federal-aid highway program. The federal-aid program has typically focused on roads and bridges on the more major highways like the interstates, primary roads and major arterials. The FHWA has played less of a direct role on our Nations secondary, local and rural highway systems. Under its Highway Bridge Replacement and Rehabilitation Program (HBRRP) between 15% to 35% of the available funds can be used by the States on local highways. As timber bridges exist mostly on local and rural highways, the FHWA has had less of an involvement in these types of bridge systems until the ISTEA legislation of 1991.

The FHWA requires the states to inventory and inspect all its bridges on public roads. Based on the National Bridge Inventory (NBI) of a total of 576,874 bridges, 41,743 (7.2 percent) are timber bridges. Figure 1 shows the distribution of these bridges nationwide within the 48 continental United States. In addition, in the NBI there are approximately equal number of bridges, 42,102 (7.3 percent) that have timber decks supported by steel stringers, and are therefore classified as steel bridges. The distribution of these are shown in Figure 2. The NBI also provides an assessment of our Nation's bridges. Based on this data, 18.6 percent of our Nation's bridges are structurally deficient. Table 1 gives the proportion of structurally deficient bridges by material type. Approximately 18 percent of the structurally deficient bridges are timber bridges.

States and counties continue to build bridges out of wood as it has numerous characteristics that make it a desirable material for transportation structures:

- It is a renewable resource.
- It is naturally resistant to the effects of deicing agents.

Figure 1—Distribution of Timber Bridges

Figure 2—Distribution of Steel Bridges with Timber Decks

Table 1—Structurally deficient bridges by material type.

Material Type	# Structurally Deficient	Percentage
Timber	19,738	18.35 %
Concrete	20,847	19.38%
P/S Concrete	3,328	3.09%
Steel	62,396	58.02%
Other	1,234	1.15%

- Construction can be completed in any type of weather without affecting the material, and with a minimal workforce.

Legislation

In 1989, to encourage the use of wood for bridge construction, Congress established the Timber Bridge Initiative program to include research, technology transfer, and demonstration bridges. The responsibility for administration of the Timber Bridge Initiative was given to the United States Forest Service. In 1991 under ISTEA, Congress established the Highway Timber Bridge Research and Demonstration Program. As with the Timber Bridge Initiative, the program includes research, technology transfer, and demonstration bridges. The responsibility for administration of this Program was given to the FHWA.

The ISTEA, Section 1039(a) authorizes the Secretary to make grants to carry out research on one or more of the following areas:

- Development of new, economical highway timber bridge systems.
- Development of engineering design criteria for structural wood products for use in highway bridges in order to improve methods for characterizing lumber design properties.
- Preservative systems for use in highway timber bridges which demonstrate new alternatives and current treatment processes and procedures and which are environmentally sound with respect to application, use, and disposal of treated timber.
- Alternative transportation system timber structures which demonstrate the development of applications for railing, sign, and lighting supports, sound barriers, culverts, and retaining walls in highway applications.
- Rehabilitation measures which demonstrate

effective, safe, and reliable methods for rehabilitating existing highway timber structures.

The ISTEA, Section 1039(b) authorizes the Secretary to make the information and technology resulting from research available to State and local transportation departments and other interested persons.

The ISTEA, Section 1039(c) authorizes the Secretary to make grants to States for construction of highway timber bridges on rural federal-aid highways. Criteria for approval is to be based on:

- Bridge designs which have both initial and long-term structural and environmental integrity.
- Bridge designs which utilize timber species native to the State or region.
- Innovative bridge designs which have the possibility of increasing knowledge, cost effectiveness, and future use of such designs.
- Environmental practices for preservative treated timber, and construction techniques which comply with all environmental regulations.

One million dollars per year is provided to the Secretary for these programs for research and technology transfer, and $7,500,000 per year ($7,000,000 in FY92) for construction of the bridges. A 20% match is required from any organization receiving research projects and from the States receiving grants for construction of the bridges.

Program Implementation

This section describes the implementation process established by each office of the FHWA responsible for carrying out the program.

Research and Technology Transfer

The FHWA Office of Engineering Research and Development - Structures Division, and the Office of Technology Applications elected to pool funds and resources with the FS - Forest Products Laboratory (FPL) and establish a single coherent national Timber Bridge Research and Technology Transfer program. This interagency cooperation also ensures elimination of duplication of research.

Timber bridge research needs was determined through a committee consisting of government, academia, and industry representatives by the FHWA under its "Outreach Program" in 1989. Anticipating a rise in funding, this list of research needs was further expanded by Iowa State University under a study sponsored by the FHWA, and FPL. The results of this study, included in a report titled, "Development of a Six-Year Research Needs Assessment for Timber Transportation Structures" forms the basis of the six year joint timber bridge program for both the FHWA and FPL. The timber bridge research activity under this national research program is divided into the following six research areas to coincide with the ISTEA Legislation:

- Area I: System Development and Design
- Area II: Lumber Design Properties
- Area III: Preservatives
- Area IV: Alternate Transportation System Timber Structures
- Area V: Inspection/Rehabilitation
- Area VI: Technology and Information Transfer

The program is administered under an Interagency Agreement with the FPL. Under the program joint project identification and prioritization is conducted by the FHWA and FPL. The research studies are cooperative studies with universities, local government agencies, and industry.

Demonstration Bridges

The FHWA demonstration bridge program is administered and managed by the FHWA Office of Engineering - Bridge Division. Candidate projects are selected by State highway agencies and submitted to the FHWA for the final decision. The candidate bridges must meet eligibility criteria of the HBRRP, 23 U.S.C. 144. The ISTEA Legislation lists four additional criteria for selection and approval of the grants. Funds for the program are set aside from the FHWA Bridge Discretionary Program. Funds are required to be obligated and construction contracts awarded during the same fiscal year as the grants. As originally written only those bridges on the federal-aid highway system were eligible for the ISTEA funding. However, as most of the timber bridges are on off system roads, the legislation was modified in the U.S. Department of Transportation 1993 Appropriations Bill to allow construction grants for timber bridges on any public roads if otherwise eligible for the bridge program. During fiscal years 1992 and 1993, applicant bridges were subjectively ranked on how well each fulfilled the above criteria. Since fiscal year 1994, selection has been made based on a Priority Rating Factor (PRF) that is computed by a formula given below.

$$PRF = \frac{10}{(RI + RN) x RS x RE} \quad (1)$$

- RI - is based on consideration of type, length, and width of the bridge for innovative designs.
- RN - is based on sufficiency rating as a measure of need.
- RS - is based on sources of timber species.
- RE - is based on effective use of funds for bridge construction versus approach roadway work (a maximum of 10 percent may be used for approach roadway work).

Special consideration is given to environmentally sensitive designs and those utilizing environmentally approved preservatives. Also, during project selection special attempt is made to obtain a geographical mix nationwide, thus giving all of the states ample opportunity to gain experience in the design and construction of timber bridges. Timber designs for bridges built on the National Highway System (NHS) have to meet the American Association of State Highway and Transportation Officials (AASHTO) standards for highway bridges. Those not on the NHS may be designed in accordance with individual State-approved standards.

Key Studies & Status

This section highlights the studies that are under way in the research and technology applications area, many of which are described in more detail in papers by others in these proceedings. This section also discusses the demonstration bridge projects funded by the FHWA.

Research & Technology Applications

Research studies cover the range of topics identified in the Legislation. This includes development of criteria to improve design and performance of timber structures; development of methods and equipment for inspection and rehabilitation; and transfer of technology through design aids in the form of standard plans, and reports, and through presentations at national conferences. Currently 31 studies are underway in the six broad areas. Studies are being conducted nationwide involving numerous universities, and other organizations. At the conclusion of the program over 46 studies will have been conducted. Summaries of all research studies underway including all the FPL funded projects are described in a paper prepared annually by Duwadi & Ritter titled, “Status of Research on Timber Bridges and Related Topics”.

Although ISTEA provided $1,000,00 per year for research and technology transfer activities, the actual available FHWA funding has been $990,000-FY92, $803,050-FY93, $911,000-FY94, $1,000,000-FY95, and $874,576-FY96 due to obligation limitations set forth in the Transportation Appropriations Bills. The following lists specific studies jointly conducted by pooling FPL and FHWA resources that have been underway utilizing the FY92-FY95 funds. Studies utilizing FY96, and FY97 funds are under development.

Area I: System Development and Design–

- Timber bridge monitoring and evaluation including full-scale load testing.
- Field evaluation of a timber bridge constructed with metal plate connected trusses.
- Develop long-span timber bridge systems using glued laminated timber.
- Development of stress-laminated truss bridges using light-frame metal plate connected trusses.
- Evaluate cold temperature effects on stress-laminated timber decks.
- Dynamic evaluation of timber bridges.
- An independent review & analysis of the stressed T & Box design concepts, laboratory works, and field performances.

Area II: Lumber Design Properties-

- Shear strength of sawn lumber beams.
- Load and resistance factor design calibration.

Area III: Preservatives-

- Accelerated laboratory testing of new wood preservatives- ecosystem studies.
- Accelerated laboratory testing of new wood preservatives - pure culture studies.
- Treatments and methods for field treating bridge members.
- Performance characteristics of various wood preservatives for stress-laminated bridge applications.
- Manual on wood preservatives.
- Treatability of heartwood.
- Copper naphthanate preservative for bridge applications.
- Moisture protection for timber members.
- Fumigants for use in preserving wood members.
- Environmental effects (leaching) of commonly used preservatives.

Area IV: Alternate Transportation System Timber Structures-

- Development of crash worthy bridge rail systems; PL-2, longitudinal decks.

Figure 3-Timber bridge research program highlights.

- Development of sound barriers.
- Development of crash worthy bridge rails -PL2, transverse timber decks.

Area V: Inspection/Rehabilitation-

- In-place evaluation of timber bridges using stress wave technology.
- Equipment and methods for determining the in-place stiffness of stress-laminated timber decks constructed of sawn lumber.
- Guidelines for the design and application of waterproof asphalt wearing surfaces for timber decks.
- Manual for timber inspection - supplement to the Bridge Inspector's Training Manual - 90.
- Development of NDE equipment for inspection of timber bridges.

Area VI: Technology and Information Transfer-

- Meetings toward development of AASHTO Specifications for timber bridge design.
- Standard plans and specifications for timber bridge superstructures.
- Conference on Wood in Transportation Structures - State-of-the-Art.

Figure 3 highlights these research areas and the significant products of the program. Major accomplishments will include: a database on the field performance of timber structures for refining design criteria to update the AASHTO codes; development of crash tested AASHTO/FHWA approved bridge rails for use on transverse, and longitudinal timber deck bridges; standard plans for bridge rails, and for several timber bridge types for use by states, counties, and local authorities; nondestructive evaluation methods and equipment for inspection of timber components; development of better preservatives and coatings, and a manual on wood preservatives; and development of a wood noise barrier for use along the highways.

Demonstration Bridges

Because of the many enhancements that have been made

Figure 4—Number of bridges funded by fiscal year.

Figure 5—Number of bridges funded by States

over the years in the area of wood treatment and preservation, engineered wood products, and general design provisions the timber bridges constructed today are significantly different than those built centuries ago. Glued-laminated timber (glulam) technology, structural composite lumber, wood reinforced with fiber reinforced composites are all engineered wood products that have made it possible to produce larger members using shorter pieces of lumber, and eliminate strength-reducing characteristics of wood such as knots in sawn lumber. Stress-laminating technology, where sawn lumber laminaes are post-tensioned together to produce a structure that behaves as a unit to distribute the loads has virtually replaced the traditional nail-laminated bridges where problems exist with nails tending to come loose through cyclic loading, and moisture changes.

The majority of bridges being built under the program have been stressed-laminated bridges, dowel laminated bridges, glulam stressed deck bridges, and glulam stringer bridges with transverse glulam deck panels.

Stressed T's and Boxes where the flanges are of sawn lumber and the webs are of glued laminated or laminated veneer lumber are also being built, although there is no AASHTO approved specifications at this time for these bridge types. Few bridges have been built where glulam beams have been reinforced with fiber reinforced plastic composites at the tension flanges.

The number of proposals received at the FHWA Headquarters for funding has varied. In FY1992, FY1993, FY1994, FY1995, and FY1996 around 46, 101, 109, 42, and 54 applications were received, respectively. Of these, grants were given to 34, 45, 27, 36, and 37 projects at the beginning of fiscal years 1992, 1993, 1994, 1995, and 1996, respectively. The Federal share for the 5 years of funding has been $36,991,842. In some cases funding has been returned by the States back to the FHWA. In these cases the funds are carried over to subsequent years. As a result of some States returning funds, and with carry overs to subsequent years, out of the actual number of grants given 11 projects were funded in FY92, 50 projects in FY93, 23 projects in FY94, 32 projects in FY95, and 37 projects in FY96. The number of bridges funded are plotted in Figures 4, and 5 by fiscal year, and by States, respectively.

Conclusion

Awareness of wood as a viable construction material for todays transportation structures has significantly increased in the past few years. This can be attributed to the Timber Bridge Initiative and the ISTEA programs. During the period from 1989 to 1995 approximately 1947 timber bridges were constructed as recorded in the NBI.

As with all materials, the construction of timber bridges is influenced by economical designs and material costs. The ISTEA allows funding for construction of the bridge, approaches, and incidental costs. On application forms many times these costs are combined, and also the actual construction costs may vary. The FHWA does not have a firm data on the actual costs of these bridges at this time. The Forest Service, in monitoring it's demonstration projects, has found that the cost of material varies from region to region depending on the availability of species suitable for bridge construction. The economics depend on the efficiency of design, the availability of a fabricating plant and preservative treatment plant, etc. The two basic trends seen by the

Forest Service are that bridges built in areas of the country where softwood species are produced have a cost advantage over other areas, and species that are readily available as "off-the-shelf items" have a cost advantage over other species (USDA. 1995).

Significant research will have been conducted over the six year period in the area of timber bridges. Some research projects will have direct applications in improving and advancing the timber bridge technology. Projects producing standard designs; crash tested rails; inspection manual; preservatives manual; noise barriers; and a database on the field performance of timber bridges are such studies. Other projects where long term performance is desired to determine its effectiveness such as preservatives, fumigants, and coatings may need further evaluation beyond FY97. Projects involving newer technologies or manufacturing processes may also need further evaluation. Through monitoring of the stressed-laminated bridges, a need to develop alternate stressing systems is being observed. Once developed these systems will need further testing and evaluation. The industry and many universities are conducting research into wood composites where wood is combined with fiber reinforced plastic (FRP) composites to increase member strengths. Performance of these will need to be monitored independently as many of these are proprietary products.

Although timber bridges have been used for hundreds of years, newer technologies are being continuously developed. These need to be adapted and expanded to improve existing systems and develop new systems which allow for efficient use of the material. Wood is one of the traditional materials along with steel, and concrete, that needs enhancements through research. Wood has numerous beneficial properties such as natural damping properties, resistance to deicing chemicals, and it is a renewable resource. Through advances in material science, engineered wood products such as glued-laminated lumber, laminated veneer lumber, parallel strand lumber, and wood reinforced with fiber reinforced plastic (FRP) composites have been developed. Aside from glued-laminated lumber, others are relatively new for bridge applications and need continued research both in the laboratory and in the field through demonstration structures to determine their potential for revitalizing our infrastructure. There are 576,874 bridges in the NBI of which 18% are structurally deficient. Many of these are short span structures suitable for a modern timber bridge.

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Wood In Transportation Program, Technology Transfer Efforts

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Abstract

The backbone of the Wood In Transportation (WIT) Program, formerly the National Timber Bridge Initiative, is technology transfer. Technical information on building vehicular and pedestrian bridges, portable bridges for temporary access, retaining walls, railroad and other transportation structures with timber has to be readily available and easy to understand. One purpose of technology transfer is to ensure that appropriated USDA Forest Service dollars are effectively used to promote and demonstrate efficient use of local timber for wood in transportation applications. This presentation will provide an overview of the Forest Service's technology transfer activities for the WIT program.

Key Words: technology transfer, timber bridges, wood in transportation

Introduction

Technology transfer activities are coordinated by the Timber Bridge Information Resource Center (TBIRC) in Morgantown, West Virginia. The Center plays a vital role in fostering information sharing with highway officials, community leaders, and others across the Nation. Technology transfer activities provide an avenue for the Forest Service to meet the goals and objectives of the WIT program. This is done by providing information acquired from demonstration projects and related research to people who will use it. Users include community, business, and government leaders who make decisions affecting local community vitality.

The Program's technology transfer effort is successful because of the collective efforts between the three main branches of the Forest Service and its external partners. Examples of cooperative activities include the quarterly newsletter, "*Crossings*", which over 4,500 individuals or organizations receive; and the development and distribution of publications, such as the "Timber Bridges: Design, Construction, Inspection, and Maintenance" manual. Other examples of technology transfer are special project reports, conferences, seminars, workshops, professional presentations, videos, and promotional material. This material is assimilated from a variety of cooperatively funded projects and activities. Resource Conservation and Development Areas, county officials, universities, state departments of transportation, and industry are the most common partnerships.

This presentation highlights cooperative efforts and partnerships developed by the Forest Service, both externally and internally. Readers will learn how the TBIRC publicizes the WIT program and how

information is managed and distributed via an office automation system, printed material, and electronic media. A brief overview of the WIT program is provided first.

WIT Program Summary

The WIT program was initially funded by the U.S. Congress in 1989. At that time, it was known as the National Timber Bridge Initiative. Its purpose is to improve rural transportation networks and revitalize rural economies by using wood for transportation structures. Since the beginning of the Program, 359 demonstration vehicular and pedestrian timber bridges, in addition to 66 special projects, have been funded in 48 states. As of June 1996, more than 200 bridges have been built with improved engineered designs and advanced preservative treatment techniques.

In the past, the major emphasis of the Program has been on vehicular highway bridges. However, because of increased interest and demand, the Program has broadened into several additional markets. The direction of the Program now includes portable bridges for temporary access, pedestrian and trail bridges, retaining walls, box culverts, sound barriers, and marine and railway structures. The Program continues to emphasize the stewardship of our forestland through the development of additional markets for low-value, underutilized local timber resources.

The primary goal of the Program is to diversify local economies, which is being carried out by the following objectives:

- Improving rural transportation networks,
- Expanding the range of markets for wood products,
- Creating service industries for wood in transportation structures,
- Commercializing modern timber bridge technology,
- Innovation that leads to cost-saving strategies and improved performance of existing designs,
- Expanding the Program to include other wood in transportation applications, and
- Improving America's forests through stewardship.

These objectives are being achieved through four distinct, yet interrelated components:

1. Wood In Transportation Demonstration Projects: These projects utilize timber structures to improve rural road systems, establish markets for wood products, and create service industries for wooden structures. In addition to demonstration timber bridges and other projects funded by the Program, timber bridges are also being emphasized on National Forest System (NFS) land in an effort separate from the WIT program. Since 1989, more than 250 timber bridges have been constructed on NFS land.

2. Research: Research is conducted to optimize the balance between existing and developing technology in the use of wood as a construction material. Much of the work is cooperative in nature. Recent advances in wood treatment, engineered wood composite products, and bridge designs provide for the increased use of wood as a construction material to assist in the cost-effective rebuilding of our Nation's transportation infrastructure.

3. Technology Transfer and Information Management: The TBIRC provides information to interested individuals and organizations. It also administers the demonstration grant program.

4. Rural Revitalization: All WIT projects have the potential to stimulate rural economies. Many projects are constructed with locally available timber using local resources.

Table 1 shows the budget of the WIT program.

Budget Item	Final FY1994	Final FY1995	Final FY1996
	… Dollars in thousands . . .		
Demonstration Projects	$1,009	$1,020	$604
Research	1,093	1,100	770
Technology Transfer	732	671	596
Totals	$2,834	$2,791	$1,970

Cooperative Efforts and Partnerships

Interested citizens, highway officials, bridge engineers, community planners, and others discover the WIT program through various means. The majority of the budget funds a variety of demonstration projects. Demonstration projects are widely publicized by cooperators, which leads to interest in surrounding communities.

For example, in 1993, the Forest Service helped fund a 40-foot, two-lane vehicular bridge in Meriwether County, Georgia. The Meriwether County Board of Commissioners and the Two Rivers Resource Conservation and Development Council were awarded the grant. This bridge withstood a 500-year flood in July 1994. Since that time, many county government officials in the area have visited the bridge and are now interested in building timber bridges in their communities.

The WIT program also funds special projects that generate interest in using timber for transportation structures. The National Timber Bridge Design Competition is a project partially funded by the WIT program. The student competition, sponsored by the Southwest Mississippi Resource Conservation and Development (RC&D), Inc., promotes the use of wood as a competitive bridge construction material. This annual event involves leaders from the transportation and engineering communities with an emphasis in the academic community.

The Development and Distribution of Publications

After various demonstration and special projects are complete, the TBIRC develops and distributes cost and contacts information and/or technology transfer reports on them. For instance, the TBIRC is working with the Southwest Mississippi RC&D, Inc. to produce a brochure to further advertise the student competition.

In Sussex County, New Jersey, the WIT program partially funded a suspension trail bridge on the Appalachian Trail that was completed in the fall of 1995. Conklin Associates designed the bridge. Tibor Latincsics, Conklin Associates, became aware of the Program through word-of-mouth promotion by individuals from the New Jersey Forestry Services. As a follow-up to this project, Conklin Associates in cooperation with the TBIRC is developing a special project report on suspension trail bridges. There is a lack of information available on this particular design. With more communities showing interest in pedestrian trails, there is a great need to make this information and related trail bridge designs available.

Research efforts of the WIT program are administered at the Forest Products Laboratory located in Madison, Wisconsin. This work is cooperative in nature with partners, such as the Federal Highway Administration, National Forest System, universities, and state departments of transportation. To get research findings to the public, the TBIRC works closely with the Forest Products Laboratory in distributing their publications that explain the latest technical advances. These publications provide engineers, local community leaders, and others with information on bridge design, fabrication, preservative treatment, construction and erection techniques, and performance.

In addition to the examples listed above, the TBIRC is currently working to fill even more gaps. For instance, the Center is finalizing a case study on portable bridges for timber harvesting. The TBIRC is also working with engineers from two counties to produce case studies on completed demonstration timber bridges. Cost information on projects is recorded in the TBIRC database as projects are completed. This information is used to develop periodic cost reports.

Displays, Conferences, and Seminars

Aside from the various publications distributed by the TBIRC, customers also learn about the WIT program from displays, conferences, and seminars. The TBIRC prepares, presents, coordinates, or helps fund these activities.

WIT information and technology has been made available to potential users at formal conferences. An estimated 14,000 state and county officials, engineers, and involved citizens have participated in these forums since the Program's beginning. To date, 48 formal conferences have been held within the guidelines of the WIT program. More conferences are tentatively scheduled.

WIT Coordinators

Forest Service technical advisors are located strategically throughout the country to help implement the WIT program. These coordinators provide information and work with local partners in developing and implementing demonstration projects. These relationships provide the opportunity for WIT coordinators to share information learned from past demonstration projects and to provide advice to cooperators on the demonstration grant component of the Program. Program coordinators are responsible for:

- Coordinating the demonstration WIT proposal process,
- Coordinating local conferences, workshops, and seminars,
- Providing technical assistance and disseminating information to potential users, and
- Providing information to the TBIRC.

Crossings Newsletter and INTERNET

Activities throughout the country, from informal conversations to conferences organized by coordinators, help publicize the WIT program. *Crossings*, the quarterly newsletter of the WIT program, and the WIT program's home page on the INTERNET reach thousands of potential customers. *Crossings* is available free of charge and distributed to 4,500 companies, agencies, and individuals. Its purpose is to provide up-to-date information on wood in transportation activities.

The reason for TBIRC's use of INTERNET is to make more information available electronically. Currently, limited information is available. In the future, the home page will be updated to include as much of the TBIRC's library material as possible.

Information Distribution

The TBIRC distributed approximately 31,000 pieces of wood in transportation information in Fiscal Year 1995. Most requests are received by telephone, mail, or the INTERNET. To more efficiently distribute this information to customers, the TBIRC is developing an Office Automation and Technology Transfer System (OATTS). Once it is complete, customer service to both external and internal partners will be improved. Some of the highlights of this effort are to update and organize basic project information, improve the tracking capability of projects, and monitor grant extensions and project status. Office Automation will also allow the TBIRC to better organize final project data such as costs and contacts information.

The technology transfer part of the system will include several elements. Some of these are a filing/inventory/catalog system for publications, an information request entry and processing system, and technical information on the INTERNET.

Presently the TBIRC receives an average of four requests per day. This figure does not include requests that coordinators receive. The following graph shows the percentage of requests by customer type for publications and designs received by the TBIRC in Fiscal Year 1995.

Conclusion

The WIT program is gaining visibility and support because of its technology transfer efforts. Demonstration projects build partnerships and promote the Program. Cost data, special project reports, and other publications educate engineers, community leaders, and others about using timber for wood in transportation applications. Conferences, displays, seminars, *Crossings*, INTERNET, and WIT coordinators introduce hundreds of potential users to the Program.

To make outreach efforts more effective, the TBIRC is implementing an Office Automation and Technology Transfer System. This system will facilitate customer relations with external and internal partners. The WIT program's overall technology transfer effort is producing results that will usher the Program boldly into the 21st century.

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Standard Plans for Timber Highway Structures

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Abstract

Approximately 41% of the 578,000 highway bridges in the United States are currently in need of repair or replacement (USDA 1995). Many of these bridges are short span crossings on rural roads and are ideally suited for wood construction. However, wood is seldom considered in the selection of a structural material for bridge construction because many engineers are unfamiliar or inexperienced with wood design. For wood to be a viable material for highway structures, engineers must have access to design tools that make timber design an easy and more familiar process. One such tool is standard plans that present a clear and concise design and are adaptable to a variety of parameters. This paper presents a summary of three sets of standard plans for timber highway structures developed through cooperative research at the USDA Forest Service, Forest Products Laboratory: *Standard Plans for Southern Pine Bridges, Standard Plans for Timber Bridge Superstructures,* and *Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks.* Copies of these plans are available through the USDA Forest Service.

Keywords: Timber, bridge, design, railing, standard plans, superstructures, wood.

Introduction

As the 21st century approaches, a large number of highway bridges will be built to maintain critical links in the national transportation infrastructure. In designing these bridges, engineers will face a choice of structural materials. Of these materials, most engineers are least familiar with the design aspects of timber structures. This is partly due to a lack of educational training in wood design, because most colleges and universities do not offer such courses. In addition, many engineers do not see the need to invest or do not have the time to become familiar with wood design. As a result, a large percentage of short- to intermediate-span bridges are constructed with concrete and/or steel. For wood to be a viable material for future highway structures, engineers must have access to design tools that make timber design an easy and familiar process.

One such design tool is standard plans. Effective standard plans present clear and concise designs that are adaptable to numerous circumstances and parameters and meet structural and serviceability requirements. Standard plans provide an immediate overview of a design, indicating the appropriate circumstances for use and requirements for execution. By assembling design information and presenting it in the form of

Figure 1 -Typical configuration of a stress-laminated deck system.

Figure 2 -Typical configuration of a longitudinal stringer bridge.

standard plans, preparation and investigation time required by an engineer is greatly reduced while their knowledge base increases.

Many standard plans are available for steel and concrete bridges; therefore, most engineers are familiar with these plans for highway structures. However, until recently, standard plans for timber highway structures were very limited and were typically outdated in terms of design requirements. This void has been identified by engineers throughout the United States as an area requiring improvement (Wipf and others 1993). To improve technology transfer regarding wood use in transportation structures, three sets of standard plans for timber highway structures, resulting from cooperative research conducted at the USDA Forest Service, Forest Products Laboratory (FPL), have been developed: *Standard Plans for Southern Pine Bridges, Standard Plans for Timber Bridge Superstructures,* and *Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks.*

The standard plans for timber highway structures developed through cooperative research at FPL are intended to serve as a useful guide to state, county, and local highway departments, as well as private engineering firms in the development of practical and economical timber bridge designs. In these plans, complete design, fabrication, and construction information are provided for several timber bridge superstructures and railing systems. Each system is adaptable to various span length and width combinations, although specific site conditions may require modifications.

The bridge superstructures included in the standard plans are either longitudinal deck systems or stringer systems. For the longitudinal deck systems, a series of laminations are placed on edge and fastened to one another with nails, spikes, steel bars, or glued-laminated sections are used. Longitudinal decks can be assembled in-place or laminations can be prefabricated into panels for easy installation. For stringer systems, longitudinal stringers support a transverse deck.

The designs were developed in accordance with the American Association of Highway Transportation Officials (AASHTO) Standard Specifications for Highway Bridges and Guide Specifications for Bridge Railings and are presented in a manner intended to clearly communicate content in an efficient and understandable manner. All designs should be reviewed by a registered professional engineer prior to construction. An overview of the three sets of standard plans that are now published for timber highway structures follows.

Southern Pine Bridges

Published in September 1995, *Standard Plans for Southern Pine Bridges* was developed under a cooperative research agreement with FPL, the University of Alabama, and the Southern Forest Products Association (Lee and others 1995). The publication includes designs for three Southern Pine timber bridge superstructures: stress-laminated sawn lumber bridges, stress-laminated glued laminated timber (glulam) bridges (Figure 1), and longitudinal stringer with transverse plank deck bridges (Figure 2).

In devising this set of plans, the objective was to provide complete information regarding all aspects of the design and construction of the bridge superstructure, enabling an engineer unfamiliar with timber bridges to easily understand and implement the design. The plans include deck details, railing and curb configurations, suggested substructure attachments, material lists, fabrication details, construction recommendations, and design examples. The designs are based on AASHTO HS 20-44 and HS 25-44 vehicle loading, and two live-load deflection criteria options, including

L/360 and L/500. The superstructure designs are applicable for single- and double-lane, nonskewed, simple-span bridges. A summary of each superstructure type included in *Standard Plans for Southern Pine Bridges* is presented in Table 1. (See Table 2 for metric conversion factors.) Specific details pertaining to each design follows.

Stress-Laminated Sawn Lumber Decks

Laminations in the stress-laminated sawn lumber deck are Southern Pine, visually graded No. 2, dimension lumber. The laminations are full span (no butt joints) measuring 2 in. in nominal thickness and 8, 10, or 12 in. in nominal width, depending upon the span length. Bridge lengths included in this design range from 10 to 20 ft. The minimum required nominal deck thickness (lamination width) is determined after the span, loading, and deflection criteria are defined. A materials list, fabrication details for all superstructure components, construction recommendations and procedures, and design examples are included.

Stress-Laminated Glued Laminated Timber Decks

Laminations in the stress-laminated glulam timber decks are Combination 24F-V3 Southern Pine glulam. The deck laminations may be any standard glulam width up to 6.75 in. or a combination thereof. The minimum required deck thickness (depth of the glulam beams) is selected when the span, loading, and deflection criteria are established, and ranges from 11 to 16.5 in. This set of plans includes bridge lengths ranging from 20 to 32 ft. As with the sawn lumber stress-laminated system, all bridge details, a materials list, fabrication details, construction recommendations, and design examples for the superstructure are presented.

Longitudinal Stringer with Transverse Plank Deck

Designs for the longitudinal stringer with transverse plank deck require the stringers and plank decking to be Southern Pine, visually graded No. 1 Dense or No. 2. The allowable stringer sizes are nominal 6 x 14 in., 6 x 16 in., and 8 x 16 in., and the nominal plank sizes are 3,4, or 5 in. wide and 8, 10, or 12 in. thick. These plans include bridge lengths ranging from 5 to 23 ft and are intended only for the AASHTO HS 20-44 vehicle loading and a live-load deflection of L/500. All details for the superstructure are presented, although fabrication details are given only for steel plate and angle components. Design examples are not presented as with the stress-laminated systems, but design considerations and assumptions are outlined.

Table 1 -Summary of superstructures included in *Standard Plans for Southern Pine Bridges.*

Bridge type	Material grade	Member size (in.)	Bridge length (ft)
Stress-laminated sawn lumber	No.2	2 by 8 2 by 10 2 by 12	10-20
Stress-laminated glulam	24F-V3	up to 6.75 by 11 - 16.5	20-32
Longitudinal stringer with transverse plank deck	No.1 Dense or No.2	6 by 14 6 by 16 8 by 16	5-23

Table 2-SI conversion factors.

English unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)

Timber Bridge Superstructures

Standard Plans for Timber Bridge Superstructures (Wacker and others [in press]) was developed through a cooperative research project with FPL, Laminated Concepts Incorporated, and the Federal Highway Administration (FHWA). Scheduled for publication in September 1996, the standards include designs and details for seven superstructure types. The longitudinal deck systems included in the plans consist of nail-laminated (Figure 3) and spike-laminated decks (Figure 4), sawn lumber and glulam stress-laminated decks (Figure l), and longitudinal glulam panel decks (Figure 4). The longitudinal stringer systems included in the plans are a glulam stringer system (Figure 2) and a steel stringer system (Figure 5), both with transverse glulam decks. For the steel stringer system, details are given only for the transverse glulam panel deck and its attachment to the steel stringers. Table 3 provides a summary of the deck systems included in these standard plans.

Table 3-Summary of deck systems included in *Standard Plans for Timber Bridge Superstructures.*

Superstructure type	Bridge length (ft)	Deck thickness (in.)
Sawn lumber deck systems		
Nail-laminated	10 - 28	8 - 16
Spike-laminated	10 - 34	8 - 16
Stress-laminated	10 - 34	8 - 16
Glulam deck systems		
Longitudinal panel	12 - 38	8 - 16
Stress-laminated	10 - 60	9 - 24
Stringer with transverse deck	20 - 80	5.125
Transverse deck for steel stringers	N/A	5 - 8.75

The intent of this set of plans is to provide basic information and specifications for a wide range of timber bridge superstructures to an engineer somewhat familiar with timber design. Detailed information for the deck configuration and design tables for numerous geometry and material combinations are provided. For a given bridge system, the user establishes the design load (AASHTO HS 20 or HS 25), span length, number of lanes (single or double), orientation of the crossing (rectangular or skewed), and deflection limit (L/360 or L/500). When these factors are defined, the standard plans yield the appropriate material requirements and corresponding member sizes. The plans are not limited to a specific species.

Crash-Tested Bridge Railings for Longitudinal Wood Decks

Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks, published in September 1995, presents five bridge railing plans that reflect the results of a cooperative research project with the Midwest Roadside Safety Facility, University of Nebraska, Lincoln, FPL, and FHWA (Ritter and others 1995). The project objective was to develop and crash test bridge railings and approach railings for longitudinal wood decks. Historically, bridge railings have been designed based on static-load design criteria. Because full-scale vehicle crash testing is becoming recognized as a more reliable method of evaluating bridge railing acceptability, the development of crash-tested rail systems for timber bridges is a critical component in advancing the approval and use of timber highway structures.

Figure 3-Typical configuration of a nail laminated deck system.

Figure 4-Typical configuration of a longitudinal glulam or spike-laminated panel deck system.

Figure 5-Typical configuration of a steel stringer deck system with a transverse glulam panel deck.

The bridge railings included in the project were designed in accordance with the requirements of AASHTO Performance Level 1 (PL-1), AASHTO Performance Level 2 (PL-2), and the National Cooperative Highway Research Program (NCHRP) Report 350 Test Level 4 (TL-4). The approach railings were tested or adapted from previous testing in accordance with NCHRP Report 230. The sets of railing plans presented in the publication are illustrated in Figures 6 and 7. Figure 6 depicts the rails that were crash-tested to meet AASHTO PL-1 requirements, including a glulam timber rail with curb, a glulam timber rail without curb, and a steel rail. Figure 7 illustrates the steel rail that was crash-tested in accordance with AASHTO PL-2, and the glulam timber rail with curb that met NCHRP 350 TL-4 requirements.

The railings are adaptable to longitudinal stress-laminated, spike-laminated, nail-laminated, and glued laminated timber decks of any length. The AASHTO PL-1 glulam timber rail with curb is applicable to superstructures with a minimum deck thickness of 8 in., and all other railings included in the plans require a minimum deck thickness of 10.5 in. The timber components may be any species as long as they comply to specified bending and modulus of elasticity requirements. Each set of plans provides detailed information that illustrates rail layout, curb and rail splices, and fabrication details for all steel components. Full drawing sets are provided in customary U.S. and SI units of measure, and the testing procedures, results, and drawings have been approved by the Federal Highway Administration Federal-Aid and Design Office for use on Federal-Aid highway projects.

Concluding Remarks

The standard plans presented in this paper are intended to be a competitive choice for the replacement of deteriorating and construction of new short- to intermediate-span highway bridges. In developing standard plans for timber highway structures, engineers are immediately provided a wealth of clearly presented information, which familiarizes them with timber highway structures. By making wood design a less cumbersome process, timber highway structures become a viable choice.

Standard Plans for Southern Pine Bridges provides complete information for three superstructure types from the design phase through the construction phase. The designs are specifically for Southern Pine lumber or glulam. *Standard Plans for Timber Bridge Superstructure* presents designs for seven types of superstructures. The bridges may be constructed from any species that meets the material requirements defined by the deck geometry. Details of the structure are given, but a materials list, fabrication details, and detailed construction notes must be assembled by the user. These plans provide a starting point for the user who must adapt them to a specific site. *Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks* provides the configuration and layout of five crash-tested bridge railings and approach transition railings. They are easily adaptable for use on longitudinal wood decks and are critical to the acceptance of timber bridges.

These three sets of standard plans advance technology transfer regarding timber highway structures, helping engineers to become familiar with wood design and to consider it as an option when designing highway structures. These plans are available to the public at no charge and can be obtained by contacting the following:

Timber Bridge Information Resource Center
USDA Forest Service
NA State & Private Forestry
P.O. Box 4360
180 Canfield Street
Morgantown, WV 26505
(304) 285-1591 voice
(304) 285-6505 fax

Glulam Timber Rail with Curb

Steel Rail

Glulam Timber Rail without Curb

Figure 6-Railing configurations crash-tested to meet AASHTO PL-1 requirements included in *Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks.*

Figure 7-Railing configurations crash-tested to meet AASHTO PL-2 (steel rail) and NCHRP Report 350 TL-4 (glulam timber rail with curb) requirements included in *Plans for Crash-Tested Bridge Railings for Longitudinal Wood Decks.*

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Standard Designs for Hardwood Glued-Laminated Highway Bridges

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Abstract

Standard plans and specifications for hardwood glued laminated highway bridges have been developed and published. The plans are based on recent research to identify laminating processes, resin systems, structural properties of efficient beam cross sections, and preservative treatment processes. The results of these efforts are summarized. The standard plans are based upon nationally recognized allowable strength design methodologies and are for HS-25 or IML-80 loads. The standard plans, which are for northern red oak, red maple, and yellow poplar bridges, include details for design and construction of highway bridge superstructures and substructures The standard plans and specifications are being revised. The status of efforts to incorporate design efficiencies suggested by ongoing research and to convert the standard plans to a load resistance factor design basis are also presented.

Keywords: Bridges, timber, wood, highway, hardwood, glued-laminated, glulam, standard plans, specifications, allowable strength design(ASD), load resistance factor design (LRFD),

Introduction and Background

Overview

Researchers at The Pennsylvania State University (PSU), in cooperation with the Pennsylvania Department of Transportation (PennDOT) and the engineering firm of Gwin, Dobson and Foreman (GDF) have developed standard plans for hardwood glued-laminated (glulam) highway bridges. The overall goals of the project were to develop or identify the technology for more efficient use of temperate hardwoods in glulam bridge construction and to produce a user friendly set of standard plans for use by consulting engineers, highway and transportation department engineers, and local governmental engineers. Also the standard designs were to specify bridges suitable for normal highway use with no special load restrictions.

Before bridge standards could be developed, it was necessary to develop the technology base for manufacture and use of efficient hardwood glulam beams and deck panels. This required identification of candidate species for the bridges, identification of the resin systems to be used, identification and refinement of the preservative treatment for the bridge members, and development or verification of the structural properties of efficient hardwood glulam beams. Only with these technologies in place could the first generation standard plans be developed.

A second generation of the standard plans is currentlv under development. These plans ‘will include the results of more recent research to improve the efficiencies of the girder and deck panel designs. The second generation plans will also convert the designs

from an allowable strength design (ASD) basis to a load resistance factor design (LRFD) basis, as well as convert the standards to metric units.

The goals of this paper are to summarize the technological developments leading to completion of the standard plans and to outline the features of the ASD-based Standard Plans for Hardwood Glued Laminated Timber Highway Bridges (BLC-560). A further goal is to summarize the progress to date on the conversion to LRFD-based standard plans and to present preliminary comparisons of the outcomes of ASD- versus LRFD-based outcomes for hardwood glulam bridge girders.

Accomplishments To-Date

Species Selection -- Results of a literature survey of the availability, strength, bondability and treatability of temperate zone hardwoods common to Pennsylvania and the Northeastern United States were reported by Manbeck et al. (1994a). Red oak and red maple were identified as the most abundant species in Pennsylvania forests (18% and 12%, respectively) and yellow poplar ranked tenth (4%). However, yellow poplar is an abundant species in states just south of Pennsylvania. They reported that, of the ten most abundant hardwoods in the region and based on treatability and bondability, red oak and red maple were the least difficult to process as glulam timbers. Yellow poplar ranked one category lower in overall processing difficulty due in large part to relative difficulty in treatability. In addition, red oak and red maple both have very good to excellent structural characteristics (Wood Handbook, 1987; NDS, 1991). Thus, red oak, red maple, and yellow poplar were selected as the species for development of hardwood glulam bridge standards.

Resin System - Room temperature cured phenol-resorcinol formaldehyde (PRF) and resorcinolic formaldehyde resin systems were identified as suitable resin systems for bonding the candidate hardwood species. Glueline shear strengths and delamination characteristics for several room temperature cure and elevated temperature cure resorcinolic and PRF adhesives were found adequate for structural bonding (Manbeck et al., 1994a; Labosky et al., 1993). Several PRF resins were recommended for hardwood glulam manufacture because of their superior performance with hardwoods. Also, melamine was found satisfactory for finger joint fabrication of the three species (Janowiak et al., 1993).

Laminating Procedures - Manbeck et al. (1994a) presented the results of a study to identify the laminating procedures (e.g., surface quality and clamping pressures) necessary for satisfactory glueline performance for the three hardwood species. They concluded: (1) All three species can be end-jointed using any type of structural finger-joint assembly or other end connection provided minimum end joint criteria are met; (2) Planar feed rates should be reduced to assure 14 or greater knife cuts per inch surface dressing-it is preferable that red oak be processed with 16 to 20 knife cuts per inch; (3) Girders should be manufactured with plainsawn lumber; (4) Clamping pressures should be between 175 and 225 psi for red oak, 150 to 200 psi for red maple and 125 to 150 psi for yellow poplar, depending upon surface quality; and (5) Clamping pressure should be maintained for at least ten hours at 70°F.

Preservative Treatment- Oil borne preservatives are recommended for hardwoods by AWPA C14 and AWPA C28 for bridge applications (AWPA, 1995). Creosote was selected as the preservative treatment for the bridge standards because it is readily available commercially and is cost effective (Manbeck et al., 1994a). Blankenhorn et al. (1996) and Baileys et al. (1994) reported that required retention levels of creosote (12 pcf) and penetrations can be achieved in red oak, red maple and yellow poplar by modifying standard treatment processes. Blankenhorn, et al. (1996) identified the treating processes and also demonstrated that the treating process does not negatively affect the glueline strength. They also identified post treatment processes which reduce bleeding from the treated elements. Manbeck et al. (1995) reported that post fabrication creosote treatment of glulam beams from the three hardwood species to approximately 12 pcf retention did not significantly reduce flexural strength or stiffness.

Structural Characteristics- The structural characteristics of hardwood glulam beams manufactured from red maple were reported by Manbeck et al. (1993) Manbeck et al. (1994b) and Janowiak et al. (1995). Compression strengths for red maple and red oak were reported by Janowiak et al. (1994). The flexural performance of yellow poplar and red oak were reported by Moody et al. (1993) and Shedlauskas et al. (1996), respectively. The outcome of these studies was that it is feasible to manufacture glulam sections in each of the three species which: (1) Are efficient in that lower quality lumber (No. 2 or No. 3 VSR) can be used in the inner 50% of the cross section with higher quality lumber used only in the outer laminations; and (2) Have flexural strength of 2400 psi and flexural stiffness of 1,800,000 psi. These studies also demonstrated that ASTM D3737 (ASTM, 1993) calculation procedures satisfactorily predict the flexural properties of hardwood glulam and that the volume effect on the strength properties of the hardwood glulam beams was similar to that for specified in the NDS (1991) for softwood glulam beams. Consequently, the American Institute of Timber Construction (AITC) has recently revised AITC 119-96 (AITC, 1996) to include hardwood glulam beam design and manufacture with cross

sections which have laminations of different grades. Prior editions of AITC 119 (AITC, 119-85) required uniform quality laminations throughout the cross section. The study by Janowiak et al. (1995) also demonstrated the suitability of fabricating red maple glulam beams with non-edge glued combination laminations. This development is significant because it allows beams wider than 5.125 inches to be manufactured from nominal 2 x 4 and nominal 2 x 6 material. Limiting required laminating stock to maximum widths of 6 inches (nominal) is a key economical factor for hardwood glulam fabrication.

Connector Performance- The static and cyclic (up to 1 million cycles) load-slip behavior of lag bolts for hardwood glulam bridge applications were determined and reported by Thomforde (1995), Janowiak et al. (1996) and Witmer (1996). The load slip behaviors were subsequently used to model and predict the live load distribution for the hardwood decks and to predict the degree of partial composite action between the deck and the glulam girders. Witmer (1996) reported between 8% and 12% increase in the flexural stiffness of hardwood glulam girders when lag bolt- or Weyco clip-connected to the glulam deck panels. Load-slip behavior of lag-connectors degraded with number of load cycles. However, the residual strength of 75% of the specimens after 1 million cycles still exceeded 1.6 times NDS design values for lag-screws.

Standard Plans- The standard plans for hardwood glulam timber bridges for clear spans between 18 and 90 ft. were completed and published in September, 1994 (PennDOT, 1994). Also, standard specifications for hardwood glulam bridge applications were developed and published in PennDOT Publication 408 (PennDOT, 1994). The standard plans have been used to design and fabricate several bridge projects in Pennsylvania (Manbeck et al., 1996). The standard plans are ASD-based and are described in more detail in the next section of this paper. The standard plans are currently being revised to incorporate results of research completed since publication of the first edition and to convert them to an LRFD-basis and to metric units. The ASD-versus LRFD-outcome comparisons will be completed by December, 1996 and the LRFD and metric version of the standard plans will be published in July, 1997. A summary of the ASD vs. LRFD comparisons is presented in a subsequent section of this paper.

ASD-Based Standard Designs (BLC-560 Series)

Overview

The BLC-560 Series "Standards for Hardwood Glulam Bridge Design" (PennDOT, 1994) were developed cooperatively for PennDOT by researchers at The Pennsylvania State University and by personnel at the engineering firm of Gwin, Dobson and Foreman af Altoona, Pa. Design standards are included for glulam bridge clear spans ranging from 18 to 90 ft. Plans and specifications for longitudinal glulam panel bridges are provided for simple spans from 18 to 22 ft in each of the three hardwood species (red oak, red maple, and yellow poplar) and for longitudinal glulam girder-transverse glulam deck bridges for simple spans from 18 to 90 ft for each of the three hardwood species. The standard plans include general design criteria, superstructure designs for both bridge types, substructure designs for both bridge types and a design example. The 38 sheets contain all the details necessary for design, specification and assembly of the bridge substructure and superstructure.

Design Conditions

Wood- The BLC-560 Series standard plans are based solely on ASD methodologies and were developed in accordance with the AASHTO Bridge Design Specifications (AASHTO, 1992). Allowable design stresses for the three hardwood species were estimated from the results of the Penn State research efforts cited earlier in this paper. Allowable values are summarized in Table 1. Allowable flexural strength and stiffnesses for red maple and yellow poplar are based directly upon the research results of Manbeck et al., (1993), Manbeck et al. (1994) and Moody et al. (1993). Allowable flexural stress and stiffness for northern red oak are for combination A layups as specified in AITC 119-85 and in the NDS (1991). The research by Shedlauskas et al. (1995) which defined more efficient layups was incomplete at the time of publication of the BLC-560 plans. The shear strengths and the bearing strengths are estimates based on values published in the NDS (1991) for similar species and lamination layups. Estimation was necessary for these parameters because AITC had not yet revised AITC 119-85 at the time the plans were published. The design values in the revised version of AITC 119-96 are summarized in Table 2. The allowable design values for girders ate for the cross section layups described in Tables 3 and 4 for the first generation BLC-560 plans and for AITC 119-96, respectively. The allowable design values for the transverse deck and longitudinal deck panels are for lamination layups using No. 2 VSR lumber. Wet use conditions were used for design of all wood components in the bridge. Live load deflections are limited to span/500 for the superstructure. All wood is specified as creosote treated to retention levels of 12 pcf.

Table 1-Allowable design values for glulam elements in the BLC-560 ASD bridge designs (PennDOT, 1994)[a]

Bridge Element	Design Value (psi)	Northern Red Oak (NRO)	Red Maple (RM)	Yellow Poplar (YP)	Moisture Coefficient (Cm)
Transverse[b] Deck Panels	Bending Stress (F_{by})	1800	1800	1400	0.800
	Shear Stress (F_v)	215	205	145	0.875
	Modulus of Elasticity (E_y)	1.6e6	1.8e6	1.4e6	0.833
Longitudinal[c] Deck Panels	Bending Stress (F_{by})	1700	1700	1300	0.800
	Shear Stress (F_{vy})	215	205	145	0.875
	Bearing Stress (F_{cp})	885	615	420	0.530
	Modulus of Elasticity (E_y)	1.6e6	1.8e6	1.4e6	0.833
Longitudinal[d] Girders	Bending Stress (F_{bx})	2400	2400	2400	0.800
	Shear Stress (F_v)	215	205	145	0.875
	Bearing Stress (F_{cp})	885	615	420	0.530
	Modulus of Elasticity (E_x)	1.8e6	1.8e6	1.8e6	0.833

[a]Dry use, normal load duration and single piece laminations.
[b]Values for uniformly distributed No. 2 or better VSR laminations and for decks up to 5.125 in.
[c]Values for uniformly distributed No. 2 or better VSR laminations and for depths up to 12 in.
[d]Values for red oak are for Combination A lamination layups for Northern red oak (AITC, 119-85); values for red maple and yellow poplar are for 2400f-1.8E lamination layups. Lamination layups details given in Table 3.

Table 2-Allowable design values published in AITC 119-96 (AITC, 1996).[a]

Bridge Element	Design Value (psi)	Northern Red Oak (NRO)	Red Maple (RM)	Yellow Poplar (YP)	Moisture Coefficient
Transverse and Longitudinal Deck Panels[b]	Bending Stress (F_{by})[c]	1700	1450	1200	0.800
	Shear Stress (F_{vy})	175	160	135	0.875
	Bearing Stress (F_{cp})	835	590	405	0.530
	Modulus of Elasticity (E_y)	1.5e6	1.3e6	1.2e6	0.833
Longitudinal[d] Girders	Bending Stress (F_{bx})	2400	2400	2400	0.800
	Shear Stress (F_v)	235	220	155	0.875
	Bearing Stress (F_{cp})	1075	895	590	0.530
	Modulus of Elasticity (E_x)	1.8e6	1.8e6	1.4e6	0.833

[a]Values for dry use, normal load duration and single piece laminations.
[b]Panels fabricated with N2 laminations; combination symbol H2, H6, and H10 for NRO, RM and YP, respectively (AITC 119-96).
[c]F_{by} values for panels 12 in. deep; multiply by 1 .10 for 5.125 in deep transverse deck panel application; reduce by $(12/d)^{1/9}$ for longitudinal deck panels deeper than 12 inches.
[d]Values for red oak for 24F-E3 lamination layup; values for red maple for 24F-E4 lamination layup; values for yellow poplar for 24F-E3 lamination layup. Lamination layups shown in Table 4. (AITC 119-96).

Table 3-Lamination layups for girders used in BLC-560 standard plans[a]

Species	Combination Symbol	Outer[b] Tension Zone	Inner[b] Tension Zone	Core[b]	Inner Compression[b] Zone	Outer Compression[b] Zone
Northern Red Oak	A[c]	5% 302-24[d]	-	90% SG<1:8; KS<0.1W	–	5% 302-24
Red Maple	24F-1.8E	10% 2.0E6	15% 1.8E5	50% No. 2 VSR	15% 1.8E3	10% 2.0E3
Yellow Poplar	24F-1.8E	10% 2.0E6	15% 1.8E3	50% No. 2 VSR	15% 1.8E3	10% 2.0E3

[a]For dry use and normal load duration.
[b]Percent of cross section depth and lamination grade; 2.0E6 indicates MOE=1.8e6 and edge knot limited to 1/6 lamination face width.
[c]Combination A as defined by AITC 119-85. Maximum slope of grain (SG) = 1:8; maximum knot size (KS) ≤ 0.1 x lamination width (W).
[d]AITC tension lamination grade.

Table 4-Lamination layups for 24F-E1.8 girders in AITC 119-96 (AITC, 1996).[a]

Species	Combination Symbol	Tension[b] Lamination	Outer[c] Tension Zone	Inner[c] Tension Zone	Core[c] Zone	Inner[c] Compression Zone	Outer[c] Compession Zone
Red Oak[d]	24f-ES	302-24	10% 10E6	15% 1.8E3	No. 2 VSR	15% 1.8E3	10% 2.0E3
Red Maple[d]	24f-E4	302-24	10% 20E6	15% 1.8E3	No. 2 VSR	15% 1.8E3	10% 2.0E3
Yellow Poplar	24f-E3	302-26	10% 2.0E6	15% 1.8E3	No. 2 VSR	15% 1.8E3	10% 2.0E3

[a]For dry use and normal load duration.
[b]AITC tension lamination grade.
[c]Percent of cross section depth and lamination grade; 2.0E6 indicates MOE=1.06e6 and edge knot limited to 116 the face width of lamination.
[d]For members greater than 15 in. deep.
[e]For members greater than 22.5 in deep.

Loads- Design loads were the controlling one of HS-25 or IML-80 for all bridge components. Wheel load distribution factors for girder moments were s/4 for 4 in (nominal decks) and s/5 for 6 in (nominal) decks. Live load distribution factors for deflections were computed as the ratio of the number of lanes to the number of beams. Wheel live load distribution width for longitudinal panel superstructures comply with Section 3.25.3 of AASHTO (1992).

Deck panels- Deck panels for transverse decks were restricted to 5.125 in thickness. Transverse deck panels were all 3 or 4 ft wide and the full width of the bridge. All deck panels are dowel connected with 1.25 in. diameter steel dowels equally spaced at a maximum of 10 in. on center. A 0.25 to 0.375 in. spacing is specified between panels. Longitudinal deck glulam bridges are constructed with 12 to 15 in. thick panels approximately 44 to 48 in. wide. Panels are interconnected with 1 in. diameter A307 steel dowels

Table 5-Typical longitudinal girder sizes in BLC-560 standard plan for hardwood glued-laminated highway bridges (PennDOT 1994).

Species	Simple Span (ft)	Girder Spacing (in)	Curb to Curb Width (ft)	Girder Width (in)	Girder Depth (in)	Bearing Extension (--)
Northern Red Oak[a]	18	29	32	5.125	28.5	N
	30	29	32	5.125	34.5	N
	40	29	32	5.125	42.0	N
	60	29	32	6.750	55.5	N
	80	29	32	6.750	73.5	N
	90	29	32	6.750	84.0	N
Red Maple[b]	18	29	32	5.125	30.0	N
	30	29	32	5.125	34.5	N
	40	29	32	5.125	43.5	Y
	60	29	32	6.750	55.5	N
	80	29	32	6.750	73.5	N
	90	29	32	6.750	82.5	N
Yellow Poplar[b]	18	29	32	6.750	31.5	N
	20	29	32	6.750	33.0	N
	30	29	32	6.750	37.5	Y
	40	29	32	6.750	43.5	Y

[a]Combination A lamination layup (Table 3)
[b]24F-1.8E lamination layups (Table 2)

spaced 2 ft. on center in red oak and red maple and spaced 1 ft on center in yellow poplar. A6x6 transverse glulam spreader beam is also specified at the bridge centerline of longitudinal panel bridges.

Girders- Typical longitudinal girder sizes for each of the three species for selected spans are summarized in Table 5. Girder sizes are specified for clear spans up to 90 ft. for both red oak and red maple. Girder sizes are specified only up to 40 ft. clear spans for yellow poplar because depths became excessive and because beam widths greater than 5.125 in. are not economically feasible for hardwood glulam fabricated with one piece laminations. Yellow poplar required 8.75 in. wide girders to yield reasonable beam depths at spans greater than 40 ft. Therefore, spans greater than 40 ft. were excluded from the design tables. Beam layups are restricted to laminations consisting of one piece of lumber in standard beam widths of 5.125 in. and 6.750 in. Beam widths were restricted to 5.125 in. in all cases except where beam depths were large enough to present potential hydraulic opening difficulties at typical bridge sites. All girder spacings were approximately 2.5 ft. Girder depths ranged from 30 in. to 43.5 in. to 82.5 in. for red maple spans of 18 ft., 40 ft. and 90 ft., respectively. Corresponding depths for red oak girders were 28.5 in., 42 in. and 84 in., respectively. Girder depths for yellow poplar ranged from 31.5 in. to 43.5 in. for clear spans of 18 ft. and 40 ft, respectively. Significantly, all the yellow poplar girders were 6.75 in. wide. All the girders were designed assuming no composite action between the girder and connected deck. Most girder designs were controlled by beam moment; a few at the higher spans were controlled by the deflection criterion. The narrow width of the girders also created some bearing area difficulties at longer yellow poplar spans (6.75 in. wide beams) and at intermediate red maple spans (Between 35 ft and 50 ft.). Special bearing area extensions are specified for these cases. The standard plans also include H-, HS, and IML-80 load ratings for each girder in the design tables.

Connections- All deck to girder connections are galvanized 0.75 in. diameter lag bolts spaced 12 inches on center along each girder. The girders are fastened to the bearing pads with 0.875 in. diameter A325 galvanized steel bolts.

Geometry- The standard plans are detailed for right skews between 45 and 90 degrees and for left skews between 90 and 135 degrees. Details are provided for curb to curb widths of 24 ft., 28 ft. and 32 ft.

Substructure and other details- The standard plans include details for substructures. Details for attachment of the superstructure to existing concrete abutments are included as are details for wood pile and steel pile abutments. Bridge rail details are also included. The rail details are based upon static analysis of the components. Details for cribbing wingwalls and gabion wingwalls are also included in the BLC-560 plans.

Reviews and approvals- The BLC-560 standard designs were reviewed by engineers at PennDOT, FHWA, and USDA Forest Products Laboratory (FPL). Final approval for the standard plans was given during in September, 1994 and have been available for distribution from PennDOT since January 1, 1995.

LRFD-Based Standard Designs (BLC-560M Series)

Major Changes

The major refinements incorporated into the second edition of the BLC-560 Series standard plans are: (1) Incorporation of composite action in the design of longitudinal girders for the girder-transverse deck bridges. Girder stiffness (EI) is increased by approximately 10% when the transverse deck panels are properly connected to the girders (Witmer, 1996); (2) Inclusion of wider (greater than 5.125 in.) multiple piece laminations (e.g., non edge glued 2 x 4's and 2x6's) in girders and longitudinal deck panels. Janowiak et al. (1995) demonstrated the satisfactory performance of such layups for hardwoods. Multiple piece laminations provide more design flexibility (i.e., wider beams and wider girder spacings and deeper longitudinal deck panels can now be specified with economically sized hardwood lumber (less than 5.125 in.) in the overall bridge design; (3) The base strength and stiffness values for the hardwood glulam members are updated to the values recently published in AITC 119-96 (AITC, 1996) (Table 2); (4) Clip connections are added as an alternative to lag bolts for connecting the transverse deck to the girders. Witmer (1996) and Janowiak et al. (1996) verified the adequacy of the clip connectors for hardwoods; (5) Inclusion of transverse deck panel designs for installation on new or existing longitudinal steel girders; (6) Conversion of the standard plans to metric units; and (7) Conversion of the design basis from an ASD to an LRFD format. Comparisons of design outcomes of hardwood bridge girder designs using ASD and LRFD formats are the focus of the remainder of this paper.

LRFD vs. ASD Comparisons

Four factors may contribute to differences between hardwood bridge girder designs using ASD methods prescribed in AASHTO (1992) and the LRFD methods prescribed by AASHTO (1994) and PennDOT. They are: (1) Addition of a live load impact factor (IM); (2) Small changes in the live load distribution factor (LLDF) from s/3048 to s/3000 (s = girder spacing in mm); (3) Changes in the live load specification from HS-25 or IML-80 to PHL-93 (PennDOT, 1995); and (4) The change in format from ASD to LRFD. The first step in our comparison was to evaluate the overall effect of the four factors.

The longitudinal girder designs published in the first edition of the hardwood glulam bridge designs (BLC-560) were the basis for comparison of the ASD and LRFD outcomes. Using ASD, the ratio of maximum design moment (based on service loads) to the allowable moment (M_{max}/M_{all}) was calculated (These were all near 1.0 since moment controlled most of the girder sizes in the BLC-560 designs.). The maximum design moment was based on the controlling one of HS-25 or IML-80 loads, no impact factor, and a LLDF of s/3048. For the same beam size, span and spacing, the ratio of the maximum factored moment to the ultimate moment resistance of the cross section (M_u/M_r) was calculated. The maximum factored moment for LRFD included an impact factor (IM) of 1.165, a LLDF of s/3000, a live load factor of 1.75, a component dead load factor of 1.25, and a wearing surface dead load factor of 1.50. Finally, the %-difference between the ratios for the LRFD and ASD analysis was computed using equation 1.

$$\%\text{-Diff.} = \frac{(M_u / M_r - M_{max} / M_{all})}{(M_{max} / M_{all})} 100 \qquad (1)$$

Similar computations were completed for both shear and bearing stresses. The only exception for shear calculations was substitution of shear capacity for moment capacity in equation 1. The only exceptions for the bearing calculations was substitution of end bearing loads for moments in equation 1.

The results of the preliminary comparisons for red oak girders for spans ranging from 18 to 90 ft. (5.5 to 27.4 m) are summarized in Table 6. The percentage difference between the ratio of LRFD to ASD moment capacities decreased from -4 % to -22 % as span increased from 18 to 90 ft. Thus, overall effect of the LRFD specification and PHL-93 loads is to reduce the girder section requirement. Furthermore, the reduction increases with girder span. Similarly, the percentage difference between the ratio of LRFD to ASD shear capacities decreased from -20 to -39 % as span increased from 18 to 90 ft. Thus, the overall effect of the LRFD specification and the PHL-93 loads is to reduce the girder section requirement. The reduction also increases with girder span. The percentage difference between the ratio of LRFD to ASD bearing load capacities ranged from -10.2 to -22.1% as span increased from 18 to 90 ft. This indicates that the overall effect of the LRFD specification and the PHL-93 loads is to reduce bearing requirements. The percentage reduction appears to increase with girder span.

Table 6. Comparison of design outcomes for northern oak glulam bridge girders using AASHTO (1992) ASD and HS-25 or IML-80 loads vs. AASHTO (1994) LRFD and PennDOT PHL-93 loads[a,b]

		Moment Capacity[c]			Shear Capacity[d]			Bearing Capacity[c]		
Span (ft)[e]	Beam Depth (in)	M_u/M_r	M_{max}/M_{all}	% Diff. (LRFD/ ASD)	V_u/V_r	V_{max}/V_{all}	% Diff. (LRFD/ ASD)	P_u/P_r	P_{max}/P_{all} DK	% Diff. (LRFD/ ASD)
18	22.5	0.92	0.95	-3.9	0.68	0.86	-20.2	0.51	0.57	-10.2
20	24.0	0.94	0.99	-5.0	0.66	0.83	-20.6	0.52	0.59	-12.1
25	28.5	0.94	1.00	-5.6	0.59	0.75	-21.6	0.55	0.64	-15.1
30	33.0	0.93	0.99	-6.0	0.54	0.70	-23.2	0.57	0.69	-17.2
35	37.5	0.93	0.98	-5.7	0.50	0.68	-26.7	0.59	0.73	-19.0
40	42.0	0.92	1.02	-10.1	0.46	0.63	-26.8	0.62	0.77	-20.0
45	48.0	0.86	0.97	-11.9	0.42	0.61	-31.3	0.64	0.81	-21. 0
50	52.5	0.86	0.99	-13.9	0.40	0.60	-32.8	0.67	0.84	-21.5
60[f]	55.5	0.82	1.00	-17.7	0.33	0.50	-35.2	0.55	0.70	-22.1
70[f]	64.5	0.80	1.00	-19.7	0.30	0.48	-36.9	0.59	0.76	-22.0
80[f]	73.5	0.80	1.01	-20.9	0.29	0.47	-37.8	0.64	0.82	-22.0
90[f]	84.0	0.77	0.99	-22.0	0.27	0.45	-39.2	0.69	0.88	-22.1

[a]All ASD Calculations are for HS-25 or IML-80 loads and impact factor = 1.0
[b]All LRFD calculations are for PHL-93 loads, impact factor = 1.165, LLDF=s/3000, live load factor=1.75, component dead load factor = 1.25 and wearing surface dead load factor=1.50
[c]LLDF=s/3048 for ASD calculations
[d]LLDF by section 13.6.5.2 of AASHTO (1992).
[e]All girder widths are 5.125 in. unless noted otherwise; all girders spaced 2.42 ft. o.c.
[f]Girder width-6.75 in.

Table 7. Change in unfactored live load moment, shear and bearing capacities of the BLC-560 Series northern red oak girders due to change in load specification from HS-25 or IML-80 to PHL-93.[a]

		%-Change[b,c]		
Span (ft)	Span (m)	Moment	Shear	Bearing
18	5.49	36.8	18.6	22.3
20	6.10	37.8	19.0	19.8
25	7.62	40.3	21 .o	15.8
30	9.14	42.8	22.3	12.4
35	10.67	45.5	17.9	11.6
40	12.19	43.2	15.0	10.6
45	13.72	38.4	13.9	10.0
50	15.24	35.5	13.6	10.1
60	18.29	32.9	13.7	11.5
70	21.34	32.5	14.9	13.9
80	24.38	33.3	17.8	16.6
90	27.43	35.2	20.8	18.9

[a]Impact factor of 1.165 included in PHL-93.
[b]LLDF=s/3000 for all cases
[c]Values represent the percentage change (+ means increase) in unfactored live load capacity due only to change in load specifications.

Work is currently underway to separate the effects of load specification changes and impact factors from the effects of the differences in ASD and LRFD calculation methods. The first step in the process is to separate the effect of the load specification (HS-25 or IML-80 vs. PHL-93) on the unfactored live load moment, shear and bearing capacities for the girders in the BLC-560 standard plans. The results of these calculations ate shown in Table 7. The results indicate that unfactored live load moment capacities of the northern red oak girders in the BLC-560 standards increase by 33 to 45 % when the load specification changes to PHL-93. Unfactored live load shear capacities also increased when the PHL-93 load specification was used (by 14 to 22 %). The unfactored live load bearing capacities increased by 10.0 to 22.3% when the PHL-93 load specification was used. The trends in the data in Table 7 indicate that much of the difference in moment, shear and bearing capacity ratios in Table 6 are load specification change related.

Work is continuing to separate and to interpret the effects of the several interacting factors on the girder design outcomes. After the effects are separated, then it will be clearer why the design outcomes are different and the basis for converting the BLC-560 ASD standard plans to an LRFD format will be identified. These analyses are ongoing. Final outcomes are anticipated by the end of 1996.

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Analysis, Design, Rating, and Drafting of Wood Bridge Superstructures

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Abstract

A program for engineering wood bridges is described. The program performs the analysis, design, and/or rating of wood bridge superstructures in accordance with the 15th Edition of the AASHTO Specification or the AASHTO-LRFD Specification. Bridges may be analyzed for design review and/or rating. In review/rating mode, the superstructure is analyzed and checked against all applicable specifications. In the design mode, minimal user input, such as span length, roadway width, bridge type, and materials are entered and the program reviews designs and issues details associated with standard plan sets. The appropriate design-specific information is included in a DXF format for inclusion into the final drawings. A Microsoft Windows-based user interface is used for the data definition. The data is passed to another module for engineering computations. The associated documentation is contained in both on-line and hard copy formats. The most commonly used bridge and deck systems are accommodated in either U.S. customary or SI units.

Keywords: bridge design, wood bridge, bridge design software

Introduction

Computer programs have been used as an aid for bridge design for many years. Since the introduction of inexpensive personal computers, access to design programs has expanded greatly within all levels of the design community. In addition to general structural analysis programs, many design programs have been developed to assist with specific design applications involving both bridge superstructures and substructures. With few exceptions, these programs have been targeted at bridges constructed of steel and concrete. Very few computer design programs have included wood as a design material option.

In order to facilitate the design of wood bridges, computerized design programs for wood bridge superstructures must be developed that are comparable to those currently available for steel and concrete bridges. Otherwise, designers may unjustifiably select steel or concrete on the basis of the design tools available. These programs are needed not only in the traditional allowable stress design format, but also in the newly-introduced load and resistance factor design format. This paper presents the development of Madero (Spanish for timber beam), a complete computer package for timber bridge superstructures. Program functionality and design are discussed, along

with graphical user interface features, analysis methods, and implementation examples.

Project Objectives

The main objective of the project is to develop a comprehensive computer program for the analysis, design, drafting, and rating of wood bridge superstructures. In developing this program, several intermediate objectives must be met. The first involves developing the analysis program required to define geometry and analyze simple- and continuous-span timber bridges. Such bridges are subjected to the loads prescribed in the *AASHTO 15th Edition of the Standard Specifcation for the Design of Highway Bridges* (AASHTO 1992), and in the *AASHTO First Edition of the LRFD Bridge Design Specifications* (AASHTO 1994). Hereafter, these are referred to as the ASD and LRFD specifications, respectively. Once the analysis routines are in place, the second step constitutes developing the design and review algorithms to perform the AASHTO checks, or where appropriate, design the section properties. The third intermediate objective is to develop the subroutines necessary to output possible material lay-ups and quantities. Finally, the procedures required to produce computer-aided-drafting (CAD) graphics must be developed so that common CAD systems can access the results.

Timber Bridge Superstructures

Descriptions

This project is concerned with timber bridge superstructures only. The five basic types are the beam, deck (slab), truss, arch, and suspension superstructures. This project is limited to beam and deck bridges. The beam and deck bridges may be further divided into the following categories: the stringer (beam) bridge, the longitudinal deck bridge, and the stress-laminated deck bridge. Even these three types can vary considerably in their use of different materials and geometries. To clarify the bridge configurations supported by this project, each specific type is described in the following sections.

Analysis Methods and Assumptions

Calculation of Load Effects for Beams - Analysis of beams is performed with the use of classical beam theory and the finite element method. Inherent assumptions are that behavior is linear elastic, plane cross sections remain plane, deformations are relatively small, beams are prismatic, and beam supports are infinitely rigid. First, a stiffness model is generated with ten beam elements per span, and influence functions for shear, moment, and translation are determined. These functions are used to evaluate the load effects due to the component and wearing surface uniform dead loads. Next, the appropriate design vehicles are used to determine the critical live load actions at tenth points of each span.

The stiffness method is well suited to beam analysis for several reasons. First, continuous spans are easily accommodated. The ability to analyze continuous spans is one of the justifications for developing Madero, since simple span analysis can be performed relatively quickly by hand. Another advantage is the flexibility of span end conditions. Boundary conditions are defaulted to pinned connections at all bridge supports, but the user has the option to modify them. Both translation and rotation can be released or fixed at any or all supports, and hinges (moment releases) can be placed at any tenth point along the spans. Lastly, the stiffness method allows for easy determination of critical load effects. Moment, shear, and translation are calculated for each tenth point and for each dead and live load. Because a critical combined loading effect could occur at any point along a given beam, all load effects for each tenth point are needed. Critical effects can then be determined by checking each load permutation.

Load Combinations - After the load effects due to each type of load are determined, the effects must be combined in a rational manner in order to investigate various load combinations and find the critical case. AASHTO-ASD, section 3.22, defines several *load groups* which are applicable to particular site conditions, loadings, or structure types. Of these, three groups commonly apply to timber bridges: Group I for standard highway loads equal to HS20 or greater, Group IA for loads less than HS20, and Group IB for overload vehicles. Each load group has a dead load factor, a live load factor, and a factor to adjust the allowable stress, so that the basic load combination equation can be written as

$$\textit{Factored Load Effect} = \beta_{dead}D + g\beta_{live}(IM\ L) \quad (1)$$

where:

β are the load factors,
g is the live load distribution factor,
IM is the impact factor,
D is the dead load effect, and
L is the live load effect.

AASHTO permits the impact load to be zero for all timber structures when using allowable stress design, so the impact factor is set to unity by default. As with the distribution factors, however, the program user has the ability to custom define any of the above factors, including impact, when it is deemed appropriate. A complete list of factors and their values can be found in the ASD specification (AASHTO 1992).

The LRFD specification defines several *limit states* which serve to control the combination of loads in much the same way as the ASD load groups. For timber bridges, there are again three limit states which generally apply: the Strength I Limit State for normal vehicular use, the Service I Limit State for deflection considerations, and the Strength II Limit State for special design or permit vehicles. In addition to live load factors, the LRFD method also includes factors for importance, ductility, redundancy, and multiple presence of live load. Standard loading also calls for the simultaneous application of the design truck and a uniformly distributed lane load, each with separate impact factors. The general format for the LRFD load combination equation is

$$\textit{Factored Load Effect} = \eta_{imp}\eta_{duc}\eta_{red} [\gamma_{dead}D + mg\gamma_{live}(IM_{truck}L_{truck} + IM_{lane}L_{lane})] \quad (2)$$

where:

η_{imp} is the importance factor,
η_{duc} is the ductility factor,
η_{red} is the redundancy factor,
γ_i are the load factors,
g is the live load distribution factor,
IM are the impact factors,
m is the multiple presence factor,
D is the dead load effect, and
L are the live load effects.

Again, all factors may be manually defined by the user, but standard AASHTO values are used by default. For a more complete list of factors and their values, the reader is referred to Section 3 of the LRFD specification (AASHTO 1994).

It was noted previously that Madero does not consider loads from sources other than structure weight and design vehicles. Evaluation of loads from pedestrians, earth pressure, and stream current may be required. Local conditions may also warrant the use of loads caused by snow, earthquakes, or other environmental effects.

Program Design and Architecture

Functional Requirements

As stated earlier, the main objective of this project is to develop a comprehensive computer program for the analysis, design, drafting and rating of wood bridge superstructures. In meeting this end, several program requirements must be considered to ensure a workable, maintainable, and user-friendly product. This section outlines the main functional requirements and describes their importance to the operation of the program.

Structural Analysis and Review - Structural analysis is a fundamental task for any bridge design software. Madero's analysis and review are performed by a FORTRAN program referred to as the *computational code* throughout this paper. In addition to the many required analysis procedures, which were previously described, Madero must have procedures for rating bridges. Calculation of design ratios and rating factors produce criteria for rating acceptable performance and making design decisions. Consideration must also be given to the ability of the analysis and review code to interact with the user. The main philosophy in developing analysis routines is that the program should automatically perform all required calculations per the appropriate specifications, yet allow considerable observation and control by the user. The following discussion centers around the review procedures and user control of the computational code.

The product of structural analysis and the load combination routines is a set of factored load effects. These load effects are converted to stresses and compared to factored resistances. (ASD factors are typically 1.0) This comparison yields a design ratio and a rating factor for each analysis point and load permutation. Design ratios are computed as

$$\textit{Ratio} = \textit{Resistance} / \textit{Total Load Effect} \quad (3)$$

so that a ratio greater than unity indicates acceptable performance for the limit state and loading investigated, and a value less than unity indicates unacceptable performance. The design ratios are calculated for shear stress, flexural stress, bearing stress, and deflection, and can be employed by the user to decide how to modify bridge components for bridge design.

Rating factors are computed as

$$RF = \frac{\text{Resistance - Dead Load Effect}}{\text{Live Load Effect}} \quad (4)$$

A rating factor greater than unity indicates bridge live load capacity greater than that for which the bridge was analyzed. A rating factor below unity reveals deficiency in live load capacity of the bridge. Rating factors are computed for shear, flexure, and bearing. Deflection is not included since bridges are generally not rated based on this service criterion. Critical ratios and rating factors are determined for a bridge and reported to the user. Considerable detail of output can be achieved if desired, including all computations and output of every step outlined above.

User control is necessary so that AASHTO assumptions or default values may be overridden if desired. In general, the user has two options for changing default data. First, the user may select data from a list of typical values, such as defining a species and grade to describe a particular material. Madero then automatically retrieves material properties, such as bending strength or modulus of elasticity, for that material. Alternatively, the user may wish to directly define all material properties. Other parameters such as load factors or distribution factors can be defined in a similar manner, but by default are automatically calculated per the appropriate AASHTO specification.

Observation refers to the user's ability to track the entire analysis process of the computer. Results from any computer program should be checked by the designer to ensure that the proper input has been defined and that all computations are being performed as expected. Madero gives the user the power of selective observation by organizing output into several different categories. For each calculation performed by Madero, accompanying output can be written that defines input, calculations, and results for that operation. References to specifications are given where appropriate, so that the user can quickly find the correct design criteria and compare hand calculations or estimates to the Madero output.

Specification Requirements - Because two AASHTO bridge specifications are current, both needed to be included in the development of Madero. The user may elect to use either the *AASHTO 15th Edition of the Standard Specification for the Design of Highway Bridges* (AASHTO 1992), or the *AASHTO First Edition of the LRFD Bridge Design Specifications* (AASHTO 1994). Requirements of both are described and contrasted later. It should be noted that both specifications are available in either a U. S. customary units version or a S. I. units version. The user has the option to use either system of units.

Input Requirements - The computational code receives input data by reading an ASCII input file. The user can create an input file two ways: directly with a text editor, or with the Windows-based user interface. The advantages of both methods are outlined in the following sections, along with a description of the help system.

Windows-based Program to Produce* ASCII *Input - Madero uses a Graphical User Interface (GUI) to define input data and produce an ASCII input file. A Windows-based GUI is desirable for several reasons. Standard Windows functions, such as editing file management, and access to help systems, have come to be expected in modem software. Use of these functions make Madero compatible with other Windows-based programs, and use a format that is familiar to most computer users. Windows also allows for a dynamic interface. For example, dialogs can assume default command parameters, or a tutorial can walk the user through a typical data definition session. From a development standpoint, a Windows-based GUI is attractive because other GUI's have already been developed for similar bridge engineering software. The C++ code used to develop the user interface for this project is in the public domain and is easily accessible to developers.

On-line Help - The help system is a Windows-based hyperdocument that puts all the information in the User's Guide on-line. In addition to providing instantaneous access to program documentation from the GUI, the help system also provides standard WinHelp features such as search, contents, and help history. Hyperlinks point to related topics and figures where appropriate. A special feature of Madero creates an easy way to reference input command definitions. When editing the command file in the GUI, the user can position the cursor within the desired command and press CTRL-Fl to bring up the help topic for that command. Details on the GUI and help system architecture are addressed later.

ASCII Standalone - Although the FORTRAN computational code could be called directly by the C++ user interface, Madero uses an ASCII input file as an intermediate step between them. The input file contains a series of commands and command parameters which Madero recognizes and interprets to

define its input data. There are several reasons for using such an input file. First of all, development of the input command language necessitates a well-documented interface that can be easily linked to other systems. That is, any GUI that can convert input to Madero's input language can communicate with Madero. This opens possibilities for linkage to future interfaces and also to other bridge software systems. Second, existing code sets a precedent for developing a GUI that produces ASCII input. Modifying existing code takes much less time than starting a new system from scratch. Lastly, ASCII input allows the user to run the computational code directly from DOS. This is advantageous if the user's system does not have Windows, or if a large number of input files are desired to be run with batch execution.

Output Requirements - Described previously, the user may selectively request output for certain analysis calculations. This philosophy is reflected throughout the entire output file generated by the computational code. A basic level of output is standard, with options to expand into greater detail. At a minimum, echoed input, a basic bridge description, and summary reports on member actions, stresses, and rating factors are included in the output file. However, if the user wishes, a detailed bridge description, dead load calculations, distribution factors, influence lines, timber adjustment factors, and detailed reports on member actions, stresses, and rating factors can be output as well. The user also controls which design locations are output. Each beam span, for example, is divided into ten analysis points; results for any or all of the analysis points can be output at the user's discretion.

Maintainability - Maintainability refers to the ease with which the program can be modified and updated in the future. In order for Madero to be a valuable design aid, it needs to operate according to the most recent AASHTO specifications. While every effort has been made to include the newest design requirements, these requirements will most certainly change as new research findings point to better design methods and specifications are periodically updated. An emphasis on modular design generally creates a more manageable piece of software.

An effort was made throughout the planning and programming processes to keep subroutines small and functions as general as possible. Such modularization has several benefits. First, and perhaps most importantly, the program is divided into definite steps so that the logic is readily apparent. A clear algorithm is necessary for the programmer to initially organize the program, and it makes for easy addition or modification as the code develops. For example, the procedures created for the ASD version made a basic framework that could be duplicated and edited to produce the LRFD routines. The final code is also more readable, so that for future modifications it can be more easily understood and subsequently edited. Another important concern is the adaptation of code from within Madero to other applications. The exclusive use of local variables makes each subroutine an independent unit, so that it may be extracted for use in another code. A program that performs timber design, for example, could take advantage of Madero's routines for handling material properties, timber adjustment factors, and associated input commands.

The philosophy of modular design applies to data structures equally as well as it does to subroutine structures. The program requires a relatively large amount of data to fully describe a bridge, including its geometric properties, material properties, loading properties, and analysis results. To efficiently transfer this data between subroutines, several large data structures were created. A data structure may conveniently be passed from one subroutine to another in the form of a single variable name, but the main advantage lies in the way each structure is defined. A given data structure definition appears only in one file, and that file is referenced by all subroutines which require the use of that particular data structure. When a new variable must be added, or when a data structure must be otherwise modified, only the single definition needs to be changed. All references to the definition are automatically updated.

Linkage to Other Rating and Design Systems - As stated earlier in the introduction to this paper, very few computer programs are available to assist in designing timber bridges. Large software systems, such as those being developed for comprehensive bridge rating, will require timber analysis as well as the more traditional concrete and steel. Therefore, due consideration should be given to the ease with which Madero can be linked to other rating and design software systems. The computational code can act as an independent analysis tool, given the proper format of input data. In fact, any external program could use Madero's analysis routines if its data were translated into the Madero command language for input. Output data from Madero could also be converted to the desired format.

On-line Tutorial **-** In order to aid the user in learning how the Madero GUI works, an on-line tutorial is accessible from the help menu. The tutorial steps through a typical stringer bridge rating example, allowing the observer to see exactly how to enter command parameters and perform other necessary functions from the GUI. The tutorial is a DemoShield®4 demonstration, and works by opening up Madero and running mouse macros that interact with the program just as a bridge designer would. Notes and explanations appear as the demonstration unfolds, and the observer is allowed to control the pace.

Overview of the Architecture

Madero consists of several subsystems designed to facilitate user interaction, structural analysis, and easy referencing of user documentation. The system has a global architecture, in that the subsystems fit together and relate to one another in a certain fashion. Each subsystem also has a local architecture which describes how it performs its main functions. Obviously there exist many levels of complexity, each of which could be thoroughly described and flowcharted, down to the last subroutine. This section does not intend such an exhaustive explanation. Instead, the general arrangement of the program is presented, so that the relationship between its functionality and its structure may become apparent.

Overall Architecture - From the broadest perspective, Madero is a linkage of several different major subsystems. As shown in Figure 1, the relationships among these subsystems are by no means linear. In fact, there are two-way communication channels and alternate paths between many modules. Figure 1 is divided into two major portions: the solid elements represent subsystems that are required for Madero to work, and the dotted elements demonstrate how an external program could link to Madero's computational code. For both, the solid arrows point along conventional paths, and the dashed arrows point to alternate routes that could be developed in the future.

The user has direct access to the GUI, tutorial, help system, and text editor. Two observations can be made by the relationships shown among these modules. First, note that the help system, tutorial, and GUI have direct access to each other, so that the user has the freedom to move back and forth between them with ease while defining the input data. Second,

Figure 1 - Overall architecture.

notice how both the GUI and the text editor point to the input file. These two paths show the two methods for creating an ASCII input file. From the input file, the path goes through the computational code and into the output file. A trip to the text editor views the output and completes the loop required for a single bridge analysis. The dashed arrow drawn from the GUI directly to the computational code skips over the input file. This represents a possible future path which passes an argument list of data structures from the C++ GUI to the FORTRAN computational code without the use of the Madero command language. The FORTRAN is in fact being directly called by the C++ GUI to create a seamless and more efficient execution of the computational code.

The external program links to the computational code in the same manner as the Madero modules: through the input and output files. As explained previously, the external database need only be translated into the Madero command language to be compatible with the computational code. Such a connection could be used to link Madero with other bridge rating and design systems, such as AASHTO Virtus™ or AASHTO Opis™. The dashed arrows show a direct link between the translators and the computational code. As was proposed with the Madero system, an external

system could bypass the input and output files if the appropriate data structures were defined and directly passed into the FORTRAN.

Figure 2 shows the major components of the graphical user interface and help system. Both systems take advantage of existing code or utilities to keep new software development to a minimum.

Figure 2 - GUI and help system architecture.

The GUI uses a system of dynamic links to create a complete environment for data input. Written in C++, the GUI is comprised of four main components: the application-specific code, application-specific resource, general dialog libraries, and Borland object windows library. The dialog libraries were developed under a previous project sponsored by the U. S. Department of Transportation, and contain code especially suited to dialog development. The Borland object windows library contributes general development tools. Particular to Madero are the resource file and C++ code written for this project, which were patterned after existing GUI code for similar bridge software.

The help system is a close companion of the GUI in that it documents the command language and provides a general guide for using the GUI. Data files for the help system were created on Microsoft Word and saved in rich text format (RTF). These files were then compiled into help file format (HLP) with the aid of the Microsoft help compiler. The GUI accesses the help system through Microsoft Windows Help, which is the standard for Windows-based applications. Since the Madero help files are in standard format, they can be opened from the help system in any Windows application.

Validation Procedures - To ensure the most error-free code possible, Madero has been subjected to many levels of testing throughout the software development. The computational code requires the most rigorous tests because of the numerous areas of coding where computations could deviate from those intended. Individual processes were checked for acceptable performance as they were developed. For example, the routines for loading material property data into data structures were tested by creating a special output file for debugging. Once adequate processes were established as passable, testing began on bridge rating for specific bridge types. An effort has been made to cover all program functions over the course of the entire verification problem set. Each problem was compared to a known solution so that results could be validated. All verification problems matched known solutions for the majority of computations. In a few instances some discrepancies were observed, but these were carefully explained, typically with reference to modifications in the AASHTO specifications.

The GUI and help systems were likewise tested for acceptable performance. In the GUI, the dialog for each command and its accompanying output string had to be verified as complying with the Madero command language. For the help system, all hyper links had to be checked to make sure paths were correct. These tests, assuming that every part of the system was checked, confirm that Madero is capable of bridge analysis through the GUI. However, much testing remains in the areas of user-friendliness and program robustness. The validation process is an ongoing one, and the program will not be in a release state until extensive beta testing has been performed by testers independent of the program development. Certainly many points brought up by beta testers, but not considered by the developers, will require modifications to the code in the future.

Approximate Transverse Deck Analysis for LRFD

Unlike the ASD specification, the LRFD specification requires transverse deck strips to be modeled as continuous beams. Two complications result. First, the critical positioning of the wheel loads is not readily

apparent. For a simple span, the critical moment from a single wheel load occurs when the load is placed at midspan. The same holds true for continuous spans. However, the single-wheel analysis case never occurs in continuous beam modeling because both wheel loads on an axle will always contribute to the load effect. In other words, no matter where a load is placed, it will affect the moment experienced by every part of the deck. Critical loading then, is a function of bridge and truck geometry and is best determined by trial and error. Second, each wheel load needs to be distributed across a 20-inch footprint. Where small spans are concerned, the effects of wheel load distribution over the footprint can be significant.

Development of Analysis Functions

The goal was to create approximate expressions for maximum and minimum moment and deflection that are applicable to geometries and loadings commonly encountered in stringer type timber bridges. To meet this end, several variational parameters were investigated, including number of supporting beams, deck span, and number of lanes loaded.

The approximate expressions for moment and deflection are applicable only to deck spans between 0.9 and 2.1 meters (typical for timber). In addition, the following geometric restrictions apply:

- the number of stringers is no less than 3,
- stringers are spaced evenly, and
- the overhanging part of the deck, measured from the centerline of the outside stringer to the face of the curb, is less than or equal to either (0.625 times the deck span minus 0.5 meters), or (0.3 meters), whichever is greater.

The overhang limitation is intended to keep the critical design section within the main deck spans and prevent the overhang from controlling. The criteria are adapted from USDOT qualifications for concrete decks (National Highway Institute 1995).

Each model consists of a deck strip supported by three or four beams and loaded with standard design truck axle weights, as illustrated by Figure 3. Rotational and translational restraints on the outside supports are removed to represent the overhang at each end of the deck. Live loads move across the deck strip, and load effects are computed at the tenth points of each span.

Figure 3 - Continuous deck strip analysis model

In order to create conservative analysis functions, mathematical expressions were developed that fit the results of the finite element analyses. The plots shown in Figures 4 and 5 summarize the critical moment and deflection values from the analyses. Results from either the 3- or 4- beam models are shown, whichever is more critical. Moment is considered at all points along the deck. Deflection, however, is checked only within the main deck spans and is ignored in the overhangs. Where the critical moment or deflection is negative, the absolute value is used. In other words, Figures 4 and 5 show the extreme load effects as a function of span, whether positive or negative, plotted on a positive scale.

Figure 4 - Critical moment for transverse timber decks - LRFD.

Figure 5 - Critical deflection for transverse timber decks - LRFD.

Patch Factor for Flexure

A *patch factor* is used to compensate for the difference between the effects of the distributed load and the assumed point loading in the development of the approximate analysis equations. The patch factor is based on the relationship between the moment due to a point load on a simple span and the moment due to a distributed load on a simple span. While this method is exact for simple spans, it is approximate for continuous spans. The patch factor is defined as

$$PF = 1 - \frac{b_t}{2s_s} \tag{5}$$

where

PF = patch factor,
b_t = tire width, and
s_s = stringer spacing

and it is applied to all moment calculations.

Equation 5 can be used in conjunction with Figures 4 and 5 to create a design aid for LRFD transverse timber deck design. Direct finite element analysis is avoided while still meeting the requirements of the LRFD specification. Applicability is limited to decks loaded with the AASHTO design truck that conform to the geometric restrictions described earlier. A detailed description of design aid development and discussion of the approximate methods can be found in (Thomas 1996).

Closing Remarks

The development of Madero has realized several important accomplishments. Most central to its purpose, the program is a working design tool for common types of timber bridges. Madero performs checks on critical shear, bending moment, bearing, deflection, and deck connections for simple- and continuous-span bridge components. The bridge designer has the flexibility to use ASD or LRFD design specifications in either U.S. customary or SI units. Another significant accomplishment is the program's ability to produce a generous amount of output upon request. Detailed output provides a means for the bridge designer to verify all calculations and understand any assumptions made by the program. As a final contribution, a set of design aids (described elsewhere, Thomas 1996) was created through the development of approximate analysis techniques for transverse decks. The continuous strip analysis required by the LRFD specification is more complicated than the simple-span analysis permitted in previous editions of the AASHTO specification. The design aids provide a quick and simple source for transverse deck analysis of standard design truck loads, while still meeting AASHTO requirements.

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Evaluation of New Creosote Formulations

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Abstract

This study compared two new formulations of creosote and one pigment-emulsified creosote (PEC) with a formulation of creosote that met requirements of the AWPA standard P1/P13 for creosote. Two softwood and two hardwood species were treated to four retention levels with each formulation. The four creosote formulations were evaluated by (1) soil-block test, (2) fungal cellar test, and (3) field test. This paper presents results from the soil-block tests and preliminary findings from the fungal cellar tests after 72 weeks. The field stakes have been exposed for only 1 year, thus these data are not available. Data from the fungal cellar tests show that softwoods are protected much better than hardwoods for all four formulations of creosote tested. The soil-block tests show comparable performance in softwood and hardwoods. No major difference between formulations was detected in the two laboratory tests.

Introduction

Creosote, or coal-tar creosote, has persisted since the earliest days of treating wood with preservatives. In the United States, creosote was first used for treating marine pilings in 1889. By the 1920s, it became the treatment of choice for the railroad industry and continues to be so today, During the late 1960s, high temperature creosote largely replaced low temperature creosote for timber preservation as a result of the decreasing gasification of bituminous coals, In 1982, low temperature creosote, conforming to Australian standard specification K-55 (AS 1965, Watkins and others 1983), ceased production in Australia.

One problem associated with the use of oil-type preservatives is the tendency to exude or "bleed" from some treated commodities, producing an oily or tar-covered surface that can cause handling problems. To minimize this exuding problem, laboratories, such as Koppers Industries Inc., USA and Commonwealth Scientific and Industrial Research Organization (CSIRO), Division of Chemical and Wood Technology, Melbourne, Australia, have developed changes in processing of coal tar, which produces distillates with fewer contaminants. This "clean distillate" is then used to formulate "clean creosote" as a preservative utilized in the treatment of utility poles.

Soil-block and fungal cellar test methods (accelerated field simulator) (Johnson and others 1982) are two laboratory procedures used to characterize the effectiveness of wood preservatives. The soil-block tests determine the minimum threshold level of the preservative that is necessary to inhibit decay by pure cultures of decay fungi. The fungal cellar test exposes treated wood to accelerated attack by mixtures of soil-borne fungi. The field stake tests are used to verify

service life of the new creosote formulations *in vivo.* Results from accelerated tests are indicative of field performance, but the correlation between laboratory and field results is still being investigated. Field stake testing is regarded as a critical, long-term evaluation that provides results most directly related to the performance of treated products in service.

The objective of this study was to compare two new formulations of creosote and one pigment-emulsified creosote (PEC) with a formulation of creosote that met requirements of AWPA standard (1995a) P1/P13 for creosote. Two softwood and two hardwood samples were treated to four retention levels with each formulation. The four creosote formulations were evaluated by (1) soil-block test (ASTM 1986), (2) fungal cellar test (AWPA 1995d), and (3) field test (AWPA 1995c). Results from the soil-block test and preliminary findings from the fungal cellar tests after 72 weeks are presented in this paper. However, stakes in the field test have been exposed for only 1 year, thus data are not available.

Material and Methods

Wood Species

The following wood species were used in this study:

Douglas-fir *(Pseudotsuga menziesii)*	Mixture of heartwood and sapwood from second-growth trees in the Pacific states of Oregon and Washington.
Pine *(Pinus* sp.)	Sapwood with 5-15 rings/ 25.4 mm (rings/in.) Wood was kiln dried without the use of antistain chemicals.
Red oak *(Quercus rubra)*	Red oak heartwood was predominantly selected to represent a dense, ring-porous hardwood. Wood was used as supplied.
Red maple *(Acer rubrum)*	Both heartwood and sapwood of northern red maple were used to represent a diffuse porous hardwood. Wood was visually selected for clear material.

Test Specimens

Evaluations of preservative efficacy were made using the following wood specimen sizes:

- Soil block: 19.05 mm (0.75 in.) cubes
- Fungal cellar: 3 by 19 by 150 mm (0.118 by 0.748 by 5.91 in)
- Field stakes, two sizes:
  19.05 by 19.05 by 457.2 mm (0.75 by 0.75 by 18 in) and
  25 by 50 by 457 mm (1 by 2 by 18 in.)

Preservatives

The following preservatives were used, which were either creosote or modifications thereof. These preservatives were selected because of the recognized contribution of creosote treatments to resist physical abrasion in addition to their resistance to colonization by biological organisms.

• Creosote (P1/P13)	Creosote meeting AWPA Standard P1/P13 was used as the reference preservative treatment
• Creosote A	New formulation being developed by Koppers Industries Inc., USA
• Creosote B	New formulation being developed by Koppers Industries Inc., USA
• P E C	Pigment Emulsified Creosote—A formulation developed and used in Australia (not available in USA)

Soil-Block Test

Two brown-rot fungi, *Postia placenta* (Fr.) M. Lars. et. Lomb. [MAD-698] and *Neolentinus lepideus* (Fr.:Fr:) Redhead and Ginns [MAD-534], and two white-rot fungi, *Trametes versicolor* L. ex Fr: Pilate [MAD-697] and *Irpex lacteus* (Fr.:Fr.) Fr. [HHB-7328 sp.], were used as test fungi. Test fungi were maintained on 2% malt agar. Both untreated and treated pine *(Pinus* spp.) and red maple *(Acer rubrum* L.) blocks were exposed to brown- and white-rot decay fungi in soil-block tests according to ASTM D1413-76 (1986). Five replications of treated blocks and 10 replications of untreated controls were exposed to decay fungi for 12 weeks. Blocks were not weathered prior to exposure to test fungi.

Four concentrations of each preservative were evaluated. To maintain some consistency among species in this study, treating solutions were prepared in a series of concentrations of active ingredients that was used with all wood species. This produces different actual retention levels in the various species, but this pattern provides a good overall evaluation in multiple species.

Several formulations of PEC have been developed in Australia. We utilized PEC 30W. This is an anionic emulsion that consists of 30% water and 3.5% finely dispersed micronized titanium dioxide pigment. The preservative is manufactured as an oil-in-water emulsion, which is inverted to a highly stable water-in-oil emulsion before use. The same four treatment levels were used.

Four concentrations of treating solution using toluene as the diluent were used for each U.S. creosote formulation to obtain four different active ingredient levels: 65%, 30%, 15%, 7.5%. Immediately after treatment, the surface of each specimen was wiped clean to removed excess chemical and weighed. Retention levels after treatment of each specimen were calculated on the basis of weight gained and were confirmed through AWPA Standard A2-94 (1996).

Fungal Cellar

The fungal cellar evaluation is currently in progress at the USDA Forest Service, Forest Products Laboratory, in Madison, Wisconsin. Fifteen replicate stakes of each wood species were treated with each of the four retention levels of each creosote formulation. The stakes are 3 mm (thick, transverse) by 19 mm (radial direction) by 150 mm (parallel to the grain) (0.118 by 0.748 by 5.91 in.). Stakes are exposed in soil that is maintained at a moisture content of 50% to 70% of water holding capacity to promote growth of soft-rot fungi (Nicholas and others 1991). Soil beds are maintained in a controlled environment at 26°C and a relative humidity of 86% to 90%.

Prior to exposure in the fungal cellar, all treated and control specimens were vacuum impregnated with water. The stakes were grouped by treatment, subjected to vacuum (about 100 mmHg) for 30 minutes before being soaked for 2 hours in distilled water. The test specimens were then inserted vertically into the fungal cellar until the top end was level with the soil.

At 3-month intervals, for 18 months, wood specimens were removed from the fungal cellar (soil bed), cleaned with a brush to remove excess soil, and placed in water-tight plastic bags until evaluation for strength loss using the bending strength apparatus shown in Figure 1, Strength loss was determined as described by

Figure l—Bending apparatus with computer interface

Figure 2—Orientation of specimen on bending apparatus

Crawford (1994). Care was taken to ensure that the test specimen was oriented in the same way during subsequent strength evaluations (Fig. 2).

Initial load measurements were made, and modulus of elasticity (MOE) was calculated using the following:

$$\mathrm{MOE} = \frac{L^3}{4}\,\frac{P}{bh^3D} \qquad (1)$$

where
MOE = modulus of elasticity (kPa),
L = constant span of 126 mm,
b = constant specimen width of 19 mm,
h = constant specimen thickness of 3 mm,
D = constant specimen deflection of 2.50 mm, and
P = variable force to maintain constant deflection (g).

$$\mathrm{MOE} = 389.94(\mathrm{P}) \qquad (2)$$

A stake that demonstrated MOE losses greater than 60% of its original MOE was considered failed.

Results and Discussion

Soil-Block Test

Retention

The actual retention levels for individual formulations at their targeted concentrations (as calculated from

Table 1—Decay resistance of pine soil-blocks treated with PEC and three creosote formulations (12-week exposure).

				Weight loss (%)			
	Concentration	Retention		Brown rot		White rot	
Formulation	(% AI[a])	(pcf)	(kg/m^3)	*N. lepideus*	*G. trabeum*	*T. versicolor*	*L. lacteus*
Control		0	0	40.10	62.50	21.20	17.60
PEC[b]	7.5	3.09	49.52	6.56	2.35	0.34	0.95
	15	6.26	100.32	1.01	0.96	0.81	1.29
	30	13.44	215.38	1.23	1.06	1.50	1.38
	65	27.04	433.33	2.17	2.29	3.38	2.53
Control		0	0	40.10	62.50	21.20	17.60
P1/P13	7.5	2.81	45.03	15.00	7.10	0	0
	15	4.95	79.33	4.30	1.20	0	0
	30	7.34	117.63	0	0	0	0
	65	19.60	314.10	1.60	1.80	1.70	1.60
Control		0	0	40.10	62.50	21.20	17.60
Creosote A	7.5	2.10	33.65	19.60	14.10	0	0
	15	4.54	72.76	12.80	5.00	0	0
	30	9.29	148.88	0	0	0	0
	65	23.19	371.63	0	0	0	0
Control		0	0	40.10	62.50	21.20	17.60
Creosote B	7.5	2.09	33.49	15.50	11.90	0	0
	15	4.18	66.99	6.20	4.70	0	0
	30	9.07	145.35	0	0	0	0
	65	22.39	358.81	1.00	0	0	1.00

[a]AI is active ingredient.
[b]PEC is pigment emulsified creosote.

weight gain during treatment) differed slightly (Tables 1,2, Fig. 3) for both the pine and the red maple.

In both species, the PEC was more readily absorbed at the higher targeted concentrations than were the other formulations. In the pine at a targeted concentration of 7.5% active ingredient (AI), the retention for all four formulations was between about 33 to 50 kg/m^3 (2.06 to 3.12 pcf (pound per cubic foot)). At a targeted retention of 30% AI, the PEC had a retention level of 215 kg/m^3 (13.42 pcf), and the other three formulations had retention levels about half that of PEC.

In the red maple at a targeted concentration of 7.5% AI, the PEC and P1/Pl3 formulations absorbed twice as much as did the other creosote formulations, with retention levels of about 50 kg/m^3 (3.12 pcf) for PEC and P1/P13 and 25 kg/m^3 (1.56 pcf) for the creosotes. At the higher targeted concentration of 30 kg/m^3 (1.872 pcf), the PEC had a retention of about 240 kg/m^3 (14.98 pcf), and the three other formulations were about 110 to 120 kg/m^3 (6.86 to 7.48 pcf).

There does not appear to be any species differences in absorption of the individual formulations. The red maple retention levels were similar to the pine retention levels for any given formulation. The only exception appears to be with creosote B, where the pine had slightly higher retention levels of creosote B than did the red maple.

Weight Loss

Although some general trends were evident, the soil-block test did not distinguish major differences among the formulations in protecting pine sapwood (Figs. 4, 5). In a comparison of creosote A and B with creosote P1/P13, creosotes A and B were slightly less effective than P1/P13 against *G. trabeum* at the lowest retention level (Figs. 4, 5). At subthreshold levels, creosote A appears to be less effective than P1/P13 against *N. lepideus.*

The protection provided by PEC at low retention was better against all fungi than the reference preservative P1/P13 creosote and the other two creosote formulations (A and B). However, the increase in weight loss observed in blocks of red maple treated at increased retention levels of P1/P13 creosote is difficult to interpret.

Table 2—Decay resistance of red maple soil-blocks treated with PEC and three creosote formulations (12-week exposure).

				Weight loss (%)			
	Concentration	Retention		Brown rot		White rot	
Formulation	(% AI[a])	(pcf)	(kg/m^3)	*N. lepideus*	*G. trabeum*	*T. versicolor*	*L. lacteus*
Control		0	0	29.20	64.80	57.50	44.30
PEC[b]	7.5	3.25	52.08	0.10	0.06	0	0
	15	6.40	102.56	0.63	0.29	0.66	0.39
	30	15.26	244.55	1.30	1.33	1.90	1.33
	65	30.98	496.47	7.76	7.56	6.07	4.56
Control		0	0	29.20	64.80	57.50	44.30
P1/P13	7.5	2.78	44.55	1.40	3.80	6.00	0
	15	4.98	79.81	0	0	0	0
	30	7.50	120.19	0	0	0	0
	65	21.49	344.39	1.70	2.00	2.30	2.30
Control		0	0	29.20	64.80	57.50	44.30
Creosote A	7.5	1.67	26.76	10.70	9.30	20.08	0
	15	3.68	58.97	1.30	2.90	0	0
	30	8.40	134.62	0	0	0	0
	65	20.88	334.62	0	0	1.30	1.10
Control		0	0	29.20	64.80	57.50	44.30
Creosote B	7.5	1.46	23.40	11.20	7.30	27.30	10.90
	15	2.91	46.63	1.20	2.20	1.90	0
	30	6.96	111.54	0	0	0	0
	65	18.33	293.75	0	0	0	1.00

[a]AI is active ingredient.
[b]PEC is pigment emulsified creosote.

Fungal Cellar Test

Modulus of elasticity (MOE) results are reported after 72 weeks of exposure (Table 3). None of the creosote formulations appeared to prevent attack by soft-rot fungi in red oak or red maple as determined by reduced MOE. Decrease in MOE approached or exceeded 60% at all retention levels for all creosote formulations in those species. At all creosote retention levels, softwoods were protected better than hardwoods. As retention levels increased, the relative difference between hardwoods and softwoods increased.

After 72 weeks of exposure in the fungal cellar, there was no loss in MOE in the Douglas-fir stakes and a minimal loss in the pine stakes treated with 65% PEC (23 pcf). Both hardwood species showed approximately 50% loss in MOE at 65% PEC (19 to 23 pcf) after 72 weeks of exposure in the fungal cellar. Loss in MOE for other formulations in Douglas-fir ranged from about 30% to 60% at AI concentration of 7.5% (1.8 to 2.0 pcf) to 30% (7.6 to 9.7 pcf). In pine, all formulations had equivalent performance.

Retention Analysis

The treatability of all four wood species at a given concentration of active ingredient of the different formulations provided a range in retention levels of active ingredient in the treated wood that ranged from slightly more than 16 kg/m^3 to approximately 320 kg/m^3 (20 pcf) (Table 4). This range in retention of active ingredient spanned the targeted retention of 160 kg/m^3 (10 pcf), which is specified for oak ties (AWPA 1995b).

At higher concentrations, retention levels for creosote P1/P13 and A were approximately 20% less than that achieved with PEC and creosote B. Still, these retention levels were greater than 160 kg/m^3 (10 pcf).

Red oak had the lowest retention at all solution concentrations for all formulations. The PEC yielded the highest retentions as calculated by weight gain. Retention levels of the other three formulations tended to be relatively comparable for each concentration of treating solution, but with an occasional spurious result for a given wood species. As with the soil-block test, general trends indicate relative little difference in effectiveness between the four formulations.

Figure 3—Retention levels of four formulations of creosote in red maple soil blocks that were treated with solutions that had comparable concentrations of active ingredients.

Figure 4--Weight toss of pine soil blocks treated with four formulations of creosote and exposed to *G. traveum.*

Figure 5–Weight loss of red maple soil blocks treated with four formulations of creosote exposed to *G. trabeum.*

Table 3—Average modulus of elasticity loss at four active ingredient (AI) concentrations.

	Modulus of elasticity loss (%)					
	Concentration (% AI[a])	Creosote (P1/P13)	Creosote A	Creosote B	PEC[a]	Control
Pine	0					96.54
	7.5	59.99	60.46	64.04	65.97	
	15	44.21	56.04	54.25	57.62	
	30	39.07	43.68	38.33	40.01	
	65	9.40	22.73	12.89	8.05	
Douglas-fir	0					81.02
	7.5	55.55	57.74	60.71	69.48	
	15	38.88	42.40	42.77	69.43	
	30	29.98	26.74	28.56	49.88	
	65	7.90	13.17	11.94	0	
Red oak	0					89.37
	7.5	85.16	86.07	91.74	83.07	
	15	78.43	82.47	79.89	79.10	
	30	70.68	73.92	76.06	73.02	
	65	52.58	62.67	59.43	53.91	
Red maple	0					97.70
	7.5	86.65	89.22	84.47	87.91	
	15	80.66	84.18	80.43	83.29	
	30	62.69	75.45	69.34	79.63	
	65	46.70	51.08	49.23	49.95	

[a]PEC is pigment emulsified creosote.

Table 4—Calculated retention at different concentrations of four creosote formulations in fungal cellar test.

	Average	Pine		Douglas-fir		Red oak		Red maple	
Formulation	(% AI[a])	(pcf)	(kg/m^3)	(pcf)	(kg/m^3)	(pcf)	(kg/m^3)	(pcf)	(kg/m^3)
PEC[b]	7.5	2.99	47.92	3.33	53.37	2.94	47.12	3.12	50.00
	15	5.76	92.31	5.76	92.31	5.55	88.94	5.36	85.90
	30	12.98	208.01	12.99	208.17	12.68	203.21	12.64	202.56
	65	27.75	444.71	23.11	370.35	19.06	305.45	23.35	374.20
P1/P13 Creosote	7.5	2.07	33.17	1.99	31.89	1.20	19.23	1.65	26.44
	15	4.15	66.51	4.09	65.54	2.45	39.26	3.52	56.41
	30	11.15	178.69	9.72	155.77	10.68	171.15	9.52	152.56
	65	20.60	330.13	20.55	329.33	14.24	228.21	18.12	290.38
Creosote A	7.5	1.98	31.73	1.79	28.69	1.17	18.75	1.63	26.12
	15	4.16	66.67	3.67	58.81	2.51	40.22	3.66	58.65
	30	8.90	142.63	8.50	136.22	5.47	87.66	7.96	127.56
	65	20.15	322.92	20.07	321.63	20.60	330.13	13.19	211.38
Creosote B	7.5	1.96	31.41	2.02	32.27	1.24	19.87	1.87	29.97
	15	4.04	64.74	1.94	31.09	2.43	38.94	1.89	30.29
	30	8.89	142.47	7.63	122.28	5.56	89.10	8.23	131.89
	65	21.35	342.15	20.52	328.85	13.28	212.82	20.24	324.36

[a]AI is active ingredient.
[b]PEC is pigment emulsified creosote.

Concluding Remarks

Results from the soil-block test indicate that products treated with any formulation in the test should have comparable durability. The tendency for formulations A and B to be slightly less effective than the P1/P13 formulation at the lowest retention level places more importance on quality control during treatment. With these two modified creosotes, low retention levels may not perform as well as low retention levels of P1/P13. Generally, there was little difference in the ability of the four creosote formulations to prevent decay at the three highest retention levels.

Results to date from the fungal cellar tests indicate the potential for poorer performance of treated hardwoods than has been observed in practice. The historical success of P1/P13-type creosotes in U.S. hardwoods at retention levels less than those tested in this study begs for a fundamental explanation of the cause of these results. The relative low retention of red oak in comparison with retention levels of other formulations per concentration of treating solution may somehow be related to the relatively poor performance of that wood species in the fungal cellar. Still, a calculated retention in excess of 160 kg/m^3 (10 pcf) was obtained with all formulations at the highest treatment concentration. Furthermore, red maple, which usually had higher retention levels than red oak for each treatment, also performed poorly in the fungal cellar.

The field stake test will be used to verify service life of the new creosote formulations. As previously noted, few data are available from the field stake test. Thus, we are unable to correlate field stake data with the fungal cellar data at this time.

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Integrated Efficacy Evaluations of New Preservatives in Alternative Wood Species

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Abstract

Through an integrated laboratory and field testing program, the durability of a variety of wood species, when treated with emerging preservatives, is being evaluated. Results from this testing program will provide a broad database on the potential of alternative wood species to be treated with diffent preservatives. It is well recognized that inorganic waterborne preservatives do not protect hardwoods as well as they protect softwoods, but the degree of difference has not been quantified. The relationships between accelerated laboratory tests and actual field performance have also not been quantitatively defined. 'Therefore, a series of linked, laboratory and field evaluations has been initiated that should provide the opportunity to judge the merits of diffrent experimental approaches to predict the ultimate field performance potential of new wood species by preservative combinations. This information will contribute to increasing the opportunities available for utilizing regional forest resources in U.S. transportation structures and strengthening the international trade of durable wood products. This paper presents an overview of the ongoing experimentation and the current status of this integrated research program

Introduction

One important component of any program that focuses on utilization of wood in U.S. transportation structures is the desire to stimulate local economies and enhance rural transportation systems through the use of locally available wood species. The emphasis on locally available species often requires consideration of wood species other than the primary commercially used wood species. Regionally distributed, underutilized hardwoods are of special interest in this program There is also a national need for a strong export trade. Preservative treatments, in concept, are being increasingly looked to as a technology that will generate additional opportunities for U.S. wood products in global markets. Both nationally (Wipf and others 1993) and internationally, development and testing of new wood preservatives that do not pose an environmental hazard are needed, Specific actions vary with individual political entities, but bans on one or more of the commonly used preservative systems are either appearing or being considered within several countries, including the United States.

The resolution of these needs presents a challenge. During the past two decades, waterborne preservatives have been used increasingly to treat softwoods. These preservatives enjoy a public perception of environmental preference over oilborne preservatives because of the lack of odor and the dry surface of the waterborne treated product. Indeed, waterborne preservatives are the preservative of choice when a high degree of human contact is involved. However, it is generally recognized that hardwoods are less protected from decay by waterborne preservatives than are softwoods. Furthermore, the commonly used waterborne preservative chromated copper arsenate (CCA) is now banned in Indonesia (Permadi 1995).

Hardwoods are commonly used for railroad ties, which are almost universally treated with creosote. However, many hardwoods that have a wide geographic distribution lack the density needed for ties, These hardwoods also lack the quality required for tine furniture and remain relatively underutilized in construction. Examples of such hardwoods in the United States are soft (red) maple that is omnipresent east of the Mississippi; silver maple which is common through the Midwest, Mid-Atlantic, and Northeastern regions; cottonwood which is prevalent along river bottoms throughout the West, North Central, and Southern regions; and yellow poplar which is abundant in the Mid-Atlantic states and Southeast region. Of these hardwoods, considerable engineering research is being directed towards development of designs and building components that will permit enhanced utilization of red maple and cottonwood, in particular. All these hardwoods lack natural durability and vary in treatability characteristics.

Another challenge in developing and testing new wood preservatives is that protocols for acceptance to standards are structured to provide comparison of the preservative chemical, not to characterize the variation within a wood species or between species of wood (AWPA 1994). Southern pine sapwood is the material that is usually utilized in these evaluations.

Through an integrated laboratory and field testing program, the durability of a variety of wood species, when treated with emerging preservatives, is being evaluated. The purpose of this paper is to present an overview of the ongoing experimentation and the current status of this integrated research program.

Materials And Experimental Methods

Linkage Through Experimental Design

This research is supported through several funding programs,[1] thus specific objectives of individual studies differ. However, all individual studies being done at various institutions and geographic locations are linked by the common objective to evaluate accelerated methods for predicting the performance life of preservative-treated wood. Either a component of the experimental design or the complete design of field trials conducted at all locations is repeated in field stake plots at the USDA Forest Service, Harrison Experimental Forest, in southern Mississippi. Furthermore, in either separate experiments or by means of subsampling, replicate stakes, which were treated at the Forest Products Laboratory (FPL) for field exposure in tropical and temperate locations, were also used to conduct parallel soil-block studies at FPL. This gives us a prime opportunity to explore whether results from laboratory soil-block studies and those from field plots in tropical locations fore cast relative performance of treated-wood products in temperate climates.

Whenever possible, a minimum of 20 replicate units/species by preservative by retention combination was used in laboratory-scale, simulated ecosystems and in field studies that permitted repeated measurement of the same unit. This was done to permit development of failure curves for small populations of the same wood by preservative by retention combination when evaluated by different methods or in different environmental settings. In a data analysis from prior FPL field plots with stakes that were nominally 50 by 100 mm in cross section, we estimated that 20 was the minimum number of replicate units needed to characterize population trends. This design protocol had some impact on limiting the number of field sites when exposures had to be made on a fixed budget, and the protocol did restrict the number of combinations that could be evaluated in soil beds that have a finite physical capacity. In the end, the experimental design will allow a detailed comparison of performance histories of treated-wood species in tropical and temperate plots. These comparisons will hopefully enable researchers to determine whether tropical environments yield the same relative performance among experimental combinations and whether tropical exposures yield any quantitative indication of potential durability of treated products.

Selection Of Wood Species

Several wood species have potential application in transportation structures. To gain information on possible interactive effects among wood species and preservatives, three species were considered as a core to be included in each evaluation, if possible. Other species were selected because of the regional abundance or anticipated unique performance profiles with waterborne systems (Table 1).

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Table I—Wood species utilized in efficacy evaluations of alternative preservatives.

Species	Characteristics
Core	
Southern Pine: wood from several sources (e.g., *Pinus, elliotii, palustris*, or *taeda).*	Commonly used preservatively treated, construction wood in eastern USA; also used as representative of pine construction in Indonesia
Red oak: *Quercus rubra* and related spp.	Representative of railroad ties
Douglas-fir: *Pseudotsuga menzezii*	Commonly used construction material in western USA, refractory heartwood; wood from second-growth Coastal Douglas-fir was used
Regionally Important	
Red maple: *Acer rubrum*	Important in eastern USA, subject of much engineering research on laminated structures
Hard (sugar) maple: *Acer saccharrum*	Common in Northcentral and Northeast USA; dense wood with potential of increased usage
Cottonwood: *Populus deltoides*	Common in central USA and along river bottoms in western USA
Yellow birch: *Betula alleghaniensis*	Prominent in Northeastern USA; interest in new opportunities of utilization of heartwood core
Sweetgum: *Liquidambar styraciflua*	Common in Southeastern USA
Tulip poplar: *Liriodendron tulipifera*	Common in Mid-Atlantic states and Southeast USA
White oak: *Quercus alba* and related spp.	Common in eastern USA; also used in railroad ties
Red (Norway) pine: *Pinus resinosa*	Common in Northcentral and Northeast USA
Eastern hemlock: *Tsuga canadensis*	Common in Northeastern and Northcentral USA
Western hemlock: *Tsuga heterophylla*	Common western softwoods; component of Hem-Fir grouping in lumber grading
Grand fir: *Abies grandis*	
Keruing: *Dipterocarpus* spp.	Used as representative Indonesian hardwood

Selection Of Preservatives

In this research program, we emphasize evaluating the efficacy of emerging preservatives in alternative wood species rather than conducting chemical research on new preservative compounds. We selected preservatives in which the active ingredient (biocidal component) was being considered within the United States, was approved by the Environmental Protection Agency (EPA), or was in use in another country. Thus far, we have worked principally with systems that contained only a single active ingredient. In future work and as our ability to design protection systems increases, we anticipate emphasis to be placed on blends or combinations of active ingredients.

Both waterborne and oilborne preservatives are considered in this research program. Although several anticipated uses for treated-wood products might entail a high level of human contact, therefore favoring inorganic systems, many transportation structures are not frequently exposed to humans. These structures also endure repeated vibration and are exposed to lengthy periods of weathering where organic systems function well. Not every preservative is used in all experimental combinations, but the program, as a whole, includes the preservatives listed in Table 2 in at least one experiment.

Preparation Of Wood Materials

Some experimental units were prepared and treated at the FPL, shipped to field locations, and installed by cooperators. Other units were prepared, treated, and installed by cooperators. All experimental materials used in FPL laboratory experiments and field plots were prepared and treated at FPL.

In soil-bed (fungus cellar) studies at FPL, stakes were 3 by 19 by 150 mm, with the longest dimension parallel to the grain. In most of our field trials, stakes were 19 by 19 by 450 mm, with the longest dimension parallel to the grain. With a few studies, however, we used stakes that were 25 by 50 by 500 mm. The Douglas-fir were incised prior to treatment. No other wood

Table 2—Preservatives Included in integrated research program.

Waterborne perservatives	Oilborne perservatives/creosotes
Acid copper chromate (ACC)	Chlorothalonil (CTL)
Ammoniacal copper zinc arsenate (ACZA)	Chlorothalonil/chlorpyrifos
Ammoniacal copper citrate (CC)	Copper naphthenate
Ammoniacal copper quat Type B (AC Q-B)	Creosote "P1"
Ammoniacal copper quat Type D (ACQ-D)	Creosote "A"
Chromated copper arsenate (CCA)	Creosote "B"
Copper dimethyldithiocarbamate (CDDC)	Creosote , pigment emulsified (PEC)
Copper naphthenate	Isothiazolone
Propiconazole	Oxine copper (copper-8-quinolinolate)
Propiconazole/pyrethroid	
Rh 287	
Sodium octaborate tetrahydrate	
Tebuconazole	
Tebuconazole/chlorpyrifos	

species was incised. Stakes were prepared from the kiln-dried wood furnish as supplied. Except for Douglas-fir, only wood that was free of knots was used for stakes. Douglas-fir stakes for field plots included wood with small-sized knots.

All wood stakes were cut from the finish to the de sired dimension, then equilibrated to a constant weight. The equilibrated stakes within each size group in each species were then weighed and arrayed by weight, The entire lot of weighed stakes was then divided into groups of 30, each with equivalent mean density and equivalent variation in density. This was accomplished by dividing the total population of stakes that were arrayed by weight into divisions of 30 replicate stakes, Each division was sequenced by density (weight/constant volume). To develop the set of 30 replicate stakes per treatment, we selected stakes from each of the divisions in a selection process that alternatively passed up and down the array from the least dense to most dense division.

Soil-Block Procedures

Standard and modified soil-block procedures were utilized within FPL. In a study that is evaluating alternative preservatives for use in softwood construction in Indonesia, blocks of Southern Pine sapwood were selected and treated with preservatives in accordance with A WPA E-10 (AWPA 1995c), except that the blocks were pressure treated by the fill-cell process in the same work tank and treating solution as were the field stakes to which the blocks are referenced. In other studies, an attempt was made to compare soil-block methodology with field test procedures. In these comparative evaluations, soil blocks were made from cross sections of 10 randomly selected stakes that were withdrawn from the set of 30 that were treated with each concentration of preservative in their respective tests. For each assay fungus used in the soil-block test, one 19-mm-long block was cut from midlength of each of the 19- by 19-by 450-mm stakes that were removed for analyses from each lot of 30 stakes per treatment. This created 10 replicate cubes per preservative treatment for each assay fungus.

Treating Procedures At FPL

Comparisons of wood preservative chemicals in wood products are usually made on a basis of equivalent loadings (retention levels) in a given wood species, Standard procedures for preparing soil blocks (AWPA 1995c) or field stakes (AWPA 1995b) define Southern Pine sapwood as the preferred wood and require selection of clear wood materials with a maximum allowable variation in density.

This research program encompasses a host of wood species. Each experiment entails treatment of a number of wood species to different retention levels with a group of preservatives (Table 3). To accomplish this array of treatments with a minimal amount of chemical, the retention series for all preservatives in all wood species is accomplished by treating all woods with the same gradient series of percentage active ingredient and allowing the resultant retention (weight/volume) to be distributed as determined by the variability in density of the wood materials being treated. As a result of the method of stake selection, a comparability in distribution of solution uptakes is anticipated for each wood by

Table 3—Concentration of active ingredients used at FPL when treating different wood species for evaluation of preservative efficacy.

Active ingredient (%)			Copper (%)	
Water-borne systems[a]	Creo-sotes	Chloro-thalonil (oil-borne)	Oxine copper (oil-borne)	Copper naph-thenate (oil)
0.08 a	7.5	0.37	0.04	0.04
0.15 a	15.0	0.74	0.08	0.08
0.30 abc	30.0	1.48	0.30	0.30
0.60 abc	65.0	2.96	0.60	1.20
1.20 abc			1.20	
2.40 abc			4.80	
4.80 c				

[a]Letter following concentration identifies type of experiment in which that concentration of active ingredient was used: a = soil-block studies; b = stake tests in U.S. field plots, c = stake tests in Indonesia. This partitioning of experiments was done only with the water-borne preservatives as indicated here.

treatment combination. However, visual identification of heartwood is not always possible, so some variability results. At each treatment, the preservative uptake of individual stakes is recorded on a computer. The identity of each stake is maintained in the field, and ultimately the performance of individual stakes will be related to the actual preservative uptake for that stake. Field results from each plot will ultimately be displayed in a computer-generated spreadsheet that relates performance to preservative retention on an individual stake basis. We anticipate that this will be an improvement from past practices of recording only the average retention and not accounting for the actual variation in retention that occurred in the population of stakes at time of treatment (Gutzmer and Crawford 1995).

In a study in which several preservatives were evaluated for their ability to protect Douglas-fir, we used a slightly different concentration of active ingredients (Table 4). All experimental materials were treated using the full-cell process. Calculations of chemical retention in each treated item were made on the basis of weight gain during treatment.

Creosote was diluted with toluene to simulate an empty-cell process. Copper naphthenate, chlorothalonil, and oxine copper were dissolved in No. 2 diesel oil, which met AWPA requirements for hydrocarbon Type A (AWPA 1995a), and then diluted with

Table 4-Concentration of active ingredients used at FPL in treatment of Douglas-fir for evaluation of preservative efficacy.

Active ingredient (%)		Copper (%)	
Water-borne systems	Creo-sotes	Chloro-thalonil (oilborne)	Copper naph-thenate (oil)
0.37	7.5	0.37	0.31
0.75	15.0	0.74	0.62
1.50	30.0	1.48	1.25
3.00	60.0	2.96	2.50

toluene to achieve desired solution concentrations of diesel fuel and active ingredient. Treating solutions with oxine copper and copper naphthenate were mixed with toluene so that the amount of No. 2 diesel fuel was only 30% by weight of the total solution. Treating solutions with chlorothalonil were adjusted so that the diesel fuel comprised 22%, by weight, of the total solution.

For field testing, 30 replicate stakes were treated for each wood species by preservative by retention. At time of treatment, the retention of each treated stake was calculated on the basis of weight gain. Stakes treated with oilborne and waterborne preservatives other than ammoniacal systems were then allowed to air dry prior to shipment to a field site. Stakes treated with ammoniacal systems were wrapped in plastic and stored at least 2 weeks at ambient temperature before being unwrapped and allowed to air dry. Ten stakes were randomly withdrawn from each set of 30 replicate stakes for chemical analysis and to use in laboratory decay studies.

Evaluations

Evaluations of wood preservative efficacy in different wood species include the following:

- Laboratory decay studies in which treated-wood products are challenged with pure cultures of wood-destroying fungi
- Laboratory-scale, simulated ecosystems in which treated wood is exposed in soil beds that are maintained to selectively favor one or more grouping of deterioration organisms
- Field trials structured to expose treated wood to selective attack by termites
- Field trials designed to subject treated wood to naturally occurring microflora and fauna, either in ground contact or above-ground exposure

Pure Cultures

Laboratory experiments with pure cultures of wood-destroying fungi offer the opportunity to challenge the treated-wood product with organisms known to be tolerant of the preservative or the generic family of preservatives under evaluation. As such, pure cultures offer an opportunity to present a challenge for a specific wood by preservative combination that represents the worst possible exposure that could occur in the field. Laboratory decay studies with pure cultures also provide the opportunity to explore specific questions about the effect on wood species, veracity of fungi from different ecosystems against a specific wood product by treatment, effect of different physiologic groups of fungi, and even sequential effects of different, known organisms. A weakness of this approach is that pure cultures do not reproduce natural environmental settings and thereby do not simulate the interactive, competitive effects of natural microbial systems. Another limitation is that there is no universal agreement on the merits of these methods as predictors of actual field performance. The ASTM-type soil-block tests are generally more severe than the agar-block tests in that soil block usually gives greater threshold values (Cockroft 1974). Some researchers (Duncan 1953) consider results from soil-block studies with oilborne preservatives to be indicative of threshold values that occur in the field. Others (McNamara 1994) remain to be convinced. The severity of challenge posed by soil-block tests is greater than what occurs within 5 years of exposure above ground (De Groot 1994), but long-term relationships are not yet identified.

The appeal of agar-block methodology lies in its simplicity, relative ease of repeating the test with different fungi in any laboratory around the world, and reported speed, i.e., results within 3-6 weeks (Levi 1969). For these reasons, agar-block studies were included in our integrated evaluation protocol. In one study, a recent variation (Worrall and others 1991; Worrall and Wang 1991) of the agar-block procedure was used to evaluate the efficacy of waterborne and oilborne formulations of copper naphthenate in red maple (Smith and others 1996). In this procedure, miniature blocks measuring 5 mm (longitudinal dimension) by 10 mm (tangential) by 20 mm (radial) were treated according to experimental design and then supported on a plastic "needlepoint" mesh (2.5 perforations/cm) that rested on the mycelium of the assay fungus. This methodology proved useful in evaluating the preservative treatments against soft-rot and white-rot fungi, but did not promote aggressive decay by brown-rot fungi. Consequently, the agar-block method proved not to be of value as a way to evaluate efficacy against brown-rot fungi. This result with brown-rot fungi was surprising in light of Levi's (1969) prior success with veneers of wood that rested directly on mycelial mats. Smith (1996) believes that the reduced decay by the brown-rot fungus G. *trabeum* in this type of test is due to a requirement for a greater wood moisture content than occurs with this experimental procedure. Nonetheless, we confirmed this general performance profile in subsequent experimentation at FPL.

We also determined that variation among replicates can be controlled with soft-rot and white-rot fungi. In prior research with brown-rot fungi in which blocks were supported on glass rods over an agar substrate, variability recognized it as being too severe to permit statistical analysis (Cowling 1957). Through this integrated laboratory and field testing program, we will have the opportunity to relate results on copper naphthenate in red maple to subsequent field performance. However, it appears that more can be gained from the agar-block procedure, especially in light of its reported quick response time.

The use of wood blocks of small volume in laboratory, soil-contact decay studies (Bravery 1979) holds promise as a technique for rapid evaluation of new preservatives (miniblock procedure). This technique has been used previously to evaluate efficacy of wood preservatives on western red cedar sapwood (Scheffer and others 1987, 1988). For these reasons, the miniblock procedure was investigated for its utility as a tool for initial evaluations of relative performance of alternative preservatives on a variety of wood species. A cooperative evaluation of this test procedure was executed with Oregon State University. Laboratory activities were conducted at the university; data analysis was done at FPL. We found that this method lacked sensitivity in detecting significant differences among experimental varieties. This was probably due to the heterogeneity of variance in the experiment. We also observed that only a few fungi caused relatively more decay in a variety of wood species than did other fungi. These results suggest that the efficiency of this type of procedure could be increased by minimizing the number of assay fungi and maximizing the number of independent replications. The miniblock procedure seemingly has potential as an accelerated method for evaluating new preservatives. However, before this procedure can be accepted as a standard protocol, the source of procedural variability, reproducibility, and sensitivity needs to be defined.

At the FPL, we are conducting soil-block studies (AWPA 1995c) that utilize blocks prepared from subsets of the lot of stakes that were treated for either field exposure or analysis, as previously described, and we are conducting soil-block tests of parallel design using pine sapwood blocks. This should give us an opportunity to determine whether relative performance profiles in the laboratory are equivalent to those that we observe in the field. We are using representative, standard brown-, white-, and soft-rot fungi in the decay tests. It is anticipated that we will be able to define separate performance profiles for treated woods against each of the three major groups of wood-decomposing fungi.

We then will be able to compare those separate profiles with field performance in different ecosystems and with performance of comparable treatments by wood combinations in soil-bed (fungus cellar) exposure. In our investigation of alternatives to CCA for treatment of pine in Indonesia, we are complementing an extensive field trial that is replicated in West Java, southern Mississippi, and Wisconsin, with a soil-block study of alternative preservatives in pine sapwood blocks. In that soil-block study, we utilize only leached blocks because the pine in Indonesia is used mostly for cooling towers. Wood-decaying fungi that are important in both countries are used in this study.

Simulated Ecosystem

Beds of nonsterilized soil can be used as a medium for accelerated evaluation of treated-wood materials in a laboratory setting that more closely simulates a natural ecosystem than do pure culture studies. Small, elongated members of treated and untreated wood are immersed vertically into a bed of soil and incubated at a temperature and soil moisture content that allows naturally occurring microorganisms to flourish. The size of the wood members can be several millimeters to centimeters in dimension. Since the inception of soil-bed methodology almost 30 years ago (Gersonde and Becker 1958), there has been a growing recognition that the predominant microflora in the soil (i. e., soft-, brown-, or white-rot fungi) can be somewhat selected through control of soil moisture content in these beds. Even termites can be introduced to more completely simulate field exposure (Johnson and others 1988). There has been a growing interest in quantifying the measurements of decomposition (Baines 1984) and characterizing the performance profiles of populations of replicates in soil beds (Thorton and others 1995)—an approach that will allow meaningful comparison of results from these systems with results from field trials.

In our current research program, the performance of several species of hardwoods treated with both oilborne and waterborne systems is being evaluated in the soil-bed facility at Michigan Technological University. Stakes treated with the corresponding retention levels of the same preservatives are also being field tested in Hawaii and southern Mississippi. In the Michigan facility, the soil-bed stakes are 10 mm thick.

Three formulations of creosote that are not currently accepted within AWPA standards are also being evaluated in soil-bed tests at FPL. One formulation is also being evaluated in several wood species in a field simulator at CSIRO, Clayton, Victoria. In addition, the experimental design for the FPL soil-bed study is replicated with two sizes of stakes in an FPL field plot in southern Mississippi and, in part, in the CSIRO field sites near Walpeup, Victoria. A soil-block study using these formulations was previously completed at FPL. In the FPL beds, the soil is kept at a high moisture content to promote attack by soft-rot fungi, Stakes are 3 by 19 by 150 mm (radial, tangential, and longitudinal, respectively), Stakes in the CSIRO accelerated field simulator (AFS) are 25 by 50 by ~250 mm (approximately half lengths of the 15 by 50 by 500 mm stakes that were cross sectioned for chemical analysis), Soil in the AFS is maintained at a moisture content that promotes attack by brown- and white-rot fungi.

Field Trials

Field trials (graveyard tests), in which the durability of experimentally treated woods is ascertained by monitoring the duration that those wood members resist attack of naturally occurring microflora and fauna, have the greatest international recognition as the method for evaluating wood preservatives. Wood members (i.e., stakes) are usually vertically inserted to a depth of half their length into the soil and monitored during years of exposure. Stakes of various dimensions are used, but there is a general perception that comparative results will be obtained more quickly with stakes of smaller dimension than with those of larger dimension. Except for some procedural concerns about the relative potential for chemicals to leach from thin stakes with large surface area/cross-sectional area ratios, graveyard tests or field trials are generally regarded as providing the most direct evidence of preservative efficacy for actual products used in field conditions.

However, a major limitation of field trials is the time required needed for comparative results, particularly in temperate environments, Sometimes, decades of exposure are required to obtain results. For this reason, there is a growing interest in exposing treated materials in tropical settings. However, this practice has raised questions as to whether performance data generated in a tropical ecosystem will predict relative performance in temperate environments (Freitag and others 1995). Trends in performance of nonpressure-treated wood packaging that was exposed above ground in an open field in Panama forecast trends for similarly exposed materials in temperate environment, but results from packaging exposed above ground in the jungle did not (De Groot and Stroukoff 1986). We have also determined that comparative evaluations with field trials can be slightly accelerated by increasing the number of replicates per treatment and referencing comparisons to population medians or quartiles rather than means (Link and De Groot 1990).

In the research program reported here, field trials of treated-wood materials are being conducted in temperate and tropical climates (Table 5). Three separate studies permit us to address whether results for field

Table 5—Location of field plots included in the integrated evaluation program.

Location	Environmental description
USDA Harrison Experiment Forest: Southern Mississippi, 2 km from Gulf of Mexico, USA	Mean annual precipitation, 1580 mm Average annual temperature, 19.6°C Soil type at plots: poarch sandy loam Mean annual precipitation, 780 mm
Madison, Wisconsin USA	Average annual temperature, 7.4°C Soil type: clay loam soil
Hawaii USA	Mean annual precipitation, 3290 mm Mean annual temperature, 23°C Soil type: hilo silty clay loam
Morgantown, West Virginia USA	Mean annual precipitation, 995 mm Average annual temperature, 11.4°C Soil type: Clarksburg silt loam
Innisfall Field Station North Queensland Australia	Mean annual precipitation, 3556 mm Mean annual temperature, 22.7°C Soil type: clay-loam of basaltic origin
Mallee Research Station Walpeup, Victoria Australia	Mean annual precipitation, 340 mm Mean annual temperature, 16.2°C Soil type: light and sandy overlaying a calcareous clay-pan
Cikampek, West Java Indonesia	Mean annual precipitation, 1,891 mm Average annual temperature, 32°C-34°C Soil type: association of red Iatosol, brown Iatosol, reddish laterit; parent material: intermedier tufvolcan

trials with stakes in tropical environments forecast performance of treated-wood products in temperate climates. These comparisons are (1) Hawaii versus southern Mississippi, (2) Indonesia versus southern Mississippi and central Wisconsin, and (3) northern Queensland versus southern Mississippi. Two of these studies are established with complete replication of experimental design and a minimum of 20 replicate stakes per variable. This should provide an opportunity to compare the performance profile of individual wood species by preservative combinations in terms of relative performance, i.e. rank, population parameters of average, median, percentile, variability, and rates of decomposition after a defined starting point.

The objective of one experiment is to determine the durability of Douglas-fir obtained from second-growth forests, which are likely to be the source of most lumber in the future. Field trials with treated stakes exposed in ground contact are being conducted in southern Mississippi and northern Queensland, The same experimental design is used at both locations but size of stakes and number of replicates per treatment differ. The stakes in Queensland are nominally 50 mm square; stakes exposed in Mississippi are 19 mm square. Only 10 replicates per treatment are exposed in Queensland; 20 replicates per treatment are exposed in Mississippi. In northern Queensland, treated materials are exposed above ground and in ground contact. The above-ground evaluations will be made on several different configurations designed to simulate construction components in Australia. Ten replicate, treated Douglas-fir members with nominal cross-section dimensions of 50 by 100 mm were used per treatment by retention combination. Four different simulated construction components were fabricated from each replicate member.

A mixture of hardwoods and softwoods is included in two other studies that pair field plots of comparable design in tropical and temperate ecosystems. Southern Pine sapwood is used as a reference wood species in field trials that are replicated in Indonesia, southern Mississippi, and central Wisconsin. Several hardwoods from the northeastern United States are included in this study, which also addresses performance of tropical hardwoods that are treated with emerging preservatives. Twenty replicate stakes per combination are exposed in

each plot. These field plots are matched with a laboratory, soil-block evaluation of the same preservatives used in the field. This laboratory study is restricted to Southern Pine sapwood only.

Another study that places great emphasis on hardwoods from a large geographic distribution within the eastern United States pairs field plots of comparable design in Hawaii and southern Mississippi. Selected combinations of woods and preservative treatment are also exposed, above ground, in Hawaii in a procedure that is selective for attack by Formosan termites. The core of this experimental design is also duplicated in a soil-bed study at Michigan Technological University. Twenty replicate stakes were used in the field and soil-bed methods.

Formulations of creosote that are not included within AWPA standards are being evaluated in soil-bed studies at FPL and CSIRO, as previously described. These evaluations are also being related to matched field plot designs in Victoria and southern Mississippi. Southern Pine, red oak, red maple, and Douglas-fir were used in field plots. Only stakes that were 25 by 50 by 500 mm in dimension were used in Australia. Two plots were installed in southern Mississippi: one with stakes of the same size as used in Australia; the other with stakes that were 19 by 19 mm in cross section. The latter experiment contained 20 replicates of treatment by wood combinations. The FPL plot in southern Mississippi that was conducted with 25- by 50-mm stakes contained only two formulations of creosote.

Concluding Remarks

In this integrated laboratory and field testing program, emphasis is placed on evaluating alternative wood species that are treated with emerging preservatives rather than on a fundamental research that focuses on the development of new active ingredients. It is perceived that this approach will enable rapid deployment of new technology to the protection of additional wood species. The development of a new preservative requires a considerable investment of time and resource to bring an idea into the marketplace. Not only must the efficacy of the active ingredient be demonstrated, but a host of environmental performance criteria must be addressed before the EPA will permit its use, i.e., grant a label permitting its use on wood. Therefore, all new preservatives have to meet criteria of standard-setting bodies, such as AWPA and EPA. As noted previously, efficacy evaluations are being conducted principally with Southern Pine sapwood, which yields little information relative to the performance of hardwoods. By working with active ingredients that have either completed EPA review or are in the process of being reviewed, we selected those compounds and systems that had immediate potential for use in the wood-treating industry if their technical performance shows promise.

Success in developing composite structures for engineering applications increases the further prospects for utilization of alternative wood species, hence the interest in durable species goes far beyond transportation structures and cooling towers. You might anticipate that greater emphasis will also be placed upon designing treatments that address unique challenges presented by the alternative species. Towards that anticipated trend, this integrated program still needs to identify an accelerated, reliable, sensitive method to evaluate preservatives that will yield results quickly enough to permit feedback into a design program for new systems.

None of the field experiments that we are conducting has progressed far enough to allow definitive conclusions. However, we have gained an impression that relative trends observed among creosote formulations in the soil-bed trials reflect relative trends observed in the field exposures. We sense that when stakes of only a few millimeters thick are used in soil-bed studies, the performance of creosote, relative to that of CCA, is much less than actually occurs in large products in the field. We have not seen this indication with the larger soil-bed stakes, but that study is not complete. Soil-bed studies involving alternative waterborne and oil-borne treatments are showing some separation of treatment efficacy. There is some indication of differences among species in performance of individual preservatives. The soil-bed studies have not run sufficiently long enough to allow definitive evaluations of relative performance or provide a pattern of results for comparison with field results.

Populations of untreated 19- by 19-mm Douglas-fir stakes exposed in southern Mississippi were degraded at the same rate as was observed in populations of larger, untreated Douglas-fir stakes exposed in Queensland. Results from soil-block studies completed at FPL have demonstrated some significant differences among preservatives in threshold (minimum retention) levels required to prevent attack by different fungi. These results indicate that we should see some marked differences in field performance.

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Field Performance of New Wood Preservative Systems in Secondary Timber Species

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Abstract

The objective of this ongoing study is to evaluate the performance of new, potential, and standard wood preservative systems in regionally important timber species. An important purpose of the work is to provide information on preservative/wood species combinations that could be used in transportation structures. Eleven preservative systems were evaluated in this study - ACQ Type B, Copper Citrate 2:1, CDDC, chlorothalonil/chlorpyrifos, copper-8-quinolinolate, tebuconazole/chlorpyrifos, RH287, propiconazole/chlorpyifos, copper naphthenate, CCA, and creosote. Field evaluations are being performed with ground contact field stakes and termite-specfic testing in Hawaii, and a laboratory soil bed test. The major wood species used with all the systems and evaluation methodologies were loblolly pine *(Pinus taeda,* softwood), northern red oak *(Quercus rubra,* dense hardwood), tulip poplar *(Liriodendron tulipifera,* medium density hardwood), and cottonwood *(Populus deltoides,* low density hardwood). More limited evaluations (field stakes only) are being conducted with eastern hemlock *(Tsugu canadensis),* red maple *(Acer rubrum),* and sweetgum *(Liquidambar styraciflua).* Information on the comparative treatability of these species correlated with formulation type is presented, as well as fungus cellar and termite test results. Results from the field stakes is not available yet.

Keywords: Wood preservative, timber bridge, ACQ, CDDC, copper citrate, chlorothalonil, chlorpyrifos, copper-8-quinolinolate, tebuconazole, RH287, propiconazole, copper naphthenate, CCA, creosote, red oak, tulip poplar, cottonwood, eastern hemlock, red maple, sweetgum.

Introduction

Despite improved timber products development and engineered timber bridge designs in the past century, there has been a decrease in the number of timber bridges built. New technology in steel and reinforced concrete in the early 20th century became desirable and economical alternatives to timber. As more steel and reinforced concrete bridges were constructed, development of better timber designs was slowed (7). At the same time advances were being made in wood preservation technology and the development of engineered wood products such as glulam beams and structural composite lumbers. Today's wood preservative and wood composite technology are capable of providing life spans of over 50 years and span up to 140 feet (7).

One economic consideration in the construction of timber bridges is material and shipping costs. Smith and Bush conducted a nationwide survey of timber bridge companies and found that the vast majority of bridges are constructed from southern pine *(Pinus spp.)* and Douglas-fir *(Pseudotsuga menziesii* (Mirb.)Franco) (11). Minor use species included red/white oak *(Quercus rubra* L. */Q. alba* L.), red maple *(Acer rubrum* L.), and red pine *(Pinus resinosa* Ait.). A survey of all Michigan road and drain commissions and 18 municipalities showed that 11% of the bridges they have worked with in the past five years have

been constructed of timber products, and projects that 26% of bridges will be constructed of timber in the next five years (5). In order to save substantial money on shipping of southern or western species and to stimulate the local economies it would be advantageous to utilize one of the many species grown in the local region.

Anticipating an increase in the number of timber bridges to be constructed in coming years, and in order to promote their construction, the USDA Forest Service launched the Timber Bridge Initiative in 1988. Studies on mechanical and biological decay performance, in conjunction with traditional and new wood preservative systems, in under-utilized species is one major aspect of the Initiative. To date, 16 demonstration bridges have been built including the 42 ft eastern cottonwood Cooper Creek bridge in Appanoose County, Iowa. The two lane bridge, constructed in 1992 from creosote treated eastern cottonwood *(Populus deltoides* Bartr. ex Marsh), was built at a cost of approximately $34,200 for fabrication and construction of the bridge superstructure, railing and wearing surface. There has been no measurable creep, and no decay has been detected (6). Demonstration bridges such as this are quickly showing that hardwoods found locally in many parts of the country are an economical, structurally capable, and long lasting alternative building material.

Resistance of a weed species to biological decay is perhaps the most important factor in the service life of a timber bridge. Little is known about the decay performance and treatability of many under-utilized hardwood species with the new wood preservative systems of today. Some early treatability work investigated the permeabilities, and fluid and gas flow rates in various hardwoods. One such study evaluated unsteady-state gas flow in three hardwood species including yellow-poplar *(Liriodendron tulipifera* L.) (8). Choong and Fogg explored differences in gas permeability between the radial, tangential, and longitudinal directions in shortleaf pine *(Pinus echinata* Mill.) and yellow-poplar (2). Choong et al. also evaluated the permeability of various softwoods and hardwoods including red oak and yellow-poplar in the longitudinal direction with creosote, and found both to be more permeable than Douglas-fro and southern pine sapwood (3). Permeability coefficients can be determined for non polar liquids by measurements of air flow, due to the inertness of these fluids to the wood. In other words, for fluids which will not interact with the wood, differences in internal structure of various wood species are responsible for differing permeabilities and not differences between the fluids themselves (10). Given the anisotropic and variable nature of wood, it is clear how variation in permeability and therefore preservative retentions arise within a given species. Thomas discusses some of the anatomical features of three hardwood species that affect penetrability (12).

Other treatability studies have focused on variation of retentions between various species. Siau used a comparison of retentions of silicone, light hydrocarbon, and paraffin oils in loblolly pine *(Pinus taeda* L.), three western softwood species, and white oak to establish a linear relationship between permeability and retention (9). Weaver and Levi found different responses of species to various preservative systems, and that CCA and creosote penetrate similarly (13). This suggests that the differences between species are responsible for treatability differences, rather, or more so, than differences between preservative systems.

There is a lack of knowledge not only in decay resistance performance of many hardwood species with new preservative systems, but also in the treatability of these species with the preservatives. Gaining an understanding of which preservative systems are retained at the highest and lowest levels in which species, and which combinations are most variable in terms of retention may help partially explain results of field tests. It would also facilitate predictions of which species/preservative combinations will be most variable in terms of decay resistancconsistency an important consideration when deciding the proper materials for timber bridge construction.

Methods

Treating

Three hardwood species (red oak, yellow-poplar, and cottonwood) and one softwood species (southern pine) were cut into various test specimens, including the 19x19x450mm (AWPA 3/4-inch) field stakes (1) used for the treating analysis. Only sapwood was selected for southern pine and yellow-poplar to allow complete penetration of the treating solution. Red oak is primarily heartwood, so only heartwood was selected for this species to reduce the treating variability that would occur if stakes were composed of both heartwood and sapwood. No attempt was made to distinguish sapwood for cottonwood, because of the difficulty in identifying the heartwood in this species. To maximize consistency in southern pine wood density, only wood with between 6 and 10 annual rings per inch were cut into stakes.

The wood specimens were allowed to condition to ambient room conditions for a period of several months before treating. Additional field stakes were cut from two more hardwood species (red maple and sweetgum *(Liquidambar styraciflua* L.), and one more softwood (eastern hemlock *(Tsuga canadensis* (L.) Carr.), in fewer numbers. These additional specimens were not included in the statistical

study due to differences in the number of replicates, which would have provided a poor comparison.

Twenty stakes of each species were treated with each of the preservative systems at four different retentions (as shown in Table 1), for a total of eighty stakes per preservative system/species combination. The large degree of replication should have provided an average free from affected by differences within species such as where the wood was cut from, grain orientation, etc. Treating began with weighing the conditioned stakes and preparing the treating solution. CDDC is the only system requiring more than one treatment. The first is treatment of the wood with a copper-amine water solution (cupric hydroxide). The stakes were allowed to dry to ambient conditions for several months before the second treatment, which was sodium dimethyldithiocarbamate (SDDC) in water. Active ingredient retentions are based on the retention of copper + retention SDDC in pounds per cubic foot. See Table 1 for a description of all preservative system formulations.

The stakes were placed in metal containers and weighted down using aluminum blocks, with plastic mesh between stake layers and between the stakes and the container or weights to ensure maximum contact with the treating solutions. Following submersion of the stakes in the treating solution, the vessel was placed in a treating cylinder, where a vacuum of 25 inches of mercury was drawn for a period of 30 minutes. After the vacuum period, the cylinder was pressurized to 100psi for one hour and then released. The stakes were then allowed to remain submerged in the treating solution for a 10 minute equalization period. Finally, the stakes were blotted dry, weighed, and placed in a second conditioning room to allow evaporation of the carriers and solvents. Retentions based on the amount of active ingredient retained by the stakes were calculated using the pre- and post-treated weights, stake volume, and active ingredient concentration in the treating solution. Since complete penetration was assumed no penetration measurements were made.

Termite field testing was done at a site in Hilo, Hawaii. The test blocks were randomized and placed horizontally on top of hollow concrete blocks 10 cm above the ground at a field site in Hilo, Hawaii. This site provides a severe termite challenge due to the very high *Coptotermes formosanus* population in the area. Untreated pine bait stakes (300 x 25 x 13 mm) were driven vertically into the ground through the holes in the concrete blocks to attract termites to the samples. To encourage the termites to feed around the test material, untreated pine “feeder” stakes (450 x 19 x 19 mm) were laid between the three rows of eight blocks and along the perimeter. The bait stakes were in direct contact with the feeder stakes, which were in contact with the test blocks. The assembled units were enclosed with a boxed lid (650 x 650 mm and 150 mm high) to maintain dark conditions and provide shelter from the weather. Termite-damaged bait and feeder stakes were replaced every six months when the test blocks were inspected. This maintains a constant high termite hazard for the blocks. The rating system was according to the 10, 9, 8, 7, 6, 4, 0 scheme, where 10 means sound and 0 is failure.

Soil bed (fungus cellar) testing is done using by placing stakes (13 x 25 x 1000 mm) into bins of soil such that three quarters of the length is buried. The soil beds are kept in an insulated room maintained at 80°F and 90% RH. Soil moisture content is mainted at a level appropriate for soft rot attack. Stakes are inspected at three month intervals and rated using the same numbering scheme as described above for the termite specimens. For both termite and soil bed testing, a single retention of each preservative was evaluated.

Statistical Analysis

Retentions of active ingredients ranged from 0.015pcf to 20pcf. In order to place each of the treatments in the same scale it was necessary to convert retentions to a volume of treating solution retained. In order to standardize the volumetric retentions, each was divided by the stake volume of 162.45 cm^3, thus making the retention a proportion of treating solution volume:wood volume. It was assumed that small differences in the concentration of active ingredients was not a significant factor in solution absorption, so all target active ingredient retentions were combined and considered a single treatment.

All statistical analyses were performed using the General Linear Model (GLM) procedure of SAS (SAS Institute Inc. 1988). The ANOVA model was designed to detect significant differences between the means for the species and preservative systems, and to test for a significant interaction term using the usual effects model. Another ANOVA using all combinations of species and preservative systems as the only treatment was also employed. This second ANOVA is called the “means” model by Milliken and Johnson (4). Tukey’s “honestly significant difference” multiple range test was used to determine which treatment means were different.

The coefficients of variation (CV) were calculated for each species and preservative system, and ANOVA effects and means models and their associated Tukey tests were analyzed. The set of 80 replicates for each species/preservative system combination was broken into four subsets of 20, with each subset representing each of the active ingredient concentrations, before performing the ANOVA and Tukey tests. This was done to allow for replication in the CV analysis.

Results and Discussion

Mean Volumes Retained

The effects model from the ANOVA analysis show that the species, preservative system, and interactions of the two are all significant explainers of variation. Of these, the species effect has an F value (7673.53) more than 10 times the other effects, and therefore is most important in explaining differences in average volumes of treating solutions retained per unit volume of wood. This tits suggestions of previous work in the Introduction that suggest that the internal structure of wood is responsible for varying permeabilities and retentions.

Figures 1-A through 1-D show the average volume of preservative retained by each species and the results of Tukey's multiple range test, which identities significant differences between each species/preservative combination in the means model. The Tukey's results are the letters at the top of each bar in the histograms. Those treatment combinations with the same letter are not significantly different, nor are treatment combinations which share any letters significantly different. The Tukey's designations are also valid across species in figures 1-A through 1-D.

It is apparent that red oak stakes retained a much lower volume than the other species. Note that the only overlap in designations from red oak to other species were between the SDDC uptake in red oak and creosote uptake in southern pine. Conversely, the cottonwood stakes retained the highest volume of every preservative. Retentions of the water-borne preservatives were similar in the cottonwood and yellow-poplar field stakes. Overall, the southern pine and yellow-poplar stakes were largely intermediate in the average retentions of the preservatives. The yellow-poplar stakes exhibited the most distinct difference between the water-borne, and oil and organic solvent borne preseservative systems. Based on the average density of each species, these results were expected. It is now apparent, however, that while the medium and low densities of southern pine and cottonwood differ insignificantly, the high density wood of red oak consistently retains significantly lower volumes of all treating solutions but creosote compared to the other species studied.

A very clear trend in the retentions of the individual preservatives is apparent from the figures. In every species, the water-borne systems were retained at higher volumes than the organic solvent systems. This is no doubt due to the ability of polar water molecules to enter wood cell walls and swell the stakes. This fact may mean that significant differences between aqueous and organic systems are not valid, however differences within the two types are valid. In addition, within the water-borne systems, SDDC was always the highest volume retained followed by cupric hydroxide, propiconazole/pyethroid, copper citrate 2:1, ACQ type C, and finally CCA at the lowest. As a group, the oil and organic solvent-borne systems were absorbed in smaller volumes than water-bornes. The following trend is evident in each species. RH287 was always at the highest volume retained followed by copper-8-quinolinolate, chlorothalonil, tebuconazole, copper naphthenate, and creosote at the lowest retention.

Mean Coefficients of Variation

The effects model from the ANOVA of coefficients of variation also show all three effects (species, preservative systems, and interactions) are significant in explaining differences in mean coefficients of variation. As for the volume/volume means, the species effect is most important with an F value of 64.02 compared to 4.15 and 2.79 for the preservative systems and interactions, respectively.

The fact that the species effect is many times greater than the others is of little surprise considering the inherently variable nature of wood, which seems to fit observations made by previous researchers. This is especially true compared to the relatively consistent nature of the treating solutions, which are composed of a large proportion of solvent and only small differences in the amounts of active ingredients.

Red oak had a significantly higher CV of 0.12, while the other three were not significantly different and ranged from 0.05 for southern pine to 0.065 for cottonwood. Among the preservatives, the only significant differences identified by the effects model was that copper-8-quinolinolate had a higher CV than chlorothalonil, creosote, RH287, tebuconazole, cupric hydroxide, SDDC, CCA, and copper citrate 2:1. From the means model analysis, four of the eleven preservative systems exhibited significant differences between species. These were cupric hydroxide and SDDC (both of the CDDC dual treatment system), copper citrate 2:1, propiconzole/pyrethroid, and copper naphthenate. In all four cases the significant differences were found between red oak and the other species.

The higher variability of some preservatives in red oak, and more variability in retentions within red oak, itself, maybe explained by varying proportions of latewood within each sample. A sample with a large proportion of high density latewood would not absorb nearly as much treating solution as one with a relatively small proportion of latewood. Another explanation would be varying extractives contents between specimens, which would result in varying permeabilities. For example, a specimen cut from near the center of the tree will have a lower extractives content than one cut from nearer the sapwood/heartwood interface end containing more extractives.

Figure 1- Mean uptake of preservative systems in southern pine, red oak, yellow-poplar, and cottonwood field stakes.

Figure 1 - continued.

Figure 2- Mean coefficients of variation for southern pine, red oak, yellow-poplar, and cottonwood field stakes treated with selected preservative systems.

Figure 2- continued.

Soil Bed Results

Results from the soil bed testing are given in Table 1. Different sections of this project were installed in the soil beds as the stakes were treated and conditioned, so there are differences in total time exposures for various preservative systems. Substantial degradation has occured in the untreated controls - essentially all the yellow poplar and cottonwood stakes have failed after 18 months exposure. The southern pine and red oak controls have intermediate ratings of about 6 after this time.

Some of the systems are performing well with little or no wood species effect, generally having average ratings in the 9's - ACQ, copper citrate 2:1, and creosote. These treatments appear to have broad use in a number of wood species at the specified retention. Other treatments have lower average ratings (around 8.5 to 9.0) but are still with consistent performance across the wood species - copper-8-quinolinolate, tebuconazole/chlorpyrifos, chlorothalonil/chlorpyrifos, and copper napthenate. The latter three treatments are formulated in a P9 Type A oil, which probably helps their efficacy. If these treatments had been evaluated at a higher retention, results comparable to the first group would probably have been observed. The remaining treatments - CDDC, RH287, propiconazole/pyrethroid, and CCA show better performance in the southern pine, lower ratings in the two lower density hardwoods, and intermediate performance in red oak. This is especially striking with CCA, where the pine samples are rating a perfect 10, while the lower density hardwoods have average ratings in the 5's after 24 months. These treatments are more selective in activity and adaptation to hardwood species needs to be done with caution.

Termite Field Evaluations

Current results from the field testing for termite resistance are given in Table 2. The ratings of the untreated controls show that termites are attacking the tested wood species roughly in the order of density with the lower density species undergoing more attack. Lower ratings for the untreated controls would have been expected by this time. The relatively slow attack rate is probably due to the untreated controls being dispersed amongst the treated blocks. In other work, we have found that some wood treatments can have a general protective effect over the specimens in an entire box. This is particularly true for preservatives with P9 oil carriers. Nevertheless, the substantial attack on the untreated controls show that termite activity is present.

In order to evaluate all the combinations of preservative and wood species, only one retention could be tested due to space constraints. The retention tested is the expected field stake threshold if the preservative was tested in southern pine sapwood. In this above-ground termite testing, the specimens should perform well in the pine substrate over the relatively short timescale reported here (18 months). The only significant attack on treated material is with the RH287/cottonwood combination. After 18 months, these replicates are averaging 7.6. As this work progresses, we expcet to see attack on other treatments in the lower density hardwoods as well. This further illustrates the need for caution when using wood preservatives developed for use in softwoods, in a hardwood species.

Conclusions

Effects and Means Models

The effects model for the mean volumes of solutions retained by the wood species clearly show that there are sigificant differences among the species, preservative systems, and interactions. Of these, the effects due to differences between species was the largest, followed by the differences between preservative systems, and finally the interaction of the two. The corresponding means model identities the specific differences between each species and preservative combination.

The effects model for the mean coefficients of variation identified significant differences in variance between species, preservative systems, and interactions similar to those in volumes of treating solutions retained. Again the species effect is clearly most important, the interaction is least significant, and the preservative system effect is intermediate, though much closer to the interaction effect. The means model for mean coefficients of variation of treating solutions retained also shows significant differences between each species/preservative system combination.

Multiple Range Test Results

The high density heartwood of red oak retains the lowest volumes of treating solutions, while low density cottonwood sapwood retains the highest. Yellow poplar and southern pine, being intermediate in densities, are also intermediate in volumes of treating solutions retained. In all species, there is a clear trend from highest volume of solution retained to lowest as follows: SDDC, cupric hydroxide, propiconazole, copper citrate 2:1, ACQ-C, CCA, RH287, copper-8 quinolinolate, chlorothalonil, tebuconazole, copper naphthenate, and creosote. In all but the red oak stakes, there is a fair to very well defined drop in volumes retained going from aqueous to organic solutions.

Red oak is certainly the most variable in the volume of treating solutions retained, which is no doubt the result of varying proportions of early and latewood and varying extractives contents between specimens. The other three

Table 1. Results from Evaluation of Various Wood Preservatives in Selected Wood Species Using a Soil Bed (Fungus Cellar)

Treatment	Wood Species	Target Ret'n	Actual Ave. Ret'n	Ave. Ratings (Months) 3	6	9	12	15	18	21	24
ACQ	Southern Pine	0.40	0.39	10.0	10.0	10.0	10.0	10.0	10.0		
	Red Oak	0.40	0.36	10.0	10.0	9.9	9.9	9.4	9.3		
	Yellow Poplar	0.40	0.43	10.0	10.0	10.0	10.0	9.7	9.9		
	Cottonwood	0.40	0.39	10.0	9.9	9.9	9.9	9.6	9.9		
Copper Citrate 2:1	Southern Pine	0.40	0.38	10.0	10.0	10.0	10.0	10.0	9.6	10.0	9.9
	Red Oak	0.40	0.38	10.0	10.0	10.0	9.9	9.9	9.1	9.5	9.2
	Yellow Poplar	0.40	0.4	10.0	10.0	9.9	9.8	9.6	8.9	9.7	9.5
	Cottonwood	0.40	0.4	10.0	10.0	10.0	9.8	9.8	8.7	9.3	9.1
CDDC - Cupric Hydroxide/ Na Dimethyldithiocarbamate	Southern Pine	0.20 / 0.96	0.16 / 0.90	10.0	10.0	10.0	10.0	9.8	9.6		
	Red Oak		0.2 / 0.98	10.0	9.8	9.7	9.7	8.4	6.8		
	Yellow Poplar		0.21 / 0.99	10.0	9.5	9.1	9.0	7.9	6.5		
	Cottonwood		0.19 / 0.93	10.0	9.6	9.5	9.5	8.6	7.2		
Copper-8-Quinolinolate	Southern Pine	0.30	0.29	10.0	10.0	10.0	10.0	9.9	9.0		
	Red Oak		0.19	10.0	10.0	10.0	10.0	9.8	8.4		
	Yellow Poplar		0.32	10.0	10.0	10.0	10.0	10.0	9.0		
	Cottonwood		0.28	10.0	10.0	10.0	10.0	10.0	8.5		
Tebuconazole / CPF / P9 Oil	Southern Pine	0.06 / 0.0175 / 7.0	0.061 / 0.0204 / 8.2	10.0	10.0	10.0	10.0	9.1	8.9		
	Red Oak		0.070 / 0.0212 / 8.0	10.0	10.0	10.0	10.0	9.1	8.8		
	Yellow Poplar		0.064 / 0.0172 / 6.9	10.0	9.9	9.9	9.9	8.9	8.2		
	Cottonwood		0.072 / 0.0208 / 8.4	10.0	9.8	9.5	9.7	8.8	8.4		
RH287	Southern Pine	0.06	0.058	10.0	10.0	9.5	9.1	8.7	9.1	8.0	
	Red Oak		0.064	10.0	10.0	9.2	8.7	8.3	8.3	7.0	
	Yellow Poplar		0.064	10.0	8.1	5.9	4.0	1.2	0.9	0.7	
	Cottonwood		0.063	10.0	9.2	7.8	6.2	3.7	3.4	2.2	
Propiconazole / Pyrethroid	Southern Pine	0.06 / 0.03	0.054 / 0.027	10.0	9.6	9.0	8.8	8.7	9.2	8.6	
	Red Oak.		0.046 / 0.022	10.0	9.8	8.7	8.3	8.4	8.0	7.7	
	Yellow Poplar		0.071 / 0.030	10.0	8.9	8.0	7.4	6.5	5.6	3.6	
	Cottonwood		0.057 / 0.028	10.0	8.9	7.9	6.7	5.3	5.1	3.8	
CTL / CPF / P9 Oil	Southern Pine	0.30 / 0.0175 / 7.0	0.32 / 0.0118 / 7.4	10.0	10.0	10.0	10.0	9.5	9.2		
	Red Oak		0.34 / 0.0202 / 7.9	10.0	10.0	10.0	10.0	9.2	9.2		
	Yellow Poplar		0.33 / 0.0124 / 7.8	10.0	10.0	9.4	9.4	9.0	9.0		
	Cottonwood		0.33 / 0.0179 / 7.6	10.0	10.0	9.4	9.4	9.1	8.6		
Copper Naphthenate / P9 Oil	Southern Pine	0.06 / 7.0	0.053 / 6.0	10.0	10.0	10.0	10.0			9.2	
	Red Oak		0.053 / 6.2	10.0	10.0	10.0	10.0			9.3	
	Yellow Poplar		0.066 / 6.8	10.0	10.0	10.0	9.9			9.1	
	Cottonwood		0.056 / 6.6	10.0	10.0	10.0	10.0			9.1	
CCA	Southern Pine	0.40	0.37	10.0	10.0	10.0	10.0	10.0	9.7	10.0	10.0
	Red Oak		0.37	10.0	10.0	10.0	9.6	9.4	8.6	8.5	8.2
	Yellow Poplar		0.39	10.0	10.0	9.8	9.0	8.1	6.8	5.8	5.3
	Cottonwood		0.39	10.0	10.0	9.6	9.2	8.5	7.6	6.7	5.9
Creosote	Southern Pine	10.0	8.1	10.0	10.0	10.0	10.0			9.7	
	Red Oak		11.5	10.0	10.0	10.0	10.0			9.9	
	Yellow Poplar		9.2	10.0	10.0	10.0	10.0			9.2	
	Cottonwood		10.7	10.0	10.0	10.0	10.0			9.6	
Untreated	Southern Pine	-----	-----	10.0	9.4	8.4	7.6	6.4	7.1	6.4	
	Red Oak		-----	10.0	9.5	8.6	7.5	6.3	6.1	5.5	
	Yellow Poplar		-----	8.7	6.8	3.8	1.1	0.8	0.2	0.0	
	Cottonwood		-----	9.0	6.8	4.0	2.1	0.8	1.5	0.7	

Table 2. **Results from evaluation of various wood preservatives in selected wood species by exposure to *Coptotermes formosanus* in a field site near Hilo, Hawaii.**

Treatment	Target Retn (pcf)	Wood Species	Calc Retn (pcf)	6 Months* Decay	6 Months* Termite	12 Months* Decay	12 Months* Termite	18 Months* Decay	18 Months* Termite
ACQ	0.40	Pine	0.43	10	10	10	10	10	10
		Oak	0.39	10	10	10	9.9	10	9.9
		Yellow Poplar	0.53	10	10	10	10	10	10
		Cottonwood	0.40	10	10	10	10	10	10
Cu-Citrate	0.40	Pine	0.40	10	10	10	10	10	10
		Oak	0.38	10	9.9	10	9.8	10	9.8
		Yellow Poplar	0.47	10	10	10	10	10	10
		Cottonwood	0.42	10	10	10	10	10	10
CDDC - Cupric Hydroxide/ Na Dimethyldithiocarbamate	0.20/0.96	Pine	0.17/0.95	10	10	10	10	10	10
		Oak	0.18/0.73	10	10	10	10	10	10
		Yellow Poplar	0.23/1.11	10	10	10	10	10	10
		Cottonwood	0.21/1.00	10	10	10	10	10	10
Cu-8	0.30	Pine	0.24	10	10	10	10	10	10
		Oak	0.22	10	10	10	10	10	10
		Yellow Poplar	0.30	10	10	10	10	10	10
		Cottonwood	0.27	10	10	10	10	10	10
Tebuconazole/CPF**	0.06/0.0175	Pine	0.047/0.0133	10	10	10	10	10	10
		Oak	0.064/0.0187	10	10	10	10	10	10
		Yellow Poplar	0.061/0.0170	10	10	10	10	10	10
		Cottonwood	0.079/0.0228	10	10	10	9	10	10
RH287	0.06	Pine	0.051	10	10	10	10	10	10
		Oak	0.064	10	10	10	9.9	10	10
		Yellow Poplar	0.062	10	10	10	10	10	10
		Cottonwood	0.064	10	10	10	7.6	9.0	7.6
Propiconazole/ Pyrethroid	0.06/0.03	Pine	0.060/0/028	10	10	10	10	10	10
		Oak	0.052/0.026	10	10	10	10	10	10
		Yellow Poplar	0.073/0.031	10	10	10	10	10	10
		Cottonwood	0.061/0/030	10	10	10	10	10	10
CTL/ CPF**	0.30/0.0175	Pine	0.26/0.0095	10	10	10	10	10	10
		Oak	0.34/0.0123	10	10	10	10	10	10
		Yellow Poplar	0.30/0.0112	10	10	10	10	10	10
		Cottonwood	0.35/0.0125	10	10	10	10	10	10
CuNaphthenate**	0.06***	Pine	0.055	10	10	10	10	10	10
		Oak	0.054	10	10	10	10	10	10
		Yellow Poplar	0.065	10	10	10	10	10	10
		Cottonwood	0.056	10	10	10	10	10	10
CCA	0.40	Pine	0.358	10	10	10	10	10	10
		Oak	0.362	10	10	10	10	10	10
		Yellow Poplar	0.427	10	10	10	10	10	10
		Cottonwood	0.395	10	10	10	10	10	10
Creosote	10.0	Pine	8.19	10	10	10	10	10	10
		Oak	11.30	10	10	10	10	10	10
		Yellow Poplar	9.67	10	10	10	10	10	10
		Cottonwood	11.5 0	10	10	10	10	10	10
Untreated	-----	Pine	-----	10	7.9	10	6.0	10	6.4
		Oak	-----	10	8.8	10	7.9	10	8.2
		Yellow Poplar	-----	10	7.9	9.0	6.2	8.9	6.0
		Cottonwood	-----	10	7.7	8.9	2.9	5.6	3.6

* Average ratings of 10 blocks; 10 indicates sound, 0 indicates failure.
** P9 oil retention = 7 pcf.
*** Retention of Copper only.

species are similar in the range of coefficients of variation, however there is no clear distinction between aqueous and organic solutions, though yellow poplar and cottonwood both follow the same trend.

Preservative Efficacy

Field termite testing is still in its early stages and few conclusions can be drawn yet. From soil bed testing, some of the preservative systems are performing well at the test retention in all the wood species tested. These are ACQ, creosote, and copper citrate 2:1 at target retentions of 0.40, 10.0, and 0.40 pcf, respectively.

Overall Conclusions

A long service life and biological decay resistance throughout the entire structure of a timber bridge are very important considerations. While field and laboratory decay tests will show which species and preservative systems are most appropriate for this application, this study has provided some information which will be useful in predicting consistency of decay performance within an individual bridge structure or between several bridges constructed of the same species and preservative.

In terms of the volumes of treating solutions required to treat a given volume of a species, it is clear that density is an important factor. As expected red oak retained significantly lower volumes of treating solutions than yellow-poplar or cottonwood. Southern pine, yellow-poplar and cottonwood had varying amounts of overlap, and fewer significant differences. Therefore, if lower amounts of treating solutions would be advantageous in the manufacture of the treated timber, red oak would be the best choice. However, this study showed that red oak is significantly more variable in retaining several preservative systems than cottonwood. All other comparisons between species are insignificant. Considering that treatments are most variable in red oak would make a less appealing choice in terms of consistency of decay performance.

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Recreational Timber Bridges in Pennsylvania State Parks and Forests

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Abstract

This paper reviews three specialty timber bridge projects designed to serve as heritage recreation features on state land in Pennsylvania, including:

(1) twin 16-foot span Kingpost truss bike trail bridges,
(2) a 50-foot span Camelback truss canal bridge;
(3) a 220 foot long three-span Burr arch/truss aqueduct;

All three bridges share the common goal of re-utilizing the best traditional structural timber forms and joinery, in combination with modern timber technologies. These projects demonstrate the importance of heritage reconstruction by incorporating historic details integral to the structures. These bridges are not simple wooden facades applied to concrete or steel stringers, but are true timber structures. The three designs were collaborations between landscape architects, Simone and Jaffe Incorporated (SJ) and structural engineers, DCF Engineering Inc. (DCF.) In each case, the bridges represent similar partnerships which include: project initiation by local citizen organizations; design funding by USDA Forest Service; and construction and ownership by the State of Pennsylvania through the Department of Conservation and Natural Resources. The projects were in various stages of design and construction at the time of this submission.

Keywords: Timber, Historic, Recreation, Bridges, Kingpost, Camelback, Burr Truss, Delaware Canal, State Park, Aqueduct, Forbes State Forest, Pennsylvania, USDA Forest Service.

Figure 1-Location of Described Bridges

1. Linn Run Kingpost Truss Bike Trail Bridges

Background

Twin trail bridges were constructed in 1995 over Linn Run, a creek which flows down the western slope of Laurel Hill within Forbes State Forest in Westmoreland County. Laurel Hill is the last, great ridge in Pennsylvania which has historically posed an obstacle to all those traveling to the west.

In 1758, Brigadier General John Forbes built his military road across Pennsylvania as a strategic supply route for the British march on Fort Duquesne (Pittsburgh) during the French and Indian War. Forbes Road, as it is known, included forts constructed 30 miles apart. A dying Forbes was carried over Laurel Hill from Fort Bedford to Fort Ligonier, built at the western foot of Laurel Hill. The French left Fort Duquesne in the fall of 1758 to attack Fort Ligonier, but were turned back by the British in what many consider to be the decisive battle of the war.

The Pittsburgh, Westmoreland and Somerset Railroad (PW&S) grade was constructed over Laurel Hill near the Forbes Road crossing. The PW&S was a short-lived, but significant logging, freight and passenger railroad which was operated between 1900 and 1916 by the Byers-Allen Lumber Company. Lumber was processed at its mill in Ligonier. In less than ten years, Laurel Hill was denuded of trees and declared to be forever barren. The State of Pennsylvania purchased a 9,500 acre tract from Byers-Allen in 1909 and named it after Forbes, as the first State Forest Reserve in the Ohio River basin. Since that time, the forest ecosystem has regenerated and sustained yields based on the PA Bureau of Forestry (Forestry) management plan. Forbes Forest now encompasses over 50,000 acres, mainly along 40 miles of the Laurel Hill ridge.

In 1990, the Loyalhanna Watershed Association (Loyalhanna,) a citizens' environmental organization based in Ligonier, proposed the development of a mountain bike system within the rugged forest to manage the growing sport as part of the state's multi-use management plan. With funding from the federal Southwestern Pennsylvania Heritage Preservation Commission (SPHPC), and a partnership with the local forest district, Loyalhanna commissioned Simone and Jaffe Incorporated (SJ) to develop the PW&S RR Bike Trail Master Plan in 1993. The plan identified the PW&S grade as a significant historic landscape feature within Forbes Forest, noting the integrity of several remaining RR structures, including: the main grade, two runaway grades, several stone drainage culverts, two bridge sites, and a watering pool fed by Linn Run.

Linn Run Bridge Program

Reconstruction of two Linn Run trail bridges were determined as priorities to improve the RR grade as the main "spine" of the mountain bike trail system. New bridges would allow mountain bikes to safely cross the creek and discourage fording, as well as establish emergency and maintenance vehicular access to previously inaccessible parts of the trail. Loyalhanna's goal to develop a trail, included adding integrity to the site by constructing new timber bridges in a historic form that would also meet modern design requirements. Forestry was concerned with bridge construction, safety, and long term maintenance. Loyalhanna commissioned the team of SJ/DCF for a design and heritage construction workshop demonstration project using a USDA Forest Service Timber Bridge Initiative grant. Traditional timber joinery techniques would be taught to Forestry personnel and local volunteers in a week-long fabrication workshop. Loyalhanna, SJ/DCF, and Forestry jointly developed an aesthetic and practical design program to ensure that the bridges respected the historic period of significance (1901 to 1916,) and were capable of handling modern demands. A single bridge design was developed to economically serve both short spans of approximately 14 feet.

Linn Run Bridge Partnership

Loyalhanna managed the entire project including; grant writing, material procurement, and publicity. SJ worked with Loyalhanna and DCF on technical design details, developed milling and treatment schedules, and coordinated workshop details with Forestry. DCF produced the final design and construction documents from the SJ concept. Members of the Timber Framers Guild of North America, were recruited by a fellow

Figure 2-Previous Footbridge at Linn Run

guild member at SJ to serve as fabrication workshop teachers. A lead timber framer and landscape architect provided technical support during installation. Forestry provided the labor and equipment for the installation of both bridges. The PA Youth Conservation Corps also helped by adding riprap to stream channel structures and installing the new timber bridge sills.

Existing Conditions at Linn Run

Two narrow foot bridges without handrails previously existed on top of WPA-era concrete stream channel structures. The structures were considered unsound as abutment foundations for the new bridges, but needed to remain undisturbed to avoid additional construction costs and time for stream encroachment permits.

With no original documentation available, the period of significance was used to select a "King Post" truss as the best representative historic timber structure for its strength, simplicity of construction, and aesthetics. A 45 degree top chord angle was chosen for the short spans so the top chords would create a strong vertical element above the required 54" high bike railing.

Engineering - Traditional Kingpost trusses had few iron elements until builders found that the addition of keeper bolts at the heel connection and tie straps at the kingpost greatly increased the load capacity. At Linn Run, members and connections were designed to meet AASHTO HS-8 (eight ton) loading. The steel connections were designed to mimic early iron connections. Field installed 2 5/8 inch diameter shear plates provided the needed capacity at the kingpost connection. Lateral stability of the trusses was secured by extending the transverse beam beyond the floor of the bridge to accept diagonal braces to the sides. Some of the DCF engineering details were altered slightly to suit field construction. Field test loading by an eight-ton backhoe with driver showed no deflection in the transverse beam to the naked eye.

Figure 3-Side Elevation - Kingpost Truss

Figure 4-Transverse Section - Kingpost Truss

Figure 5-Completed Kingpost Truss Bridge

Species Selection - Hemlock and White Pine were the major species harvested during the peak of the timbering era. Local barns and other structures made of hemlock have historically shown little signs of checking or twisting - an important consideration for posts. Hemlock was selected for the entire bridge. The timbers were air dried for a month before fabrication.

Timber Protection - Modern techniques were selected which would improve safety and efficiency, but not sacrifice the integrity of the original structure type. The finished bridge members were returned to the mill for CCA preservative treatment after joinery fabrication. This ensured that framers did not breath treated dust during joinery, and that the treatment "shell" of protection would not be perforated during construction. The treatment process left a subtle green hue to the finished timbers after they were left to dry for two weeks between treatment and installation. Every mortise, tenon, pin hole, and nail hole was executed before treatment. Every joint was designed with weep holes or sloped bottoms to prevent standing water. A "flitch cut" cap was installed to protect truss top chord from weather and to discourage people from climbing.

Table 1 - Linn Run Bridges • Project Costs

Consultant fees	$8,096
Materials	$7,158
Equipment	$ 645
Personnel	$6,811
Overhead	$ 406
CASH EXPENSE TOTAL	$23,116
Bureau of Forestry - labor	$9,801
Bureau of Forestry - equipment	$ 480
Youth Conservation Corps	$3,750
LWA	$ 525
Americorps	$ 420
Construction Volunteers	$1,400
White Oak Pins	$ 200
Accommodations	$ 450
IN-KIND EXPENSE TOTAL	$17,026
TOTAL PROJECT COST	$40,142

Hardware - A local blacksmith was found to fabricate and galvanize antique style hardware for the bridges.

Foundations - The new bridges were designed to bear on new creosoted RR tie sills set in gravel behind the existing concrete walls.

Kingpost Bridge Construction

Fabrication was performed in the Forestry workshop during a week-long training session with approximately 20 people in May, 1995. Modern tools such as a portable bandsaw and chain mortiser were used to rough out joinery for most bridge members. Forestry staff and PA Youth Conservation Corps workers installed the new bridges during a week in June, 1995.

Project Significance - Linn Run Bridges

Building the Linn Run Bridges was a symbolic and tangible way of involving the entire community in environmental, cultural, historic and economic issues of the region, (such as stewardship of natural resources, timber management, forest renewal, railroad history, modern milling, and judicial use of preservative treatments.) The new bridges physically demonstrate new business opportunities between today's local timber industry and the region's growing recreation industry.

The workshop helped perpetuate the historic craft of timber framing, and showcase a highly successful partnership between: federal and state governments, public utilities, private foundations, and local citizens organizations. A professional video documentary of the project, produced by a local television station, has aired on public television. The Linn Run site is scheduled for additional heritage/interpretive improvements in 1997.

2. Delaware Canal Camelback Bridges

Background

The Delaware Canal is a 60-mile long by 60-foot wide state park in Pennsylvania which parallels the Delaware River from the Lehigh Canal at Easton to the historic seaport of Bristol. The canal began operation in 1831, and the "Camelback" truss was the original bridge designed to span the waterway at local roads and farm lanes. The bridge approaches were raised to allow mules and drivers on the towpath, and canal boats on the waterway to pass below. The average bridge span was 48 feet between stone abutments. From a distance, the Camelback profile was similar to Palladio's design for a framed truss, dated about 1550. Today, the Delaware Canal is a National Historic Landmark, National Recreation Trail and part of the Delaware and Lehigh Canal National Heritage Corridor. Camelback bridges are an enduring symbol of the Delaware Canal. Three original design Camelbacks still exist, but are closed to vehicular traffic or weight restricted to three tons. In 1993, the Friends of the Delaware Canal (Friends,) in partnership with PA Bureau of State Parks (Parks,) received a Timber Bridge Initiative grant from the USDA Forest Service to study existing bridges over the canal and to design a "modern" timber Camelback truss bridge capable of carrying an HS-20 AASHTO design load. Parks projected constructing five camelback replacements over the next five years. SJ/DCF was retained to produce three optional designs, including:

a modern HS-20 Camelback truss;
a facade truss for existing canal stringer bridges and;
a documentation of the original bridge design.

Description of Original Camelback Structure

The original Camelback bridges were elegant in their simple appearance, but complex in operation. The main truss included a horizontal top chord between two sloping top chords. Two posts were joined into the top chord and notched into a composite transverse beam below. A wrought iron rod was slung over a cast iron saddle on the chord above each post to suspend the transverse beams. These rods were inclined slightly away from the post into the cartway and toward the outside of the bridge to also serve as lateral bracing for the trusses. The transverse beams divided the span approximately into thirds. The three bays were bridged by stringers which simply overlapped side by side on

Figure 6-Original Camelback Truss Bridge

the transverse beams. Transverse decking extended slightly past the outside of the truss. Lower chords were constructed of two wrought iron rods located on both sides of each truss and connected to a cast iron "shoe," or heel connector, which also received the foot of the sloping top chord and sat unanchored on the bearing shelf of the stone abutment. Three wrought "links" or sections of chord were connected by forged interlinking eyes to form a full bridge-length rod. The cast shoe was formed to accept the foot of the rafter chord, and to cradle a transverse wrought iron pin on the outside end. The pin was slipped through eyes at the end of the chord rods and nested in a cove behind the shoe. The wrought pin was fabricated with a head on one end and a slot for a wedged pin to lock the other end from working free from the chord eyes. The weight of the bridge placed the rods in tension by pushing the sloping top chords in the longitudinal direction. As the chords stretched, the unanchored shoes were free to move. The lower chords actually rested on the transverse beams which were set at a slightly higher elevation than the level of the abutment shoes. As the lower chords were stretched by loading the bridge, they placed additional pressure into the transverse beams in an effort to straighten - creating a completely flexed structure.

Modern HS-20 Camelback Design

The demands on the Camelback bridges have increased since 1831. The simple canal crossings to farms and houses are now expected to serve heavy oil delivery and refuse trucks.

Team Responsibilities - SJ/DCF designed and documented the new timber superstructure. Parks engineered the foundation, approaches and site.

Design Program - As the owner, Parks required the SJ/DFC design to meet certain design standards including: HS-20 capacity, a l2-foot wide clear cartway, a redundant design for safety, and low maintenance materials. The USDA Forest Service grant required a demonstration of hardwood glued lamination with preservation technology at a highly visible site, in partnership with state and local organizations. Friends' design requirements included: maintaining or reinstating towpath headroom under the new bridge, and aesthetic considerations such as - bridge scale, local stone abutments, and traditional color finish.

Design Improvements - SJ/DCF chose to recreate a true Camelback truss at a slightly larger scale and replace overlapping stringers with single span curved glulam stringers that bear on both abutments and transverse beams. Cast iron hardware was replaced by simplified galvanized steel saddles and shoes. Wrought iron tension rods were replaced by galvanized steel rods with turnbuckle tensioning. Transverse members were increased in depth to handle the increased bending moment demanded by the 20-ton truck design load and the widening of the original cartway by two feet. The transverse suspension rods were housed on the inside post faces to remove the old obstruction to the cartway. The glulam stringers were designed with different radii for top and bottom curves, which permitted a 16 1/12

Figure 7-Side Elevation - Modern Camelback Truss Bridge

inch member depth at the abutment bearings and a 27 l/2 inch member depth at center span. The new design included bolting stringers to the transverse beams to ensure that they work in concert with the trusses. Top chords were designed with the traditional gable cap boards.

Modern Camelback Engineering

Trusses were designed to support an HS-20 load through suspension of the floor at the posts - which creates compression in the truss chords and tension in the rods. Parks required a redundant design in the event that a large vehicle were to strike the trusses, causing them to fail. The trusses were designed with 8" x 12" Southern Pine top chords and 1 l/2 inch diameter bottom chord ties. Diagonal 4" x 8" red oak braces completed the truss profile. The floor stringers were designed using glued laminated red oak 10 3/4 inches wide as the redundant structural elements. The floor beams are tapered and curved to simulate the original "hump" as closely as possible.

Camelback Timber Treatment

The traditional Camelback bridges were painted red to protect the timber and for identity. Penta in light oil was selected as the preservative treatment for the truss members, to allow the new bridge to also be painted.

Significant Camelback Location

Several locations were contemplated by Parks for the prototype site. The Virginia Forrest Picnic Area was selected as a major state park access location - directly adjacent to PA Scenic Highway Route 32, and approximately 3 miles north of New Hope. The new HS-20 Camelback prototype will replace a ten year old, 10-ton weathering steel prefabricated pony truss removed due to extensive rusting. The bridge site is visible from heavily traveled River Road, and the new ramped approach will re-establish a useable towpath below. The new bridge will allow mule-drawn tour boats, which were previously forced to stop at the old bridge, to now continue an additional mile north, connecting the tourist towns of New Hope and Lumberville via canal.

3. Tohickon Aqueduct, Delaware Canal State Park, Point Pleasant, PA

Brief History

The Tohickon Aqueduct, in Point Pleasant, PA was originally built in 1831, over the largest Delaware River tributary along the 60 miles of Delaware Canal. The waterway served as an active commercial transportation route for 100 years. Point Pleasant, located at mile 30, was well known for its accommodations within Bucks County. Historic survey notes indicate that the original timber aqueduct in Point Pleasant consisted of seven spans. Unfortunately, no original construction documents remain.

Previous Tohickon Aqueducts - Several old photographs show a subsequent 3-span iron truss version of the aqueduct with wooden waterway or "trunk." The structure was built on stone abutments and two concrete covered stone masonry piers. Each span was approximately 60 feet. The old photos show the iron aqueduct brimming over with sheets of water spilling into the Tohickon and people swimming in the "old swimmin' hole" as it was called. Two days after a Labor Day community swim in 1932, the iron aqueduct failed catastrophically. Fortunately, no one was injured.

Commercial operation ceased after the aqueduct failure. The canal was acquired by the Commonwealth as a state park and, after World War II, a reinforced concrete and steel flume was built atop the existing abutments and two reconstructed stone piers. The concrete aqueduct served until the mid 1980's, when concrete buttresses on the outside of the walls began to lose integrity and no longer support the water pressure. By 1993, the structure was empty, condemned and the towpath closed to foot traffic.

Creating an Aqueduct Partnership - In 1991, the Point Pleasant Community Association (PPCA) began discussing the reconstruction of a historic aqueduct using timber. A concept design was developed within the association and proposed to Parks, which in turn proposed a challenge: if PPCA could deliver the timber superstructure engineering documents, Parks would provide all foundation and sitework engineering. The agreement was endorsed by local legislators. A $10,000 grant from the USDA Forest Service Rural Development Program funded preliminary engineering of the timber superstructure, through a local township. A similar grant funded prefinal engineering. Both grants were matched by equal amounts of Parks engineering time. A third Forest Service grant from the

Figure 8-Tohickon Iron Truss Aqueduct, 1932.

Timber Bridge Initiative grant was awarded to PPCA for final construction documentation and partial construction observation. Again the match had an equal Parks commitment. Parks enlisted engineering expertise of the PA Bureau of Flood Protection (BFP) for foundation design. SJ/DCF was retained by the community to perform the superstructure design.

By 1993, the aqueduct was deemed the highest priority state park capital project in Pennsylvania. Two million dollars were allocated in the state budget for the reconstruction. However, design proceeded slowly. The typical competitive bidding for state-funded design contracts was waived when Parks elected to perform the foundation design in-house and use the community's design team produce the superstructure. Funding for the design team needed to be raised entirely by the local community, in phases. Coordination between SJ/DCF and state engineers was fruitful, but complex with each working on different schedules and interdependent parts of the project. Early design phases included consultation from timber bridge builder Jan Lewandoski, of Greensboro Bend, VT. The Pennsylvania Flood of 1996 also delayed state engineers for several months. Technical aspects of the project however, were the greatest challenge.

Figure 9-Tohickon Aqueduct Failure, 1932.

Aqueduct Program Requirements

The project program included: preserving and reusing the original stone abutments and piers; designing a new heritage superstructure; creating a new trunk waterway for future tour boats; and maintaining the existing canal invert and water level elevations. Two existing stone piers created three spans of approximately 60 feet each. The piers were in excellent condition and reuse was required for financial, historic and aesthetic reasons. The new superstructure design therefore, needed to span those distances and maintain the original canal depth, but not restrict the highwater clearance in the creek below. The width of the new superstructure needed to bear on existing piers which could not be substantially widened. Finally, the entire superstructure needed to be retrofitted into existing stone masonry at a 60 degree skewed alignment.

As a National Historic Landmark, project review by the state historic preservation officer was required. A universally accessible towpath which could serve pedestrians, bicyclists, and horses was required to cross the three spans. The USDA Forest Service required the project to demonstrate timber as a modern construction material. A team goal was a modular timber structure in which members could replaced or repaired in place.

Aqueduct Design Development

Concept plans by the landscape architect employed a Town Lattice truss design with arches, similar to ten Town truss bridges still existing in Bucks County. DCF completed an analysis of the Town truss, but the results, coupled with early historical references pointed to the need for a stiffer truss. DCF chose a Burr Truss Arch for preliminary design analysis. Historic research turned up two existing versions of aqueducts; Duck Creek Aqueduct State Park in Metamora, Indiana; and the Roebling Aqueduct, a historic suspension structure across the Delaware River near the confluence of the Lackawaxen River in New York. The Duck Creek is a single span, Burr Truss Arch with a recently reconstructed simple timber trunk suspended below the bottom chords. The Roebling Aqueduct has been preserved and converted to a single lane vehicular bridge by the National Park Service. The new Tohickon Aqueduct could not be designed as a suspension structure; and a truss constructed above the trunk would create a series of new problems. SJ/DCF

Figure 10-Tohickon Aqueduct, 1994.

determined to place the trunk between the two trusses, and borrow the slanted wall trunk design from the Roebling Aqueduct. This configuration served two purposes: the sloped walls reduced the volume of water to be carried; and also served as lateral bracing for the trusses.

Protecting the superstructure was a primary concern. State engineers and SJ/DCF both believed that the new trunk could not depend upon historical timber construction to hold water. Inevitable leaks in a wooden trunk would directly compromise the new superstructure. A waterproof liner was selected as the best means to protect the investment in the timber superstructure - essentially creating a "covered bridge."

Aqueduct Engineering

Design Loads - The aqueduct design is based upon a uniform load of 4,000 PLF consisting mostly of water in the trunkway. The superimposed towpath loading was 100 PSF.

Redundant Design - Double Truss/Double Arch Based upon design load estimates and computer modeling, DCF engineered an interlocking truss/arch system that would work in concert under loading, but was capable of carrying the entire load by either trusses or arches independently. The truss was designed with structural timber posts and braces, and glued laminated upper and lower chords. The arches were designed to be a four leaf, laminated structural timber, two-hinged arch; stitch bolted together at regular intervals. The posts were notched to receive the arch.

Species Selection - Species selection varied for each member, depending upon exposure to weather, required preservative treatment, required precision tolerances, or direct contact with the public.

Truss Chords and Transverse Beams were specified as red oak, treated with Penta-WR. *Truss Braces, Longitudinal Sleepers, Trunk Posts, and Structural Trunk Decking* were specified as red oak, creosote treated. *Truss posts* were specified as creosote treated, recycled southern pine, due to anticipated exposure to weather and need for dry material to accept preservative treatment. Also, the diagonal brace seats cut into the posts were required to remain stable in the cross grain dimension to prevent shrinking and possible loosening of braces. *Truss Arches* were specified as white oak, untreated, except *bearing end laminations* against masonry foundations which were specified as red oak, creosote treated. *Towpath Decking, Railing and Trunk Facade Liner* were all specified as Southern Yellow pine, CCA treated. All joinery was specified to be performed before treatment.

Transverse Beams - The transverse beams were specified as glued laminated members due to load and span requirements. The beams bear upon lower chords and are suspended by rods from the arches. The ends of the beams fall within the structure's envelope and where exposed, are capped with copper.

Lateral Bracing/Trunk Construction - The slanted trunk posts are fastened to the transverse beams and the inside face of the upper truss chord. The trunk was designed to permit a twelve foot wide clear waterway inside the trunk facade at a depth of one foot. Channel design depth for water level was 4'-9", plus an additional foot to the top of abutment flume walls.

Membrane Liner - The membrane liner was specified as a reinforced, 45 mil polypropelene polymer geomembrane, with a non-woven polyester fiber geotextile bonded onto both sides. The liner was designed to cover inside the structural trunk walls and floor in a continuous length and seal to abutments. The design secured the liner to the structural trunk above the canal water level and lapped under the truss cap. A facade trunk liner was designed to protect the membrane liner and retain the timber appearance. The facade was designed as removable pallets, with wall sections that locked down floor sections, without fasteners perforating the membrane below water level.

Towpath - Similar to all previous versions, the towpath was cantilevered to the downstream side. Joists were designed to suspend out from the bottom of the top chord, braced by inclined posts from the bottom chord. The walkway was designed with a five-foot clear width. The top rail was sloped to the outside to

drain and to discourage walking, sitting, or resting objects on the handrail.

Truss Siding and Caps - A vertical board siding was chosen to relate to other covered bridges in the county. The siding was designed to be cut concentrically to cover the arches while exposing the structure of treated trusses to view. The accentuated arches relate to the two-span concrete arch of the River Road Scenic Highway bridge within view, 150 yards up Tohickon Creek. Truss caps were specified as painted steel roofing.

Figure 11-Elevation -Tohickon Burr Arch Aqueduct

Figure 12-Profile/Plan - Tohickon Burr Arch Aqueduct

Figure 13-Transverse Section - Tohickon Burr Arch Aqueduct

Hardware - All hardware was specified as either hot dipped galvanized or stainless steel.

Foundations - BFP determined that portions of both stone abutments required reconstruction to bedrock, using a new concrete leveling pad to rebuild native argillite stone walls. BFP designed new concrete sills to bear the new timber trusses and arches within the rebuilt stone walls. Similar sills were redesigned to top the two piers. BFP resolved to remove the top three courses of existing stone and replace them with a formed concrete sill/cap designed to accept the loads of both offset trusses and arches.

Hydraulic Controls - SJ recommended combining overflow and waste gates and locating the hydraulic controls in the abutments rather than historically in the trunk. This design keeps the trunk liner unperforated, and directs water releases away from the superstructure. The overflow/waste gate wells in both abutment flumes drain through piped outlets in the abutment wing walls.

Sitework - BFP redesigned canal approaches to the aqueduct, specifying a geosynthetic clay liner where the earth and the concrete abutment apron meet.

Construction Access - BFP determined that access would be required in the creek bed for both demolition of the existing concrete and steel flume, and construction of the new superstructure.

Maintenance - A primary design consideration was long term maintenance. Species selection, joint detailing, preservative treatment, periodic inspection and cleaning all serve to reduce maintenance, but repair or replacement of members will eventually be required. SJ/DCF attempted to anticipate those future needs and provide a system where members can be exchanged in place. Such a modular design provides a significant advantage of timber over concrete. Perforations in the plastic liner are easily repaired, while repairs in concrete are ultimately futile.

Flexibility - Another advantage is flexibility of a timber structure - even for a supposed uniform load. The new aqueduct may face repeated waterings and dewaterings due to frequent canal breaches and repairs scheduled in the upcoming years. After the initial test loading and unloading an aqueduct rebuilt several years ago as a reinforced concrete structure produced several hairline cracks a near the tops of the walls where reversals in stress are maximized. When the aqueduct is loaded these joints seep. Unloaded, these openings increase and will be on-going maintenance problems which require patching on a regular basis.

Conclusion

Although the Tohickon Aqueduct is certainly the largest and most complex, its development parallels the other two bridges. Each of the three bridges is a special, human-oriented structure, designed specifically for its special environment, by employing the best traditional forms, details, craftwork and modern timber technology. Not all bridge sites warrant this attention. But at special sites, which are subjected to increased expectations of a more sophisticated user population, the integrity of true truss bridges help to create a sense of place and add to tomorrow's engineering landmarks. The projects re-establish the partnership between landscape architect and structural engineer as designers of significant structures in the context of historic landscapes. The bridges help define a practical and conservative heritage reconstruction philosophy. Community interaction with government at every stage of a project means that the structures will have many stewards in their future.

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Yellow Poplar Glued-Laminated Timber: Product Development and Use in Timber Bridge Construction

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Abstract
This paper outlines the research efforts involved in incorporating yellow-poplar *(Liriodendron tulipifera)* as a viable resource for structural glued-laminated timber (glulam). The development of an efficient combination of yellow poplar glulam is presented, along with a description of how this was implemented in the industry standard for hardwood glulam. A case study is then presented that utilized this new technology by constructing a timber bridge.

Keywords: Yellow Poplar, Glulam, Timber Bridge, E-Rated Lumber

Introduction
Glued-laminated timber (glulam) is a vital element for many proposed timber bridge designs, and one key issue in bridge research is the use of local hardwood species that are underutilized for these type of applications. The efficient utilization of a hardwood species, such as yellow poplar, has been limited because established design methodologies for hardwood glulam are highly conservative.

Much research has been conducted to show that yellow poplar possesses properties that make it feasible for use in structural applications. In addition, technological advances in nondestructive testing has allowed for lumber to be graded on the basis of its mechanical properties, rather than visual grading alone (Green and others 1994). With the abundance of yellow poplar that grows in the northeastern United States, it would be logical to utilize this species to further extend the available forest resource.

Background
Provisions for the design of glulam timbers made from hardwood species of lumber have been available in a specification published by the American Institute of Timber Construction, AITC 119 (AITC 1985). The homogeneous combinations in this standard were available for five different levels of lumber quality. Design stresses applicable to yellow poplar are shown in Table 1.

These homogeneous combinations of glulam are referred to as axial combinations. In axial combinations, all laminations are subjected to similar stresses, such as tension or compression, or in bending members that have loading applied parallel to the wide faces of the laminations. For glulam made from softwood lumber, design properties for these types of combinations are available in the standard AITC 117-Design (AITC 1993). In the case of bending members that have loading applied perpendicular to the wide faces of the laminations, referred to as bending combinations, the opportunity arises to use lower quality lumber in the inner laminations because of the reduced stresses. For glulam made from

Table 1—Design properties for homogeneous combinations of glulam. Yellow-poplar glulam from AITC 119 standard (AITC 1985) and Southern Pine glulam from AITC 117 standard (AITC 1993).

Combi-nation symbol	Ratio of size of maximum permitted knot to finished width of lamination	Number of laminations	Extreme fiber in bending)		Horizontal Shear	Modulus of elasticity
Properties for yellow-poplar glulam						
			(MPa) (lb/in^2)	Steepest grain slope	(Pa) (lb/in^2)	(GPa) (x10^3 lb/in^2)
A	0.1	4 to 14	11.0 (1600)	1:16	1030 (150)	10.3(1,500)
	0.1	15 or more	11.0 (1600)	1:16	1030 (150)	10.3 (1,500
B	0.2	4 to 14	10.6 (1540)	1:16	1030 (150)	10.3(1,500)
	0.2	15 or more	11.0 (1600)	1:16	1030 (150)	10.3(1,500)
C	0.3	4 to 14	8.3(1200)	1:12	1030 (150)	9.3 (1,350)
	0.3	15 or more	9.1(1320)	1:12	1030 (150)	9.3 (1,350)
D	0.4	4 to 14	6.2 (900)	1:8	1030 (150)	8.3 (1,200)
	0.4	15 or more	7.2(1040)	1:8	1030 (150)	8.3 (1,200)
E	0.5	4 to 14	4.1 (600)	1:8	1030 (150)	8.3 (1,200)
	0.5	15 or more	5.2 (760)	1:8	1030 (150)	8.3 (1,200)
Properties for Southern Pine glulam[a]						
No.1D	0.27	4 or more	15.9 (2300)	1:10	1380 (200)	13.1(1,900)
No.2M	0.35	4 or more	12.1 (1750)	1:8	1380 (200)	9.7 (1,400)

[a] No.1D is combination number 50, and No.2M is combination number 47 (AITC 1993).

softwood lumber, available bending combinations are published in the standard AITC 117-Manufacturing (AITC 1993a), and their design properties are published in AITC 117-Design (AITC 1993). For glulam made from hardwood lumber, bending combinations were not recognized in the standard. As hardwoods were not recognized in AITC 117, and those combinations in AITC 119 used the same grade of lumber throughout, there were no opportunities for utilizing high and low grades of hardwood to develop efficient bending combinations.

Note that for combination A, Table 1, the maximum permitted knot is 10 percent of the finished width of the lamination. This knot restriction produces a lamination grade that exceeds Select Structural yellow poplar lumber, which allows a 20-percent maximum edge knot (NELMA 1991). For comparison purposes, the design properties for two grades of Southern Pine glulam are also included in Table 1. Note that design values for bending strength and stiffness are highly conservative for yellow poplar glulam, when compared with Southern Pine glulam having similar grade characteristics. A comparison between similar-knot categories, No.lD Southern Pine glulam (0.27) and combination C yellow-poplar glulam (0.3), shows that Southern Pine has a design bending strength that is approximately 1.92 times greater and design modulus of elasticity (MOE) that is 1.40 times greater. This comparison between No.2M Southern Pine glulam (0.35) and combination D yellow poplar glulam (0.4) yields ratios of 1.94 and 1.17, respectively.

Considering the large cost differential between hardwood and softwood lumber, and the limitation of the existing hardwood glulam standard, it would be difficult to justify the use of hardwood glulam other than for specialty applications. If yellow poplar glulam was available with properties similar to Southern Pine glulam, then it would have a better chance of being a competitive structural product. Better utilization of the hardwood lumber resource and establishing design properties of hardwood glulam that are competitive with those of softwood glulam is desirable.

Glulam Beam Research

In an effort to utilize yellow poplar as a structural material for glulam manufacture, a cooperative research project with West Virginia University (Moody and others 1993) was conducted to develop an efficient

bending combination using the existing design methodology established for softwood species of lumber (ASTM 1992).

Glulam Beam Design

The combination, shown in Figure 1, utilized E-rated lumber in the outer laminations and visually graded lumber in the core laminations. For stiffness, the outer 10 percent of the laminations was sorted for an average modulus of elasticity (MOE) of 13.8 GPa (2.0 x 10^6 lb/in^2) and the next inner 15 percent of the laminations was sorted for an average MOE of 12.4 GPa (1.8 x 10^6 lb/in^2). The remaining 50 percent of the core laminations was made from visually graded No.2 lumber. To ensure strength, edge-knot criteria were imposed on the E-rated lumber grades. For the bottom 10 percent of the laminations on the tension side, a maximum edge knot of 1/6 the width of the cross section was imposed; this is approximately equivalent to a Select Structural visual grade. All remaining E-rated lumber zones had a 1/3 edge-knot criteria, which is approximately equivalent to a No.2 visual grade (NELMA 1991). Thus, 90 percent of the cross section was made from No.2 and better lumber. These criteria followed the pattern found to be efficient in research that formed the basis for E-rated bending combinations in AITC 117 (AITC 1993a).

The standard design procedures for softwood glulam (ASTM 1992) indicated that the design values for this combination would be 12.4 GPa (1.8 x 10^6 lb/in^2) for MOE and 16.5 MPa (2,400 lb/in^2) for bending strength.

Glulam Beam Evaluation

To evaluate the performance of this design, Moody and others (1993) evaluated a total of 45 glulam beams having the combination shown in Figure 1. Three sizes were targeted: 8-, 12-, and 17-lamination beams made from nominal 5l- by 102-mm, 5l- by 152-mm, and 51- by 203-mm (2- by 4-in., 2- by 6-in., and 2- by 8-in.) lumber, respectively. Total length of these three sizes were 6.1, 9.1, and 12.2 m (20, 30, and 40 ft), respectively. In addition, samples of the end-jointed yellow-poplar lumber were also tested.

Table 2 gives the results of the yellow poplar glulam beams that were tested. The important properties to note are average MOE and the 5th percentile modulus of rupture (MOR) values, adjusted to a standard size of 130 mm wide, 305 mm deep, and 6.4 m long (5.125 in. wide, 12 in. deep, and 21 ft long).

Figure 1--Layup of yellow-poplar glulam beam made from E-rated outer laminations and visually graded core laminations.

Table 2—Results of yellow poplar glulam beam tests.

Property	Laminations 8	12	17
Sample Size	15	15	15
Moisture content (%)	8.2	7.5	8.0
MOE			
Average, GPa	13.0	13.4	12.3
(x10^6 lb/in^2)	(1.89)	(1.94)	(1.79)
COV (%)	4.5	3.3	3.4
MOR			
Average, MPa	55.6	52.1	45.3
(lb/in^2)	(8,060)	(7,560)	(6,570)
COV (%)	17.3	16.1	18.0
5th percentile, MPa	38.9	37.5	31.2
(lb/in^2)	(5,640)	(5,440)	(4,530)
Adjusted 5th, MPa	17.4	18.8	17.0
(lb/in^2)	(2,520)	(2,730)	(2,470)

In summary, all three beam sizes met or exceeded the targeted design levels of 12.4 GPa (1.8 x 10^6 lb/in^2) for MOE and 16.5 MPa (2,400 lb/in^2) for design bending strength (adjusted 5th). Thus, it was shown in this study that yellow poplar is a viable resource for structural glulam timber manufacture. Nondestructively testing the laminating lumber, or E-rating, enabled the utilization of lower grade lumber. An additional result of this study was that the existing methodology for designing softwood glulam beam combinations was applicable to hardwood glulam timber.

Hardwood Glulam Standard

This research on yellow poplar glulam was part of a combined research effort on hardwood glulam that enabled the American Institute of Timber Construction to take action on updating the existing hardwood glulam standard (AITC 1985). Studies by Manbeck and others (1993) and Janowiak and others (1995) on red maple and a study by Shedlauskas and others (1996) on red oak also showed that efficient bending combinations of hardwood glulam could meet the targeted 2400F-1.8E design level.

As a result of these studies, the AITC 119 hardwood glulam standard was revised so that efficient bending combinations were included. Table 3 shows two revised layups: 14F-Vl, which corresponds to combination B from Table 1, and 24F-E3, which corresponds to the targeted combination shown in Figure 1. Note that as the beam depth increases, stricter requirements are implemented in the tension laminations, which occupy the outer 5 percent of the laminations in the outer tension zone.

Table 4 shows the established design values for this combination. Note that for the 24F-E3 combination, the newly established design values for bending strength (tension zone stressed in tension) and bending stiffness are 50 and 20 percent larger, respectively, than the highest values available in the previous version of the standard (Table 1). In addition, the design shear strength value was adjusted to 1.07 MPa (155 lb/in^2), to be consistent with ASTM D3737 procedures.

Figure 2-Overall dimensions of McFarland Road bridge

Table 3—Revised bending combinations for yellow-poplar glulam (AITC 1996).

			Minimum grade of lamination				
			Percent/Grade/Species—Each Zone				
Combination symbol[a]	Depth of member	Tension lamination	Outer tension zone	Inner tension zone	Core zone	Inner compression zone	Outer compression zone
14F–V1	4 lams to <305 mm (<12 in.)	302-20	25% SS	--	N2	--	25% SS
	305 to 381 mm (12 to 15 in.)	302-22					
	>381 mm (>15 in.)	302-24					
24F–E3 Yellow Poplar	4 lams to <305 mm (<12 in.)	302-20	10% 2.0 E6/YP	15% 1.8 E3/YP	N2/YP	15% 1.8 E3/YP	10% 2.0 E3/YP
	305 to 381 mm (12 to 15 in.)	302-22					
	>381 to 572 mm (>15 to 22½ in.)	302-24					
	>381 mm (>22½ in.)	302-26					

[a]The following combination is not balanced and is intended for simple-span-members.

Table 4—Design values for yellow-poplar glulam bending combinations (AITC 1996)

	Bending about X-X axis					
	Loaded perpendicular to wide faces of laminations					
	Extreme fiber in bending, F_{bx}		Compression perpendicular to grain, $F_{c\perp y}$			
Combination symbol	Tension zone stressed in tension (MPa) (lb/in^2)	Compression zone stressed in tension (MPa) (lb/in^2)	Tension face (MPa) (lb/in^2)	Compression face (MPa) (lb/in^2)	Shear parallel to grain (Horizontal), F_{vx} (MPa) (lb/in^2)	Modulus of elasticity, E_x (GPa) (x10^6 lb/in^2)
YP						
14F–V1	9.7 (1400)	4.8 (700)	2.8 (405)	2.8 (405)	1.07 (155)	9.0 (1.3)
24F–E3	16.5 (2400)	8.3 (1200)	4.1 (590)	4.1 (590)	1.07 (155)	12.4 (1.8)

Thus, based on this research, the industry standard for hardwood glulam timber has been revised to include efficient bending combinations. These combinations have design properties equal to those for existing softwood glulam combinations, which provides designers the flexibility to choose a species that may be abundant to their particular region.

Timber Bridge Case Study

The timber bridge described in this case study is located on McFarland Road in Washington County, Maryland. This bridge was constructed in 1995 through the Demonstration Timber Bridge Program of the USDA Forest Service, administered by the Timber Bridge Information Resources Center in Morgantown, West Virginia. The timing of the recently revised hardwood glulam standard, along with the abundance of yellow poplar in the surrounding area, provided the opportunity for this bridge to be the first timber bridge to be designed using the efficient bending combinations of yellow-poplar glulam shown in Figure 1.

This project was a rehabilitation of an existing concrete-slab bridge. The deck had been severely deteriorated, having large sections of exposed rebar in some areas. In a previous attempt to extend the service life of the bridge, a temporary steel panel bridge had been placed on top of the existing concrete slab deck. This added to the height of the deck surface, which required that the approaches be raised to the new deck elevation, requiring a 16.1 km/h (10 mph) speed limit posting. With time, the steel panels became severely rusted as a result of annual exposure to road salts, and the bridge had substantial section loss at many points.

The rehabilitation project consisted of replacing the deck structure and using the existing substructure. The existing substructure consisted of cut-stone abutment stems and wingwalls with concrete caps. The depth of the abutment is approximately 5.5 m (18 ft), which provides ample hydraulic opening. The new deck superstructure (Fig. 2) consisted of 10 longitudinal glulam stringers, which were 7-m- (23-ft-) long and 89- mm- (3-1/2-in.-) wide yellow-poplar glulam diaphragms (Fig. 3). The bridge had an out-to-out width of 8.6 m (28.3 ft), with a road clearance of 7.5 m

Figure 3—Placement of glulam diaphragms.

Figure 4—Yellow-poplar glulam bridge railing

(24.7 ft). A 76-mm (3-in) bituminous wearing surface was applied to the surface of the 127-mm (5-in.) transverse yellow-poplar glulam deck.

This structure is a two-lane timber bridge that was designed to carry legal loads (HS20) and meet AASHTO requirements of L/500 for live-load deflections (AASHTO 1989). The design life of the superstructure is 50 years, and all components are treated with creosote. The preservative-treated beams were steam treated prior to erection to prevent bleeding.

The bridge railing, which was also made of yellow-poplar glulam timber, had an overall height of 838 mm (33 in.) (Fig. 4).

Figure 5–Final construction of McFarland Road Bridge.

Concluding Remarks

The development of efficient glulam combinations using yellow poplar provides another option for the construction of timber bridges using locally available underutilized wood species. Although there is little doubt that bridges constructed from this material will provide many years of acceptable service, the economics of hardwood glulam will be a determining factor in future acceptance.

The bridge in McFarland County was one of the first hardwood glulam bridges constructed, and costs were significantly greater compared with similar glulam bridges constructed of primary softwood species. This was expected and is typical when new materials and technology are initially introduced. As time passes and more bridges of this type are constructed, it is hoped that economics will improve and yellow poplar glulam will be a viable bridge material (Fig. 5).

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McCurdy Road Stress-Laminated Timber Bridge

A Viable Option for Short-Span Design

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Abstract

The McCurdy Road bridge is a 4.9-m- (16-ft-) long and 7.9-m- (26-ft-) wide single-span, stress-laminated deck superstructure, pressure treated with ammonical copper quat (ACQ) preservative. The bridge was constructed in June 1995 as the most economical replacement structure when compared with several alternative short-span designs. Results of this case study indicate that a stress-laminated timber bridge is a structurally and economically viable option for 3.1- to 6.1-m (10- to 20-ft) crossings. Future stress-laminated timber bridges in Richland County will be improved by utilizing lessons learned from the design, construction, and performance monitoring of the McCurdy Road bridge.

Keywords: Timber, bridge, stress laminated, preservative, economics.

Introduction

The United States is experiencing an infrastructure dilemma in which approximately 50 percent of all inspected bridges have been deemed structurally deficient or functionally obsolete. All states have adopted annual inspection programs to identify and rank bridges. The seriously deficient bridges are rehabilitated or replaced as necessary, while others are maintained as best as possible with available funds.

Short-span bridge replacements are a significant problem in the maintenance of local highway infrastructure. These bridges are defined as structures with total lengths between 3.1 to 6.1 m (10 to 20 ft) and may include multiple culvert crossings. Long-span bridges receive a greater portion of the available highway funds for repair and replacement. Therefore, the remaining short-span bridges in a local government inventory become a significant drain on the highway maintenance budget. However, all bridges must be maintained at the same minimum level because any bridge failure, long or short span, can lead to either road closure and/or personal injury.

The economics of short-span bridge maintenance are adversely impacted by Federal and State rules which, in most cases, exclude these bridges from cooperative programs intended to aid local agencies in their bridge replacement plans. Most cooperative programs require bridges to be at least 6.1 m (20 ft) long, which excludes most short-span bridges. This places the responsibility for design, construction, maintenance, and repair of these bridges on local governments.

A typical example of the impact of short-span bridges on the highway budget can be found in Richland County, Ohio. In this county, the bridge inventory numbers 361 bridges, of which 149 (41 percent) are considered short span. In this day of decreasing budgetary dollars, Richland County is forced to maintain nearly half their bridges without support from State and/or Federal funds.

One such short-span crossing in Richland County is the McCurdy Road bridge. This paper describes the development, design, economics, and construction of the McCurdy Road timber bridge. This single-span, stress-laminated timber bridge fulfilled Richland County's need for a low-cost, short-span replacement bridge.

Background

The McCurdy Road timber bridge is located in the southeastern part of Richland County, Ohio, approximately 0.4 km (0.25 mile) south of State Highway 97 on McCurdy Road (Fig. 1). This bridge spans a branch of the Babble Brook, which is a tributary of the Mohican River. The original crossing was a concrete box culvert built in the late 1930s (Fig. 2). After many years of use, this culvert was severely deteriorated, resulting in exposed steel and crumbling concrete. The bridge was posted with a 5.4-tonne (6-ton) load limit that restricted the crossing of heavy vehicles, which included fire equipment. Emergencies south of the bridge required a 9.7- to 12.9-km (6- to 8-mile) detour via Ashland County, which is located east of Richland County. In addition to the load restrictions, the cost of structure maintenance had increased beyond efficient means to maintain minimum traffic. Under the Ohio Department of Transportation (ODOT) condition grading system, this bridge was rated code 2 in 1993 (ODOT 1995). A code two designation is one step away from structure closure. (Code 1 represents structure closure.) Therefore, the McCurdy Road bridge was readied for replacement.

In examining possible replacement bridge types for the McCurdy Road bridge, considerations were made regarding economics because of limited funds. In addition, aesthetics were considered due to increased recreational activities in the area. Three alternative bridge designs were examined. The designs included three- and four-sided concrete culverts, corrugated metal arch pipe, and a stress-laminated timber bridge. In evaluating each design, several factors were examined: ease of construction, cost of construction, expected life of the structure, maintenance costs, and overall life of structure costs. All three designs adequately fulfilled the aesthetic requirement.

Concrete Culvert

A box culvert is considered a viable option for spans under 6.1 m (20 ft). In this case, three- and four-sided culverts were considered. The four-sided culvert was quickly eliminated, because the required size was not readily available. Special ordering would have increased the cost to prohibitive levels. A three-sided box culvert was available in the standard sizes required for

Figure1–Location of the McCurdy Road bridge.

Figure 2—Original McCurdy Road crossing in 1986.

this project. A spread footing and a pile foundation were considered as options, depending on the geotechnical examination.

The construction costs (discussed later) for the three-sided culvert with a spread footing were similar to the timber bridge option. However, long-term service life and maintenance cost estimates were not reliable, because long-term performance data are not available on these type of structures. As a result of the uncertainty of costs, this option was not selected.

Corrugated Metal Arch Pipe

The second option examined was the use of two 2.3-m- (90-in-) diameter corrugated metal arch pipes. This option was considered to be very economical during the construction phase of the project, because the material is relatively inexpensive and installation requires only light-duty equipment. However, this type of design requires increased maintenance costs over time and would have required complete replacement during the life cycle of the other crossings examined. Therefore, this option was not chosen.

Stress-Laminated Timber Bridge

A stress-laminated timber bridge was also examined as an option for bridge replacement. This type of bridge consists of a longitudinal deck superstructure, installed on timber pile abutments using timber backing boards and wing walls. This bridge was the most economical of the three types of designs examined. It also allowed the county to complete the installation in-house, because only light-duty equipment is required for construction.

After reviewing the three bridge options presented, Richland County personnel determined that a stress-laminated timber bridge would be an ideal bridge replacement. However, Richland County was unfamiliar with the design and construction of modern timber bridges; therefore, they applied for assistance from the Timber Bridge Information and Resource Center (TBIRC), which is part of the USDA Forest Service. With funds designated under the Timber Bridge Initiative (TBI), the TBIRC granted a portion of the total bridge cost to Richland County. Thus, the McCurdy Road bridge was included in the Forest Service timber bridge program.

Objective

The objective of this study was to examine the McCurdy Road timber bridge as a viable, low cost bridge replacement product. In addition, the economics of short-span timber bridge construction are examined.

Design, Economics, and Construction

The design and construction of the McCurdy Road timber bridge were a cooperative effort involving several organizations and individuals. The following presents an overview of the design, economics, and construction of the completed timber bridge.

Design

Design of the McCurdy Road bridge was completed by a consulting engineer retained by Richland County. Aside from those design aspects related to stress laminating, the bridge design was in accordance with the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (AASHTO 1992) for two lanes of AASHTO HS20-44 truck loading. Design criteria for stress laminating were based on the AASHTO *Guide Specifications for the Design of Stress Laminated Wood Decks* (AASHTO 1991).

The design geometry of the McCurdy Road bridge provided for a single-span, simply supported structure, 4.9 m (16 ft) long, 7.9 m (26 ft) wide, and 254 mm (10 in.) deep (Fig. 3). The stress-laminated deck consists of full-length, 51-mm (nominal 2-in.) Southern Pine lumber. Butt joints were not necessary in this deck because of the relatively short span.

Design values for the Southern Pine laminations were based on the *National Design Specification for Wood Construction* (AFPA 1991) for lumber visually graded No. 2 in accordance with Southern Pine Inspection Bureau rules (SPIB 1991). The tabulated design values for the Southern Pine species combination were 7,240 kPa (1,050 lb/in^2 for bending strength, 11,030 MPa (1,600,000 lb/in^2) for modulus of elasticity (MOE), 620 kPa (90 lb/in^2) for shear strength, and 3,900 kPa (565 lb/in^2) for compression strength perpendicular to grain. All design values were adjusted with the appropriate wet-use factors, and laminations were specified to be at or below 19 percent moisture content prior to preservative treatment and bridge installation.

The stressing system consisted of six high strength steel bars, which were inserted through predrilled holes at the center of the laminations. The design specified the use of 25.4-mm- (1-in.-) diameter threaded and galvanized steel bars with an ultimate tensile strength of 1,030 MPa (150,000 lb/in^2), which meets the requirements of ASTM A722 (ASTM 1988). The bars were not uniformly spaced along the face of the deck (Fig. 4). The non-uniform bar spacing creates uneven interlaminar compression along the length of the deck. Interlaminar compression ranged from 1,020 kPa (148 lb/in^2) at the deck ends to 737 kPa (107 lb/in^2) at the center of the deck, based on a design force in the bars of 178 kN (40,000 lb). Usually, 690 kPa (100 lb/in^2) is

Figure 3–Design configuration of the McCurdy Road bridge.

Figure 4–Stressing bar spacing on the McCurdy Road bridge.

the specified interlaminar compression along the entire deck. Steel bearing plates measuring 200 by 200 by 25.4 mm (8 by 8 by 1 in.), steel anchor washers, and spherical hex nuts were specified to anchor the bars along the deck edges.

Design of the bridge rail and curb system was based on AASHTO static-load requirements for vehicular traffic. The bridge rail and curb consisted of a 150- by 305mm (6- by 12-in.) sawn lumber rail and a 150- by 305-mm (6- by 12-in.) sawn lumber curb with 150- by 305-mm (6- by 12-in.) sawn lumber scupper blocks. The rail and curb were attached to 150- by 305-mm (6- by 12-in.) sawn lumber posts, spaced 1.8 m (6 ft) on center along the bridge edges, starting 0.3 m (1 ft) from the bridge end.

For protection from deterioration, all steel components, including stressing hardware, stressing bars, and anchorage plates, were galvanized in accordance with AASHTO M111 (AASHTO 1990). All wood components were treated with ammonical copper quat (ACQ) preservative in accordance with American Wood Preservers' Association (AWPA) Standard C14 (AWPA 1989). In addition, an asphalt wearing surface and fiberglass reinforced waterproofing membrane were specified for the bridge to protect the deck surface from premature deterioration caused by direct exposure to traffic and the elements.

Economics

The cost of construction of the McCurdy Road bridge was $61,809, including labor, materials, plant equipment, and engineering. Ninety-one percent of the total cost was associated with construction, and the remaining 9 percent was the result of engineering and other project support. A detailed cost breakdown is given in Table 1.

Table 1–Project costs.

Description	Cost ($)	Total (%)
Construction costs		
Labor	13,950	22.57
Equipment	11,628	18.81
Timber materials	27,677	44.78
Misc. bridge materials	2,574	4.16
Other materials	530	0.86
Subtotal	56,359	91.18
Other project costs		
Management	450	0.73
Engineering (design)	5,000	8.09
Subtotal	5,450	8.82
Total	61,809	100.0

A feature of the McCurdy Road project was that the bidding of timber materials included delivery, assembly drawings, and the rental of hydraulic jacks for stress laminating the superstructure. This practice, which is a modification of design building procedures, produced a suitable design at an estimated savings to the project of 25 percent.

The cost of the McCurdy Road bridge can be compared with four projects installed or planned by Richland County. These projects include a simple concrete box culvert similar in crossing size and hydraulic capacity to the original McCurdy Road bridge, a corrugated metal pipe culvert, and two larger concrete culverts designed similar to the roadway width of the McCurdy Road timber bridge.

Concrete Culvert– The first project includes the installation of a 1.8- by 4.3-m (6- by 14-ft) concrete box culvert very similar in size and capacity to the original McCurdy Road bridge. The basic culvert materials were bid, including installation by the supplier. The cost of the concrete box culvert was $71,000 for construction, which included materials and installation.

An additional $10,000 was expended for engineering, including design, project supervision, and bidding, bringing the total project cost to $81,000. When compared with the McCurdy Road timber bridge, the cost of this culvert is 33 percent greater.

Corrugated Metal Pipe Culvert– The second project includes the installation of a 2.3- by 3.6-m (90- by 140-in.) corrugated metal elliptical culvert pipe 12 m (40 ft) in length. Initial construction of this project was estimated to cost $38,400, with an additional $10,000 for engineering and project management. This brings the total cost of the initial installation to $48,400.

Many rural streams in Richland County are located in an agricultural area with corrosive ground water conditions. Corrugated metal pipes have experienced failure after 15 to 25 years under corrosive water conditions, making total replacement of this project a requirement within the 50-year design life. The total project cost for 50 years is likely to exceed $100,000, because total replacement would be necessary at least once. If cash flow is an immediate concern, the corrugated metal pipe option may be a viable alternative. However, the total 50-year cost exceeds the cost of the McCurdy Road bridge by 62 percent.

Larger Concrete Culverts– The third and fourth projects are concrete culverts designed to replace bridge structures on heavily traveled roads on the county highway system. The third project replaces a small bridge and is intended to accommodate a 7.3-m (24-ft) roadway width, which is equivalent to a 8.5-m (28-ft) bridge width. The total cost of $159,400 includes $129,000 for construction and $30,400 for engineering. This cost exceeds the McCurdy Road timber bridge cost by $97,590 or 158 percent,

The fourth project accommodates a 4.2-m (13.8-ft) roadway, widened by 0.6 m (2 ft) on the left for curved and super-elevated pavement. The equivalent bridge width is 7.3 m (24 ft). The total cost of $138,000 includes $94,000 for construction and $45,000 for engineering costs.

A comparison with the two larger concrete culverts was made, because they closely resemble the present width of the McCurdy Road timber bridge.

Construction

Construction of the McCurdy Road timber bridge was completed by the Richland County Engineer bridge construction staff in June 1995. The construction process included site preparation and installation of the abutments, superstructure, and wearing surface. A detailed account of each step of the process is presented as well as an explanation of the construction schedule.

Site Preparation– Site preparation included removal of the existing concrete box culvert structure, excavation of the channel and adjacent bank slope, and removal of the pavement, guardrail, and other incidental items. These activities were required to fit the new bridge in the crossing. Utility relocation was also performed in the initial site preparation phase.

Abutment Construction- Abutment construction started by driving twenty-two 30-cm (12-in.) Southern Pine timber piles (Fig. 5). The pile configuration included seven piles per abutment and two in each wing wall. The end pile on each abutment was used to support the inboard end of the wing wall, thereby eliminating the need for one pile on each wall.

Figure 5–Abutment construction.

Piles were driven to a calculated 22.68-tonne (25-ton) resistance and a minimum drive length of 3.1 m (10 ft). An alternate pile driving plan was available to provide for rock imbedment and concrete encasement of pile tips should it be determined that the 3-m (10-ft) minimum drive become impractical. Driving was done using a 11-kN (2,500-lb) drop hammer; however, a light diesel hammer could easily be substituted in this phase of the work. Each pile cap was constructed using two 30- by 30-mm (12- by 12-in.) timbers that were 4.9 m (16 ft) long. Pile caps were spliced over the center pile using two 75- by 300-by l,000-mm (3- by 12- by 36-in.) Southern Pine splice plates bolted through the pile caps.

The abutment construction process was completed by installation of the back wall, including the installation of filter fabric on the earth side and the partial backfilling of the abutment to increase construction stability. Installation of the timber caps on the wing walls and final backfilling of the abutment was deferred until completion of the superstructure to provide space for work and allow for final adjustment, if necessary.

Superstructure– Construction of the superstructure (deck) was started by placing laminations edgewise on the abutment caps. A total of 210, full-length, 51- by 254-mm (2- by 10-in.) laminations were used in the deck. After the steel bars were inserted through the predrilled holes in the laminations, the bearing plates, washers, and hexagonal nuts were installed on the bar near the deck edges. After all the stressing hardware was installed, bar stressing was initiated.

Bar stressing for the McCurdy Road bridge consisted of four full-design bar tensionings over 5 days. Initially, the bars were slightly tensioned to bring the laminations in uniform contact. Each bar was then stressed to 178 kN (40,000 lb) using a single hydraulic jack (Fig. 6). One day after the first stressing, the bars were again stressed to 178 kN (40,000 lb), following the same procedure as the first stressing. This stressing procedure was completed two additional times: 3 and 5 days after bridge installation. At the completion of the day 5 stressing, elongation of the bars was observed by measuring the bar extension behind the anchor nut.

Figure 6–Stress laminating the deck with a single hydraulic jack.

At the completion of the stressing process, the position of the deck was checked and minor adjustments were made. The deck assembly was then anchored to the pile caps using eight 20- by 610-mm (0.75- by 24-in.) anchor bolts on each end. After the deck was anchored, the timber rail system was attached to the outside edges of the deck.

Wearing Surface– Wearing surfaces are specified for each timber bridge deck in Richland County. Previous construction has shown that the detrimental effect of horse shoes, buggy wheels, tire chains, and other items seriously scar the deck surface and significantly accelerate deck degradation.

Richland County specifies a two-phase process using bituminous construction fabrics and 40 mm (1.5 in.) of hot-applied asphalt concrete pavement. Current practice is the use of Chase-Royston 10 AN Easy Pave ER[1] membrane glued to the deck using a solvent material designed for this purpose (Fig. 7). The application of the hot asphalt melts the bituminous material, creating a positive bond between the deck and the wearing surface.

After the wearing surface was applied, the bridge was opened for traffic. The completed bridge is shown in Figure 8.

Schedule– The construction schedule proceeded nearly as planned, with a minor delay caused by the spring weather conditions. The only construction-related delay was the delivery of the hydraulic jacks without the proper anchor bolts for the stressing bars. These two delays impeded completion by 7 days. Details of the completed structure are given in Figure 9.

Future Considerations

To assess the structural performance of the McCurdy Road bridge, Richland County contacted the USDA Forest Service, Forest Products Laboratory (FPL), for assistance. Through a mutual agreement, a 2-year monitoring program was developed by FPL and implemented through a Cooperative Research and

[1]The use of trade or firm names in this publication is for reader information and does not imply endorsement by the U.S. Department of Agriculture of any product or service.

Figure 7–Application of waterproof membrane.

Figure 6–Completed McCurdy Road bridge.

Development Agreement with the Richland County engineer. Under the monitoring plan, FPL and the Richland County engineer will monitor the moisture content, bar force, static-load behavior, and general structure condition of the McCurdy Road bridge (Ritter and others 1991). The FPL will publish a report of their findings after the completion of the monitoring period.

Discussion

Several lessons were immediately apparent during the construction and initial evaluation phase of the project. Technical information obtained and design opinions developed include considerations in the bearing and anchor plates, stressing bars, lumber treatment, and guardrail specifications.

Bearing Plates

The design of the McCurdy Road structure provided for 200- by 200- by 25.4-mm- (8- by 8 by l-in.-) bearing plates. Observation of the laminations during the stressing process showed signs of lumber crushing near the bearing plates. The use of larger bearing plates would have decreased or eliminated the amount of crushing.

Anchor Washers

Additional observation of the stressing process showed minor distortion of the washers used as anchor plates. Although not considered critical to the operation of the structure, the substitution of 75- by 75-mm (3- by 3-in.) anchor plates would provide an improvement in workmanship.

Bar Spacing

Bar spacing affects the amount of interlaminar stress introduced in the deck. On the McCurdy Road bridge, the non-uniform spacing combined with the uniform bar force created varying interlaminar stresses in the deck. These variations make it difficult to accurately measure and maintain the required amount of interlaminar stress in the deck. Typically, with decks of this thickness, a 610-mm (24-in.) bar spacing is used with 16-mm (0.625-in.) bars to obtain the required interlaminar stress. Further discussion of this subject will follow in an FPL research report, expected to be published in 1997.

Lumber Treatment

Lumber for this project conformed to industry standards of sawing, finishing, and drying prior to treatment. Because ACQ is a waterborne preservative, the post-treatment moisture content of the treated lumber was very high and was retained in the material at the time of construction. Observations during construction and the post-construction evaluation indicate that the high moisture content may contribute to lumber distortion.

It is suggested that the designer consider a modification of the lumber specifications to provide for post-treatment moisture control. This subject will also be addressed in detail in future FPL reports.

Guardrail

The guardrail assembly incorporated in the project included the use of a steel, deep beam rail face as a primary reinforcement and rub strip per ODOT standards. Evaluation of the completed structure shows that the aesthetic specification of the design objective may have been met by inclusion of the timber rub strip. This consideration does not affect the structure but should be considered to improve the appearance of future structures.

General

Name: McCurdy Road bridge

Number: RIC-TR-378-0.40

Structure Number: 7033591

Location: Richland County, Ohio

Date of Construction: June 1995

Design Configuration

Number of Spans: 1 (simple beam)

Structure Type: Stress-laminated deck

Total length (out-out): 4.9 m (16 ft)

Skew: None

Clear Span: 4.3 m (14 ft)

Bearing Length: 4.6 m (15 in.)

Width (out-out): 7.9 m (26 ft)

Width (curb-curb): 7.3 m (24 ft)

Number of Traffic Lanes: 2

Design Loading: AASHTO HS-20

Wearing Surface Type: Asphalt Concrete Pavement; 38 mm (1.5 in.) thickness, ODOT specification 404

Material and Configuration

Deck:
- Species: Southern Pine
- Size: 50 mm by 250 mm by 4.9 m (2 in. by 10 in. by 16 ft)
- Grade: No. 2 grade (visually graded)
- cut: S4S
- Preservative Treatment for all Wood Members: ammonical copper quat (ACQ)

Rails:
- Species: Southern Pine
- Size: 150 mm by 305 mm by 4.9 m (6 in. by 12 in. by 16 ft)
- Grade: No. 2 grade (visually graded)
- cut: S4S
- Rub Strip: Deep Beam Rail

Posts:
- Species: Southern Pine
- Size: 150 mm by 300 mm by 1.2 m (6 in. by 12 in. by 4 ft)
- Grade: No. 2 grade (visually graded)
- cut: S4S

Curb and Scupper:
- Species: Southern Pine
- Size: 150 mm by 300 mm by 4.9 m (6 in. by 12 in. by 16 ft)
- Grade: No. 2 grade (visually graded)
- cut: S4S

Stressing Bars:
- Type: Dwydag systems
- Diameter: 25.4 mm (1 in.)
- Number: 6
- Design Force: 178 kN (40,000lb)
- Spacing: Varies: 305 mm (12 in.), 760 mm (30 in.), 840 mm (33 in.), 1070 mm (42 in.), 840 mm (33 in.), 760 mm (30 in.), 305 mm (12 in.)

Anchorage Type and Configuration:
- Discrete Bearing Plate, Washer, and Hexagonal Nut
  - Bearing: 200 by 200 by 25.4 mm (8 by 8 by 1 in.)
  - Anchor: 50 mm (2 in.) washer and hex nut

Figure 9–Information sheet.

Concluding Remarks

Results of this study indicate that it is feasible to build a structurally adequate timber bridge that is economical for the 3.1- to 6.1-m (10- to 20-ft) span length. A comparison of cost shows that a short-span timber bridge structure is a viable option to a conventional concrete and steel bridge or large metal or concrete culverts. Application of a timber bridge design allows the engineer options that reduce cost and enhance construction capabilities for short-span bridges.

In addition, the McCurdy Road bridge project accomplished Richland County's two immediate objectives of providing a needed structure to the local commerce system and testing the capabilities of county forces to do the work with locally available equipment.

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Grant Administration: William D. Cunning, Operations Supervisor
Public Coordination: William Klinger, Drainage Supervisor
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Portable T-Section Glulam Bridge for Low-Volume Roads

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Michael A. Ritter, Forest Products Laboratory, USDA Forest Service

Abstract

Recent interest in portable bridge systems has increased as a result of a heightened awareness for reducing environmental impacts at road stream crossings. This paper describes the design and initial testing of a portable timber bridge consisting of two non-interconnected longitudinal glued-laminated timber (glulam) deck panels; each panel is 1.8 m (6 ft) wide and 12.2 m (40 ft) long. The deck panels are fabricated in a unique double-tee cross section. The panels exhibited linear elastic behavior and achieved 92% composite action under static bending tests. The complete bridge system appears to be cost effective with a superstructure cost of $\$381/m^2$ ($\$35/ft^2$). If the bridge is reused and installed at ten different sites, the estimated cost per site is $5,120, which is competitive with the cost of traditional stream crossing structures, such as fords or culverts.

Keywords: Portable bridge, Glued-laminated timber, Glulam, Bridge deck, Double-tee.

Introduction

Portable bridges have traditionally been used in military or construction applications. In typical civilian construction applications, portable bridges are used when a permanent highway bridge is being replaced and a temporary bypass is needed during the construction.

Portable bridges are also needed to serve as temporary structures during disaster situations, e.g. when a flood washes out a highway bridge. In addition, there are many situations where temporary access is needed across streams in remote areas for the construction or maintenance of utility structures.

Currently, much interest in portable bridge systems is occurring in the forestry and related natural resource industries. Access to our nation's forest resource requires an extensive roadway network over a wide spectrum of geographical conditions. In general, these roads are designed for low-volume traffic conditions and are often single lane and unpaved. Because forest management activities are both diverse and sporadic, traffic volumes and loads can vary significantly. During resource management periods, traffic volumes are low and consist primarily of light passenger vehicles. However, during forest harvesting operations, roadways may be subjected to higher-volume truck traffic with loads in excess of the maximum legal highway load. In either case, roadway use is commonly limited to short periods over a relatively long forest management cycle. For example, roadway access may be required for only 6 months during a 10-year cycle. As a result, there is a trend to close these roads when they are not needed for management activities.

Forest roads typically require a large number of structures to cross streams and other topographical features.

Rothwell (1983) and Swift (1985), in separate studies on forest roads and skid trails, found that stream crossings were the most frequent sources of erosion and sediment introduction into streams. Fords and corrugated-metal or concrete culverts have been common stream crossing structures on forest roads for many years. Thompson et al. (1996) reported that during the construction of a gravel ford on a stream approximately 1.8 m (6 ft) wide, peak sediment concentration in water samples taken downstream from the ford was nearly 2,810 mg/l greater than that of samples taken upstream from the ford. In addition to the disturbance caused during construction activities, using fords introduces sediment into the stream each time vehicles drive through the ford. Thompson et al. (1996) also showed that on the same ford, when light vehicular traffic drove through the stream, sediment concentration in water samples taken downstream from the ford was as much as 255 mg/l greater than that of the upstream samples.

Although some problems with fords are alleviated by culverts, there can be considerable sediment loads introduced into the stream during the excavation and fill work that accompanies culvert installation. Thompson (1996) reported that during the installation of a typical corrugated metal pipe culvert, sediment concentration in water samples taken downstream of the culvert was more than 950 mg/l greater than that of the upstream samples. Other results reported by Swift (1985) showed that the cumulative amount of soil placed in a stream at the road-stream crossing during construction was more than 10 times greater than the sedimentation during logging operations. In addition, culverts may clog with debris and wash out during heavy runoff periods, thereby introducing additional sediment into the stream. In the case of roads or skid trails that are not permanent, the stream crossing structure may be removed after logging operations or other activities are complete. Removal of a culvert also appears to introduce heavy sediment loads into the stream.

Historically, bridges for low-volume forest roads have been of two types: permanent or temporary. Permanent bridges, which are constructed of wood, steel, or concrete, depending on span requirements and economic considerations, are typically designed for service lives of 40 to 50 years. These permanent bridges are not economically feasible for short use periods and often require expensive maintenance for continued service. In addition, permanent bridges for limited-use, low-volume forest roads are commonly designed to a lower standard than most public access facilities and can be a potential liability to the bridge owner if public access is possible. A common temporary bridge has been the log stringer bridge that is either removed or left to deteriorate at the end of the use period. The use of temporary log stringer bridges has substantially declined over the last decade because of the difficulty in locating logs of the size and quality required for bridge construction. In addition, if the temporary bridge is not installed or removed properly, there may be adverse water quality impacts.

One solution to short-term bridge needs on low-volume forest roads is the concept of portable bridges. If properly designed and constructed, portable bridges can be easily transported, installed, and removed for reuse at multiple sites. This ability to serve multiple installations makes them much more economically feasible than a permanent structure. In addition, if they are installed and removed so that disturbance to the site is minimized, they can alleviate many water quality and other potential environmental problems. Thompson et al. (1995) reported that proper installation of a portable bridge could significantly reduce levels of sediment introduced into the stream compared with other crossings, such as fords and culverts.

Many advantages of timber bridges, which include using locally available materials, having long service lives, being relatively lightweight, and being prefabricated, make them ideal for portable stream crossings. The objective of this paper is to discuss the design and initial evaluation of a portable longitudinal glulam deck bridge. The bridge uses two non-interconnected panels that are fabricated in a unique double-tee cross section. Design, installation, and cost are discussed along with results of tests on the bridge components and the finished bridge.

Background

A variety of portable bridge designs have been constructed from steel, concrete, and timber, with steel and timber bridge designs being the most prevalent types (Mason 1990, Taylor et al. 1995). Although log stringer bridges and non-engineered timber mats or “dragline mats” have been used for many years, the recent advances in timber bridge technology include several engineered designs that can be easily adapted for use as portable bridges. Probably the most promising designs for spans up to 12 m (40 ft) consist of longitudinal glulam or stress-laminated decks that are placed across the stream. These designs can be quickly and easily installed at the stream crossing site using typical forestry equipment, such as hydraulic knuckleboom loaders or skidders. Also, it is possible to install these bridges without operating the equipment in the stream, which minimizes site disturbance and associated erosion and sediment load on the stream.

Hassler et al. (1990) discussed the design and performance of a portable longitudinal stress-laminated deck bridge for truck traffic on logging roads. This bridge was constructed of untreated, green mixed hardwoods. It was 4.8 m (16 ft)

wide, 12.2 m (40 ft) long, 254 mm (10 in.) thick and fabricated in two 2.4-m- (8-ft-) wide modules. Taylor and Murphy (1992) presented another design of a portable stress-laminated timber bridge. It consisted of two separate stress-laminated panels 1.4 m (4.5 ft) wide placed adjacent to each other with a 0.6-m (2-ft) space between panels. The overall width of the complete bridge was 3.3 m (11 ft). The panels could be constructed in lengths up to 9.7 m (32 ft).

Taylor et al. (1995) presented the results of using a portable longitudinal glulam deck bridge designed for use by logging trucks and other forestry equipment. It was 4.9 m (16 ft) wide and 9.1 m (30 ft) long. It used four glulam deck panels, 1.2 m (4 ft) wide and 267 mm (10.5 in.) thick. The bridge was designed to be installed on a mud sill with the bridge deck extending 0.6 to 1.5 m (2 to 5 ft) on either side of the stream banks, thereby leaving an effective span of approximately 6.1 to 7.9 m (20 to 26 ft). They concluded that the bridge performance was satisfactory and that if it could be reused at least 10 times, its cost was comparable or less than the cost of installing fords or culverts. It had an initial cost of $15,500 and an estimated cost per site of approximately $2,550.

Keliher et al. (1995) described the use of another longitudinal glulam deck bridge designed specifically for log skidder traffic. This bridge consisted of two glulam panels 1.2 m (4 ft) wide, 216 mm (8.5 in.) thick, and 7.9 m (26 ft) long. The glulam panels were placed directly on the stream banks and were not interconnected. They were placed by using the grapple on skidders or winching into place with a skidder or crawler tractor. This bridge performed well in service and was well received by forest landowners and loggers that used it. However, its relatively high initial cost of approximately $8,000 may discourage some users from purchasing this type of bridge over the non-engineered designs frequently used for off-highway vehicles.

Design, Installation, and Cost

Design

The portable longitudinal deck timber bridge designs discussed previously have been limited to spans of approximately 9.1 m (30 ft) because of practical limitations on the thickness of the deck panels. However, there is a need for more efficient technology to allow the use of portable timber bridges on spans up to 15.2 m (50 ft). Therefore, a longitudinal glulam deck bridge was designed and constructed in a double-tee cross section to test the feasibility of achieving longer spans for portable bridges, while retaining the concept of a longitudinal deck bridge. This bridge was purchased by Georgia Pacific Corporation and designed to be used as a portable bridge carrying log trucks and other forestry equipment. The bridge was manufactured by Structural Wood Systems, Inc. of Greenville, Alabama.

The bridge consists of two longitudinal panels, 12.2 m (40 ft) long and 1.8 m (6 ft) wide, giving a total bridge width of approximately 3.6 m (12 ft) as shown in Figure 1. The design vehicle for the bridge was an American Association of State Highway and Transportation Officials (AASHTO) HS20 truck (AASHTO 1993) with no specified deflection limitation. The panels are not interconnected, therefore, each panel is assumed to carry one wheel line of the design vehicle. The panels were designed to be placed side by side on a mud sill, which can be placed directly on the stream banks. Each panel was constructed in a double-tee cross section (dimensions given in Figure 2). Vertically laminated flanges were 171 mm (6.75 in.) thick and 1.8 m (6 ft) wide and fabricated using No. 1 Southern Pine nominal 51- by 203-mm (2- by 8-m) lumber. Two 286-mm- (11.25-in-) wide and 314-mm- (12.375-in-) thick webs were horizontally laminated to the lower side of the flange. The webs were fabricated using Southern Pine nominal 51- by 305-mm (2- by 12-in.) lumber that met specifications for 302-24 tension laminations (AITC 1993). The designers did not necessarily intend that webs in future bridges of this type be constructed using all 302-24 lumber. However, the laminator had a large supply of lumber in this size and grade and therefore chose to use it in this prototype bridge. At the ends of the bridge panels, the flange extended 0.6 m (2 ft) beyond the end of the webs. This extension of the flange was intended to facilitate the placement of the bridge panel on a mud sill.

Interior diaphragms measuring 286 mm (11.25 in.) wide and 210 mm (8.25 in.) thick were provided between the webs at three locations along the length of the panels: one at each end and one at midspan. In addition, to provide additional strength in the weak axis of the flange, 25-mm- (l-in.-) diameter ASTM Grade 60 steel reinforcing bars were epoxied into the glulam flange and the diaphragms. The reinforcing bars were placed in holes drilled horizontally through the flanges at the panel third points. Additional reinforcing bars were placed horizontally through the diaphragms near the ends of the panels.

At each end of the panels, 19-mm- (0.75-in-) diameter bolts were installed through the horizontal axis of the flange. At the inside edge of the flange, a 152- by 152- by 13-mm (6- by 6- by 0.5-in.) steel plate was attached to the bolts. At the outside edge of the flange, a 305-mm (12-in.) long 152-by 152-by 13-mm (6- by 6- by 0.5-in.) steel angle was attached to the bolts. Chain loops were welded to the square plates and the steel angles to facilitate lifting

Figure 1- Sketch of bridge installation showing overall dimensions of the portable longitudinal T-section glulam deck bridge.

of the panel ends and securing the panels at the site. The angles served as supporting brackets for a curb rail that extended the length of the bridge. Additional curb brackets were provided at third points along the outside edge of the flange. The curb rail consisted of a single 140-mm- (5.5-in-) deep, 127-mm- (5-in-) wide, and 11.6-m- (38-ft) long Southern Pine Combination 48 (AITC 1993) glulam beam, running the length of the bridge. For economic considerations, the curb rail was intended only for delineation purposes and not designed as a structural rail.

A wearing surface was not provided on the bridge. However, a 1.8-m (6-ft) long 152- by 102- by 13-mm (6- by 4- by 0.5-in) steel angle was attached with three 19-mm- (0.75-in-) diameter lag screws to the top face of the flange at each end of the bridge to prevent damage as vehicles drive onto the bridge. In addition, to prevent damage during installation of the bridge, a 6-mm- (0.25-in.-) thick steel plate was attached to the end of each web with 19-mm- (0.75-in-) diameter bolts. To facilitate lifting of the bridge panels, lifting eyes were placed 0.9 m (3 ft) from either side of the bridge panel midspan. These eyes consisted of a 51-mm (2-in.) inside diameter steel pipe with a 13-mm- (0.5-in-) thick steel plate flange welded to one end. The eyes were installed in holes drilled through the bridge deck flanges and attached using 19-mm- (0.75-in.-) diameter lag screws. The intent of the lifting eyes was to allow a chain or wire rope to be fed down through one eye and back up through the other eye to form a sling. Then, the ends of the chain or wire rope could be attached to a shackle or hook on a crane, loader, or backhoe. All steel plate, angles, lag screws, and bolts conformed to ASTM A36 or ASTM A307. A primer coat of paint was applied to all steel hardware before

Figure 2--Cross-section view of longitudinal T-section glulam deck panels with curb rail attached. Diaphragms and connectors are omitted for clarity.

installation.

The steel hardware was installed on the finished deck panels before they were shipped from the laminating plant. The deck panels were then shipped to a treating facility where they were preservatively treated with creosote to 194 kg/m^3 (12 lb/ft^3) in accordance with American Wood Preservers' Association (AWPA) Standard C14 (AWPA 1991). The treating process had no detrimental effect on the steel hardware and did not affect preservative penetration or retention in the wood under the steel. The installation of hardware before shipping to the treating facility allowed the finished bridge to be installed with no further fabrication or assembly on the part of the bridge owner.

Installation

The bridge was installed for the first time on March 14, 1996, on land owned by Georgia Pacific Corp. near Newnan, Georgia. Installation was completed by personnel from Georgia Pacific Corp. and a local construction contractor that was hired to build the logging road into the site.

Before construction began, mud sills were prefabricated by personnel from Georgia Pacific Corp. The sills were 762 mm (30 in.) wide and 4.9 m (16 ft) long and were constructed from nominal 152- by 152-mm (6- by 6-in) Southern Pine timbers that were bolted together with 19-mm- (0.75-in-) diameter bolts. The timbers were preservatively treated with chromated copper arsenate (CCA).

Installation began by clearing the road approach to one side of the stream crossing with a crawler tractor. The contractor then used a tracked backhoe to unload the bridge panels from a truck and place them in a clearing approximately 46 m (150 ft) from the stream crossing. The backhoe was used to level each stream bank and then reach across the stream to place the first mudsill on the far side of the stream. At this point, the backhoe was used to carry the first bridge panel from the clearing to the stream and place it on the mud sill. A chain was placed through the lifting eyes on the bridge panel and secured in a hook on the bucket of the backhoe to lift and carry the panel as shown in Figure 3. The backhoe placed the second panel in a similar fashion. After the second panel had been placed, the second mud sill was pushed under the bridge panel ends on the near side of the creek. It was not necessary to operate any equipment in the stream during the installation. Therefore, since the stream channel was not disturbed, water quality was not impacted during the installation.

Clearing the stream banks and placing the bridge panels was completed in approximately 2.5 hours. After the panels were in place, wire ropes were secured to the chain loops at each of the bridge corners and to nearby trees. This securing of the bridge required an additional hour. Additional time was also required to complete the final road approaches to the bridge. It is anticipated that removal of the bridge will be accomplished in a manner similar to the installation.

Cost

Cost for the materials, fabrication, treating, and shipping of the glulam bridge was $17,000. Based on a deck area of 44.6 m^2 (480 ft^2), the cost was approximately $381/m^2 ($35/ft^2). The cost for the mud sills was $600. The cost for labor and equipment to install the bridge was $1,680. Therefore, the total cost to install this bridge the first time was $19,280. Assuming removal costs are similar to installation costs, the projected total cost to install and remove the bridge at 10 different sites is approximately $33,600. When this is added to the initial cost of the bridge and mud sill, the total cost to install the bridge at 10 sites is $51,200 or $5,120 per site. This cost per site is competitive with the cost of installing permanent culverts or fords on the larger streams where this bridge will be used.

Bridge Evaluation Methodology

The monitoring plans for the bridge called for stiffness testing of the individual lumber laminations prior to the fabrication of the deck panels and the completed glulam deck panels after fabrication. In addition, static load test behavior and general bridge condition were assessed. These evaluation procedures are discussed in the following sections.

Lamination and Finished Panel MOE

Modulus of elasticity (MOE) tests were performed at the laminating plant prior to fabrication of the deck panels to determine the stiffness of each lumber specimen used in the flanges and webs. These tests were conducted using commercially available transverse vibration equipment. During the tests, an identification number and the MOE was placed on each lumber specimen to facilitate resorting the lumber at a later time.

After fabrication of the deck panels, static bending tests were conducted to determine the apparent MOE values of each panel. These bending tests were conducted using a testing frame at the laminating plant and consisted of applying a single point load at the center of each deck panel. A steel beam was used to distribute the load across the width of the flange. The panels were placed in the test jig as they would be installed in the field, i.e., the bearings

Figure 3-Bridge panels being lifted into place (left) and the completed bridge installation (right).

were placed under the flange overhangs at the end of the bridge immediately adjacent to the end of the webs. This resulted in a test span (from center of bearing to center of bearing) of 11.28 m (37 ft).

During testing, deflection readings were taken with dial gages and linear-variable displacement transducers (LVDTs) at several locations along the length of the panels: at each bearing, approximately 610 mm (24 in.) from each bearing, and at midspan. Force was applied to the panel using a hydraulic cylinder and measured by a load cell placed between the hydraulic cylinder and the deck panel. During the tests, the force was steadily increased to approximately 66.72 kN (15,000 lb) with deflection readings taken at 11.12 kN (2,500 lb) intervals. The maximum force used in the tests resulted in a bending moment approximately 70% of the design moment. Deflection readings were recorded to the nearest 0.025 mm (0.001 in.). These force and deflection data were then used to calculate the apparent static bending MOE of the deck panels.

Analytical Assessment of Bridge Panel MOE

At the conclusion of the stiffness testing of the lumber, a target "E-rated" layup was developed for the flanges and webs. This layup, which is shown in Figure 4, consisted of five different lumber groups. The MOE values shown in Figure 4 represent the target mean MOE of the lumber used in the flanges or the various laminations of the webs. Personnel in the laminating plant were able to sort the lumber into the different MOE classes and place the lumber laminations in the desired panel locations during the manufacturing process. The identification numbers for the boards used in the flanges and the identification numbers and locations of each board used in the webs were recorded during the fabrication process. These data were then used as input for a transformed section analysis computer program developed at FPL, which included an estimate of the effect of shear deflection. Using lumber data, the program was used to predict the MOE of the finished deck panels for comparison with bending test results.

Load Test Behavior

Static load testing was conducted June 4, 1996, approximately 3 months after bridge installation. The test consisted of positioning a fully loaded truck on the bridge deck and measuring the resulting deflections at a series of transverse locations at midspan and the abutments. Deflection measurements were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded).

The load test vehicle consisted of a fully loaded tandem-axle dump truck with a gross vehicle weight of 161.1 kN (36,220 lb) and a track width at the rear axles of 1,829 mm (72 in.) (Figure 5). Measurement of wheel line loads indicated that the right side of the rear axles was approximately 4.4 kN (1,000 lb) heavier than the left side. The vehicle was positioned longitudinally on the bridge so that the two rear axles were centered at midspan. This resulted in maximum bending moments approximately 55% of the design moment. Transversely, the vehicle was placed for four load cases as shown in Figure 6. For load cases 1 and 3, the vehicle wheel line was positioned directly over the panel outside web. For load cases 2 and 4, the vehicle was positioned with the truck wheel line over the flange centerline at the center of the panel width. Measurements of bridge deflection from an unloaded to loaded condition were obtained by placing calibrated rules

Figure 4-Theoretical distribution of lumber MOE classes in the T-section glulam deck panels.

on the underside of the deck, and at the abutments and reading values with a surveyor's level to the nearest 0.2 mm (0.01 in.).

Condition Assessment

The general condition of the bridge was assessed at the time of the first load test. This assessment involved visual inspection of the bridge components, measuring moisture content of the wood members with a resistance-type moisture meter, and photographic documentation of bridge condition. Items of specific interest included the condition of the top surface of the deck panel flanges, the bottom face of the webs, the curb system, and anchorage systems.

Results and Discussion

The performance monitoring of the bridge is still in its initial stages and will continue for 2 years. Results and discussion of the initial performance data follow.

Lamination and Bridge Panel MOE

The lumber used to fabricate the flanges was nominal 50 by 203-mm (2- by 8-in.) No. 1 Southern Pine. Results of MOE tests on this lumber prior to gluing indicated that it had a mean flatwise MOE of 18,130 MPa (2.63 x 10^6 lb/in^2) with a coefficient of variation of 17.3%. The flatwise MOE can be converted to an edgewise value by applying a flatwise adjustment factor of 0.965 (Williams et al. 1992). This resulted in a mean edgewise MOE of 17,510 MPa (2.54 x 10^6 lb/in^2).

The lumber used to fabricate the webs was nominal 50-by 305-mm (2- by 12-in) Southern Pine graded at the laminating plant to meet the specifications of 302-24 tension laminations (AITC 1993). Results of MOE tests

Figure 5-Load test truck configuration and axle loads. The transverse vehicle track width, measured center-to-center of the rear tires, was 1.8 m (6 ft).

on this lumber prior to gluing indicated that it had a mean flatwise MOE of 16,960 MPa (2.46 x 10^6 lb/in^2) with a coefficient of variation of 12.5%.

Bending test data were used to calculate the MOE of the two finished bridge deck panels. For panel 1, the MOE was 16,340 MPa (2.37 x 10^6 lb/in^2). For panel 2, the MOE was 15,860 MPa (2.30 x 10^6 lb/in^2). Based on the force-deflection plots from these tests, the deck panels appeared to exhibit linear elastic behavior up to the maximum loads used in the tests.

Analytical Assessment of Bridge Panel MOE

Data for the location of each board and its corresponding MOE were used as input to the transformed section program. This analysis assumed that there was complete composite behavior in the double-tee deck panel and that the cross section of the panel was uniform along the entire span. The latter assumption was not entirely accurate since the webs were tapered near their ends. Based on the transformed section analysis of the original target E-rated layup, the theoretical predicted MOE for the deck panels was 17,860 MPa (2.59 x 10^6 lb/in^2). When the actual lumber MOE data were used in the transformed section analysis, the predicted MOE for Panel 1 was 17,720 MPa (2.58 x 10^6 lb/in^2) versus an actual MOE of 16,341 MPa (2.37 x 10^6 lb/in^2). For Panel 2, the predicted MOE was 17,250 MPa (2.50 x 10^6 lb/in^2) versus an actual MOE of 15,860 MPa (2.30 x 10^6 lb/in^2). The difference between the actual MOE and the predicted MOE indicates that the actual beam stiffness was approximately 92% of the theoretical stiffness (which assumes a uniform cross section along the entire span). This is primarily due to the tapered webs at the ends of the actual deck panels. It is also possible that there was a loss in stiffness due to shear lag in the flange. Additional experimental testing and analysis will investigate this further in order to optimize the efficiency of the system.

Load Test Behavior

Transverse load test deflections are shown in Figure 7, as viewed from the south end (looking north). For each load test, no permanent residual deformation was measured at the conclusion of the testing. In addition, there was no detectable movement at either of the abutments. For load cases 1 and 3, the symmetry of loading resulted in deflection profiles that are approximately mirror images of one another and deflection differences of corresponding data points for the two positions are within approximately 1 mm (0.04 in.). Maximum deflections for these load cases occurred in panel 1 and measured 16.2 mm (0.64 in.) at the outside panel edge for load case 1 and 16.5 mm (0.65 in.) at the interior panel edge for load case 3. It is probable that the maximum deflection for load case 1 occurred at the interior edge of panel 2; however, deflections at that point were not measured. The greater deflections recorded at the outside panel edges are expected because the truck was loading the flange about the weak axis.

For load cases 2 and 4, deflections are nearly identical and differences at corresponding data points for the two load cases are within 1 mm (0.04 in.). With the wheel line centered on panel 1 for load case 2, the approximately uniform load distribution across the panel width results in similar deflection at each data point. For load case 4, it was anticipated that the panel 2 deflections would also be uniform and approximately equal those for panel 1, load case 2. The approximate 2.5 mm (0.10 in.) difference in the load case 4 web deflections for panel 2 is likely due to minor differences in the truck transverse position. The maximum deflection recorded for load case 4 corresponds to a deflection value of approximately L/975, at 55% of design bending moment.

Condition Assessment

At the time of the first load test, limited traffic ,had crossed the bridge. Therefore, there was little overall change from the bridge's original condition. There was a small amount of damage to the outer face of the tension lamination of one web; however, the damaged area did not appear to significantly reduce the structural adequacy of the bridge. This apparently occurred during preparation for installation when the panel was dragged on the ground. The small amount of overall damage may be attributed to the use of the lifting eyes, which eliminated the need for the construction crew to wrap chains or cables around any exposed wood surfaces. Some surface checking was noticed on the top surface of the flange, but it did not appear to affect the structural adequacy of the flange. There were locations where excess creosote had accumulated on the top surface of the flange.

Figure &Transverse load positions (looking north) and deck panel numbers for the load test. For all load cases, the two rear axles were centered over the bridge centerspan.

Concluding Remarks

Based on initial testing, the longitudinal T-section glulam deck bridge is performing well and should provide acceptable service as a portable logging bridge. The following specific conclusions can be made at this time:

- It appears to be feasible and practical to construct a longitudinal glulam deck panel as a double-tee.
- The total time to install the bridge was less than 3.5 hours. Installation was easily accomplished using common construction equipment, and because the stream channel was not disturbed, water quality was not impacted during construction activities.
- The cost of the bridge superstructure was \$381/m^2 (\$35/ft^2), which is competitive with other timber bridge superstructure systems. The estimated cost for

Figure 7-Transverse deflection for the load test of the longitudinal T-section glulam deck bridge measured at the bridge centerspan (looking north). Bridge cross-section and vehicle positions are shown to aid interpretation and are not to scale.

installation and removal of the bridge at 10 different sites was $5,120 per site, which is competitive with other traditional stream crossing structures on similar size streams.

- Static bending test results indicated that the T-section glulam deck exhibited linear elastic behavior when subjected to loads approaching the design load. The stiffness of the deck panels was approximately 92% of the theoretical stiffness, based on results from a transformed section analysis that assumed a uniform cross section along the entire span.

- Results from load tests indicated that the T-section glulam deck exhibited acceptable levels of deflection. The maximum midspan deflection recorded when the truck wheel line was positioned near the center of the panel was equivalent to L/975 at 55% of design bending moment.

- Minor damage to one tension lamination apparently occurred during the installation process. Also, minor checking was observed on the surface of the flange. Long-term observations are needed before determining if these factors will affect the structural adequacy of the deck panels.

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Glued-Laminated Timber Panels for Bridge Deck Replacement

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Abstract

Research was conducted to evaluate the viability of using Oklahoma timber species in glued-laminated panel designs for bridge deck replacement. Panels were fabricated from southern pine, red oak, and cottonwood. Laboratory tests were conducted to determine the mechanical properties of panels fabricated from the three different species and allowable spans were computed based on test results. Panels were installed on seven in-service bridges ranging in width from 4.82 to 6.89 m (15.8 to 22.6 ft), and in length from 9.5 to 37.2 m (31 to 122 ft). Advantages of timber panels relative to other decking materials include reduction in out-of-service time, easy installation, reduced dead loads, increased resistance to road chemicals, and a wider tolerance to weather conditions during installation.

Keywords: Bridge deck, glued-laminated, wood, timber, southern pine, red oak, cottonwood

Introduction

The condition of the nation's rural bridges is of high concern within the transportation industry and general public (Walzer and Chicoine 1989). Of specific concern are rural bridges located in agriculturally significant counties, because they exhibit more structural and functional inadequacies than those in other locations (Marathon 1989). This is particularly true for the agriculturally-oriented state of Oklahoma, in which 57 % of the 20,682 rural highway bridges are considered structurally deficient or functionally obsolete (Anderson et al. 1994).

The bridge deck, being exposed to both weather and vehicular traffic, is one of the more vulnerable components of the bridge system. An analysis of the Oklahoma Department of Transportation bridge inventory system indicates that there are 5,291 bridges with decks in fair or worse condition (Oberlender and Vonkarey 1993). Low-cost, low-maintenance alternatives are needed by county governments to improve the rural transportation infrastructure.

During the last 20 years, extensive engineering research has led to significant advances in the design and use of glued-laminated timber for bridge construction (Williamson 1990). Although economic analyses of timber versus traditional steel and concrete bridges are few, indications are that timber bridges are cost competitive (Behr et al. 1990). In general, timber bridges built in those areas of the country where timber

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species are produced have a cost advantage over other areas (Leichti 1992).

Research was conducted to evaluate the viability of using Oklahoma timber species in glued-laminated panel designs for bridge deck replacement. Specific objectives were to: (1) determine strength characteristics of panels fabricated from Oklahoma-grown wood species (southern pine, red oak, and cottonwood); and (2) determine the acceptability of these wood species for bridge-deck installations under varying conditions.

Raw Materials

Locating and obtaining raw materials for the test and bridge panels was of concern during the project. Little difficulty was encountered in obtaining No. 2 common or better 2x4 and 2x6 dimension lumber of southern pine. Initially, kiln-dried material of southern pine was obtained from local merchants within two to four weeks of placing an order. Longer delays were encountered towards the end of the project as prices for southern pine lumber had increased almost 60% and No. 1 common lumber was becoming more difficult to obtain.

Adequate raw material of cottonwood and red oak was more difficult to obtain. No local merchants had kiln-dried material readily available. Subcontracts were made with a local sawmill to provide both 2x4 and 2x6 stock of random lengths. Initial orders were for No. 1 common or better cottonwood and No. 2 common or better oak. This material was then transported to a kiln-drying facility where it was dried to a target moisture content between 12% and 15%. Delays were experienced in both the cutting and kiln-drying phases. Excessive variation in board thickness and width resulted in difficulties during fabrication. Height differences between mating boards within a panel were of primary concern. Warping of both cottonwood and red oak boards was also evident following the drying process. Excessive variation in board dimensions combined with warping of boards and the normal defects found in lumber resulted in elimination of a substantial number of boards.

Surfacing of cottonwood and red oak boards was required to reduce dimension variability between boards. This added to the total cost of panel fabrication. To reduce the number of cull boards, orders for raw materials towards the end of the project were specified No. 1 common or better. This did not successfully address the problems with red oak. Future attempts to obtain kiln-dried cottonwood and red oak will require additional administrative efforts to ensure raw material quality.

Fabrication

Panels were fabricated by the industry cooperator, Rocking Horse Timber Bridges Inc., located in Seminole, Oklahoma. All test panels and demonstration bridge decks were built with raw materials from Oklahoma. Finger joints were glued using Borden "Cascomel" MF-684U adhesive mix on southern pine, and the adhesive mix plus a 5% Perkins GF-10 bonding agent on cottonwood and red oak. All finger joints were cured in a radio-frequency machine at about 93°C (200°F). Face-gluing of the lumber was accomplished with a mix of Borden "Casophen" Lt-5210 resin and "Cascoset" FM-6210 slurry. Both finger-joint and face-joint adhesives conformed to ASTM D 2559-84 standards and met the requirements of ANSI/AITC A 190- 1 - 1983. All AITC requirements were met except for having a certified lumber grader or trained quality control supervisor on staff.

The original clamping system for holding the boards during gluing consisted of hand tightened clamps 0.3 m (1 ft) on center along both long edges of the 1.2-m (4 ft) timber panels. Clamps were tightened to a torque of approximately 237 Nm (175 ft lbs). Rods extended between these clamps. The configuration required two panels in order to provide concentric pressures. All test panels plus five of seven demonstration bridge decks were fabricated using this system. Shortcomings of this clamping system resulted in height differences between mating boards, bowing across the 1.2-m (4 ft) dimension, and face-glued surface failures. These failures were corrected with a hydraulic clamping system designed and constructed by the Biosystems and Agricultural Engineering Department at OSU. The 27 kN (3 ton) movable A-frame is capable of accommodating 1.2-m (4 ft) wide panels up to 9.1 m (30 ft) in length. Eight hydraulic cylinders placed 1.2 m (4 ft) apart apply approximately 8.3 MPa (1200 psi) pressure to the panel. This system was used for fabricating panels for the last two demonstration bridge decks.

Preservative Treatment

Creosote treatment of the test and bridge deck panels was contracted to treatment facilities in Oklahoma. Panels were placed in long containers for pressure treatment at approximately 970 kPa (140 psi). Treatment required approximately eight hours and followed American Wood Preservers Association standards. Treated panels were allowed to drain for several days prior to testing or bridge-deck installation.

Laboratory Tests

Specimens

Test panels were fabricated from lumber graded No. 2 common or better. A total of 36 panels were built for the test program, with one-third from southern pine, one-third from red oak, and one-third from cottonwood. Half of the test panels were built with 2x4 boards and had final nominal dimensions of 10 cm (4 in.) thick, 1.2 m (4 ft) wide, and 3.0 m (10 ft) long. The other half of the panels were built from 2x6 boards and had final nominal dimensions of 15 cm (6 in.) thick, 0.6 m (2 ft) wide, and 3.0 m (10 ft) long. The 1.2-m (4 ft) wide panels typically contained 34 laminations, and the 0.6-m (2 ft) wide panels contained 17 laminations. Laminations contained one fingerjoint along their length, with approximately one-third of the fingerjoints in a panel falling in the maximum moment region. One of the 2x4 red oak panels was severely damaged in shipping and is not included in the reported data.

Static bending tests were conducted according to ASTM D198-84. The first test on each panel was conducted on the full panel, loaded at the third points to produce stress parallel to the laminations. A schematic of the load frame is shown in Figure 1. Longitudinal bending tests were conducted under displacement control to beyond ultimate load. Load and displacement were monitored continuously throughout the test.

After a panel had been tested in longitudinal bending, two 10-in. (25.4 cm) wide strips were cut from one end of the panel. These strips were tested by loading at the third points to produce stress perpendicular to the laminations. In these transverse bending tests, span length between supports was 46 cm (18 in.) and only maximum load was recorded. A total of 32 (20 2x4, 12 2x6) southern pine, 34 (31 2x4, 3 2x6) red oak, and 24 (12 2x4, 12 2x6) cottonwood specimens were tested in transverse bending.

Results

For each longitudinal bending test, modulus of elasticity (MOE) was computed in accordance with ASTM D3737-89a and modulus of rupture in accordance with ASTM D198-84. The mean and standard deviation of the test data for each species are shown in Table 1, with data for 2x4 and 2x6 panels combined. A statistical analysis of the data indicated no significant difference in the means for these two groups. Lower five percent exclusion limit (L5%EL) values were computed as suggested in ASTM D2555-88. Lower five percent exclusion limits were used to compute the allowable design values in Table 2, following procedures suggested in ASTM D3737-89a. Allowable design values include a factor of safety and an adjustment to normal duration of loading. Allowable design values computed from experimental data are compared to values from the National Design Specification (NDS) Supplement (National 1988).

Figure 1 - Schematic of setup for longitudinal bending tests

Table 1 - Results for Longitudinal Bending Tests (1.0 psi = 6.89 kPa)

Property	Southern Pine	Red Oak	Cottonwood
Modulus of rupture (psi)			
Mean	6,570	6,590	6,660
Standard deviation	422	788	511
L5%EL	5,870	5,230	5,820
Modulus of elasticity (x 10^6 psi)			
Mean	1.56	1.40	1.60
Standard deviation	0.098	0.169	0.086

Table 2 - Allowable Design Values for Longitudinal Bending (1.0 psi = 6.89 kPa)

Property	Southern Pine	Red Oak	Cottonwood
Bending stress F_{byy} (psi)			
Test result	2,080	1,690	1,880
NDS	1,750	2,240	1,240
Modulus of elasticity (x 10^6 psi)			
Test result	1.6	1.4	1.6
NDS	1.4	1.6	1.2

Table 3 - Results for Transverse Bending Tests (1.0 psi = 6.89 kPa)

	Southern Pine		Red Oak		Cottonwood	
Statistical property	2x4	2x6	2x4	2x6	2x4	2x6
Mean (psi)	190	103	223	63	409	368
Standard deviation (psi)	74	48	118	34	137	126
L5%EL (psi)	68	24	29	7	184	161

Results for transverse bending tests are shown in Table 3, with values for 2x4 and 2x6 panels reported separately. The 2x4 and 2x6 data were not combined because of substantial differences in the means.

Observations

Examination of the data presented in Table 1 reveals that the mean bending strengths of panels made from the three different species are not substantially different. However, the standard deviation for the red oak panels is higher than for the other two species. Also, stiffness of the red oak panels, as measured by apparent MOE, is less than stiffness of panels made with southern pine and cottonwood. This may be due to the fact that several of the red oak panels were noticeably cupped when they were delivered for testing. In this cupped condition, the full width of the panel did not contact the supports in the loading frame. As load was applied, panels were gradually forced into full contact with supports, sometimes by longitudinally splitting the panel along a lamination. Once the panel was in contact across its full width, stiffness increased, but the net result was a more flexible panel.

In Table 2, it can be seen that the strength and stiffness of the southern pine panels tested in the Structural Engineering Laboratory at OSU are respectively, 19% and 14% higher than NDS values; tested strength and stiffness of red oak are 25% and 13% less than NDS values; and tested strength and stiffness of cottonwood are 52% and 33% higher than NDS values. Also note that the allowable design bending strength based on test results for red oak is lower than for southern pine and cottonwood, even though the mean strengths in Table 1 are approximately the same. The computation of allowable design values includes a consideration of variability in the data. The higher standard deviation for the red oak pulls the allowable design strength for this species to below that for the other two species.

The underlying cause of the lower than expected allowable design values for red oak and the higher values for cottonwood may, in part, be explained by

the transverse-bending strengths reported in Table 3. The cottonwood transverse bending strength is much higher than the transverse-bending strengths of southern pine and red oak. The mean transverse bending strength of the red oak panels is similar to the southern pine panels, but there is more variability in the data. It may be that the soft, porous cottonwood absorbs a portion of the glue applied to its surface, and produces a very strong connection between laminations. The much harder red oak did not appear to absorb the glue, and produced very poor connections between laminations. In support of this hypothesis, it was observed during testing that the red oak panels regularly failed in transverse bending by breaking between laminations, and cottonwood typically failed by breaking within laminations. Both the density and performance of the southern pine panels fall between red oak and cottonwood.

Allowable Spans

The design procedure presented by Ritter (1990) is used to determine allowable spans. Calculations are based on bending strengths and stiffnesses reported here, and on allowable horizontal shear strengths published by NDS. Allowable horizontal shear strength from NDS for southern pine is 1.21 MPa (175 psi), for red oak is 1.59 MPa (230 psi), and for cottonwood is 0.758 MPa (110 psi). Design live load is an HS20-44 truck, live load displacement is limited to 2.5 mm (0.1 in.), and wet-use conditions are assumed. It is also assumed that supporting stringers are at least 15 cm (6 in.) wide.

Allowable spans are given here only for southern pine and cottonwood panels. The fabrication quality and performance under load of the red oak panels was so erratic that recommending a general-purpose allowable span is not appropriate. The allowable clear span between stringers for a 2x4 southern pine panel is 79 cm (31 in.), for a 2x6 southern pine panel is 130 cm (52 in.), for a 2x4-in. cottonwood panel is 45 cm (17.5 in.), and for a 2x6 cottonwood panel is 110 cm (43 in.) (Wang 1992). Allowable spans for the 2x4 southern pine panel and both cottonwood panels are governed by horizontal shear. The allowable span for the 2x6 southern pine panel is governed by live load deflection. Computed allowable spans are sufficient to permit both thicknesses of both species to be used on county bridges.

Field Installations

Deck panels were fabricated from No. 2 or better lumber. Substructure-stringer spacing dictated whether 2x4 or 2x6 lumber was used in fabricating the 4-ft (1.22 m) wide replacement panels for each bridge deck. A total of seven demonstration bridge decks were installed between April 1992 and September 1993. Panel lengths (bridge widths) ranged from 4.82 to 6.89 m (15.8 to 22.6 ft), and bridge lengths ranged from 9.5 to 37.2 m (31 to 122 ft).

Bridge Deck 1

The first demonstration deck was fabricated from No. 2 or better 2x4 southern pine lumber. No modifications were made to the bridge's super- or substructure prior to deck replacement. Installation began by removing about 1.8 m (6 ft) of bridge planking from one end of the bridge using a back hoe. The first 1.2 x 6.1 m (4 x 20 ft) deck panel was placed in position using a forklift and "locked" into place using a total of eight offset steel clips [Figure (2b)]. Three offset clips were placed on the outer two girders and two clips were used on the girder closest to panel midlength. Offset clips were placed on alternate sides of girders to prevent horizontal movement.

Because of difficulties in gaining access underneath the panels to install offset clips and the time required for clip installation, it was decided to discontinue offset clip installations on subsequent panels and use standard bridge clips along each edge of the panel [Figure 2(a)]. To clip the edge of the panel being installed (panel X) adjacent to an installed panel (panel Y), enough planking was removed to place panel X approximately 46 cm (18 in.) away from panel Y. Panel X was squared and aligned with respect to panel Y. Six to eight bridge clips were then installed along panel X's edge closest to panel Y using 60d ring-shank nails. After bridge-clip installation, panel X was then pushed toward and against panel Y using the bucket of the back hoe. Panel X's exposed edge (toward the old planking) was then secured using six to eight bridge clips. Clips were placed on alternate sides of girders to prevent horizontal movement. Clips from panel X were located to avoid contacting clips from panel Y. This process continued for the remaining panels. The last three panels at the end of the bridge were also secured from the bottom using offset clips to minimize panel movement resulting from vehicles coming onto the deck.

Approximately half of the panels were installed the first day. The decision was made to reopen the bridge that evening by removing just enough planking to place an unsecured panel into the 1.2-m (4 ft) wide

Figure 2 - Bridge with deck-connection clips: (a) 2.375 in. x 4.0 in. by 11 gauge steel clip attached to deck using 60d ring-shank nails; (b) 4.0 x 4.0 x 0.25 steel clip attached to deck using 0.375-in. diameter lag bolts (1.0 in. = 2.54 cm)

opening. Deck installation was completed by mid-afternoon of the second day.

Several minor problems were encountered during installation. First, the panel surfaces were unexpectedly slick from the creosote treatment. To correct this situation, a thin layer of sand was spread onto the panel surface. Second, several panels partially split in the glue joints during handling and placement. The fabricator was made aware of these defects. It was determined that these splits would not affect the strength, stability or service of the bridge. (An inspection 19 months after installation provided verification.) Third, because the bridge had an above-deck truss, minor difficulties were experienced in maneuvering the panels between the trusses and bracing. This was corrected on subsequent bridges by specifying panel lengths approximately 2 in. (5.08 cm) shorter than the full bridge width.

Bridge Deck 2

Installation of this bridge deck was similar to the system used on deck 1. No modifications were made to the bridge's super- or substructure prior to deck replacement. The 1.2 x 4.9 m (4 x 16 ft) panels were made of No. 2 or better 2x4 red oak lumber. All panels were secured along the edges using standard bridge clips. In addition, the first three panels at each end of the bridge were secured using offset clips to further reduce movement as vehicles enter and exit the bridge. The wood's hardness made fastening the bridge clips especially difficult. Workers were not able to drive 60d ringshank and common nails into the red oak lumber. As a result, 60d common nails were cut to

approximately 7.6 cm (3 in.) long and used to fasten the bridge clips.

There are several other concerns with this demonstration deck. Because of wood quality (straightness) and fabrication process, some individual 2x4 laminated members were offset as much as 1.9 cm (0.75 in.) compared to adjacent members. In addition, some panels did not lay flat, i.e. cupping across the width. Another concern was splitting at several glue joints. The fabricator indicated problems in getting the proper glue formulation and heat-treatment process. This was evident in the number of glue-joint splits during installation. The cupping problem also contributed to the splitting. Still, the strength, stability and service of the bridge was not compromised. (An inspection 13 months later confirmed this observation.)

Bridge Decks 3-7

Five additional bridge decks were installed: two consisting of No. 2 or better 2x4 southern pine, one consisting of No. 2 or better 2x6 southern pine, one consisting of No. 1 or better 2x6 southern pine, and one consisting of No. 1 or better 2x6 cottonwood. While cottonwood proved to be a satisfactory lumber for decking, only one deck was installed because of limited lumber availability and cost. The longest demonstration deck installed was approximately 37.2 m (122 ft) in length with the complete replacement taking less than five working hours.

The only significant problem encountered during the installation of these five decks involved the first demonstration bridge deck using 2x6 lumber. It became apparent during installation that many of the deck panels were cupped across the panel width. This resulted in several panels giving an audible “cracking” sound when they were initially loaded. No visible splits in the panels were observed. However, in a few days it was apparent that some panels were not secured to the beams, mainly because of the excessive cupping of these panels. Because of the additional stiffness of the 2x6 lumber, offset clips did not bring the panels into contact with the beams as they did with panels on other bridges made from 2x4 lumber. Upon inspection, it was determined that the existing panels could be clamped to the flanges of the supporting steel beam. This procedure removed the noise and visual movement that was evident just after installation. The deck was then coated with an asphalt layer approximately 7.6 cm (3 in.) thick. After 11 months in service, there is no indication of significant panel movement.

In another installation, the only difficulty not experienced previously was the height difference in newly-installed girders. After installation was complete, wooden shims were placed in the gaps between steel girders and the wooden deck to minimize panel movement and to ensure uniform load bearing.

During installation of the bridge decks, county commissioners identified a number of advantages of timber panels when compared to other decking materials. These advantages include that (1) there is a significant reduction in time the bridge is out of service; (2) there are lower dead loads with timber deck, possibly allowing a higher load rating; (3) the weather is not a factor during installation; (4) inexperienced county road workers can efficiently handle timber panels and can easily learn installation techniques; and (5) wood is more resistant to road chemicals.

Summary And Recommendations

Test panels were fabricated using southern pine, red oak, and cottonwood. Static-bending and transverse-bending tests were performed on test panels at the OSU Structural Engineering Laboratory. Results show that both pine and cottonwood prove satisfactory but red oak panels regularly fail at the face-glued joint, thus producing erratic results. Bending strengths and stiffnesses based on OSU laboratory tests indicate that allowable spans for 2x4 and 2x6 panels are sufficient to allow the use of southern pine or cottonwood on county bridges.

Seven replacement bridge decks were successfully installed using southern pine, cottonwood, and red oak lumber. Southern pine is the preferred species at this time because of the availability of consistent lumber quality and ease of acquiring kiln-dried materials. Cottonwood is acceptable for decking, but is more difficult to obtain than southern pine. Red oak is currently unacceptable for decking due to questions regarding availability of raw material, panel fabrication, and the difficulty in fastening panels to the superstructure.

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Integrated Remedial Protection of Wood in Bridges

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Abstract

While timber bridges can perform well under a variety of conditions, many bridges experience premature internal decay due to poor specification, inadequate preservative treatment or poor construction practices. Arresting deterioration in these bridges poses a major challenge since the wood under attack is normally deep beneath the surface treatments and is highly resistant to impregnation by most conventional liquids. In this report, we discuss the use of fumigants and water diffusible fungicides for arresting these attack and preventing renewed invasion. The benefits of the two chemistries are discussed in relation to the potential for attacks and speed of control required.

Keywords: fumigants, timber bridges, remedial preservation treatment

Introduction

Properly performed preservative treatment of wood produces an excellent barrier against attack by most agents of biological deterioration, however, this barrier is often disrupted during fabrication or as the wood seasons and checks. Nowhere is this problem more acute than in timber bridges. These structures are subjected to extensive design considerations, but often require extensive field fabrication during installation which exposes untreated wood to potential biological attack. In addition, many fasteners are driven through the treated zone into the untreated wood, again exposing the zone beyond the treated shell to entry by moisture and fungal spores. Finally, the larger timbers employed in bridges are generally not completely seasoned to their in-service moisture contents prior to treatment. These timbers can check extensively as they season in service, again exposing untreated wood to fungal and insect attack. The rate of decay in large timbers exposed above ground varies with species and the climate to which the bridge is exposed, but the ultimate result is the development of internal decay which reduces bridge service life (Scheffer, 1971).

These problems have led to a general perception that timber bridges have shorter service lives and require more maintenance than comparable bridges constructed with other materials (Smith et al., 1995; Smith and Bush, 1995).

A variety of methods have been developed to improve the depth of initial treatment to reduce the potential for internal decay (Graham, 1983). These practices include incising, through boring, radial drilling and

kerfing, but not all of these activities are compatible with timber used in bridges. In addition, studies have shown that even wood treated using these methods experiences low levels of internal deterioration. As a result, there is a substantial need for field treatments which can be applied to timber in bridges to arrest deterioration and prevent renewed attack (ASHTO, 1983; Ritter, 1990).

Deterioration in large wood members has long posed a major challenge to those charged with prolonging the useful life of a bridge (Ritter and Morrell, 1990). Most oil-based treatments lack the ability to migrate through wood for substantial distances. As a result, they cannot reach the points where decay fungi are actively growing. For many years, the treatment options for deteriorating timbers were limited, but the development of fumigants for wood application in the late 1960's provided a new, highly effective retreatment option (Graham, 1973, 1979). Fumigants are capable of moving as gases through the heartwood of nearly all wood species (Ruddick, 1984; Morrell et al., 1992a).

First developed for use on utility poles, fumigants are applied as liquids to steep angled holes drilled into poles and volatilize to move as gases through the wood. Three chemicals were initially explored for this purpose. Chloropicrin (trichloronitromethane) is a tear gas which has strong lachrymatory properties, Vorlex (20% methylisothiocyanate in chlorinated C_3 hydrocarbons) is a potent nematocide, and metham sodium (32.7 % sodium n-methyl-dithiocarbamate) has a long history of use for treating agricultural fields. Field trials with these chemicals showed that fungi were virtually eliminated from wood poles within one year after treatment (Figure 1). While these results were similar to experiences in soil application, it was the surprising ability of these chemicals to remain in wood for long periods after treatment that made them especially attractive for remedial protection. Fumigants are typically not detectable within 14 days after soil fumigation, yet these same chemicals were detectable in wood at levels which remained inhibitory to fungi for up to 20 years after treatment. Chloropicrin remains detectable at high levels in a number of species for many years after treatment (Morrell and Scheffer, 1985; Schneider et al., 1995). As result of these tests, fumigant usage in wood has steadily risen as utilities seek to extend the useful life of their wood structures (Morrell, 1989). Of the original three fumigants employed for this purpose, chloropicrin and metham sodium continue to be used. Vorlex, which was nearly as effective as chloropicrin, was difficult to apply and was never widely used for this purpose. In addition, a third fumigant, solid methylisothiocyanate (MITC) encapsulated in aluminum for safer application is registered for wood use. While fumigants are widely used by electric utilities, their use in timber bridges is less uniform, despite their potential for substantially extending wood service life. In this report, we will review the properties of the currently registered fumigants, outline the methods for application to timbers, describe newer formulations which are under development and finally, discuss several alternative chemicals which are available for remedial treatment of timber bridges.

Fumigant Application

Fumigants are normally applied to the wood through steeply sloping holes drilled across the grain (Graham and Helsing, 1979). These holes are then plugged with tight fitting wooden dowels which reduce the risk that the vaporizing fumigant will be lost to the outside environment. The goal of the steep sloping hole is to maximize the amount of chemical which can be applied while minimizing the number of strength-reducing holes which must be drilled. In round timbers, the drilling pattern derives from the pattern of inspection holes used to detect internal decay (Graham and Helsing, 1979). In timbers, the chemicals are normally applied through perpendicular holes drilled into the upper face on either side of any checks which might be present. In other areas of a bridge, fumigant application can become more problematic since care must be taken to avoid connectors and since it is sometimes difficult to drill vertically into a timber. At least one fumigant is available in a solid encapsulated formulation which permits application to timbers through holes drilled at almost any angle.

Properties of Existing Fumigants

Chloropicrin and metham sodium are both liquid fumigants. Chloropicrin is highly volatile and its handling properties have generally limited its usage to areas away from inhabited buildings. Applicators must wear full face respirators during application of this chemical, creating considerable public image problems in some areas of the country. There have been a number of attempts to gel or otherwise encapsulate chloropicrin, but none have been commercially successful (Goodell, 1989). One formulation of chloropicrin is available in semi-permeable tubes which slow the release rate for a short period prior to application (Fahlstrom, 1982). These plastic tubes have a permeable membrane on the top which degrades over a several day period, releasing chemical into the

Figure l-Effect of fumigant application on survival of decay fungi in Douglas-fir poles.

wood. Tubes are normally filled at the beginning of a work day. This formulation has found its primary application for remedial treatment of bridges, where the large numbers of contiguous timbers being treated makes the process economical. The system also has some benefits because it permits application farther above the ground than would be possible with liquid chloropicrin. Liquid chloropicrin can leak from checks or other wood defects during application, posing a hazard to workers, while the tubes limit this risk. Despite its drawbacks, chloropicrin remains the most effective of the currently registered fumigants.

Metham sodium is the most widely used fumigant for remedial wood treatment. This compound is not, as applied, a very effective fungicide. Instead, metham sodium decomposes in the presence of organic compounds (such as wood) to produce a variety of fungitoxic compounds including MITC, which was the primary fungicide present in Vorlex (Morrell, 1994). Metham sodium smells like rotten eggs and is caustic, but it is the least toxic of the currently registered wood fumigants. It is has also proven to be the least effective of these chemicals (Figure 1). While chloropicrin has provided up to 20 years of protection, metham sodium eliminates decay fungi within one year, but provides only seven to 10 years of protection in Douglas-fir timbers (Helsing et al., 1984). Part of this differential performance reflects the lower amount of active ingredient applied. Chloropicrin is 96-97% pure, while metham sodium is a 32.7 % solution of the sodium salt. Thus, for a given amount of treatment hole, metham sodium provides much less protective chemical. In addition, studies suggest that the rate of decomposition of metham sodium to MITC is very poor and is sensitive to wood species, moisture content, and temperature (Morrell, 1994). As a result, only about 12% of the total liquid metham sodium applied actually becomes fungicidal. One final drawback of metham sodium is its high toxicity to aquatic life. As a result, metham sodium is not recommended for use in wood near standing water.

A field test of metham sodium in a Douglas-fir timber bridge located near Salem, Oregon shows that the MITC was present at fungitoxic levels at significant distances from the point of application 3 years after treatment (Table 1, 2). These results were similar to those found for Douglas-fir poles treated with equivalent dosages and suggest that fumigant treatment of bridge timbers should provide comparable protection against fungal invasion. Eventually, chemical loss might be expected to increase from bridge timbers; however, since these members have a higher surface to volume ratio. Fumigant is rapidly lost from the wood surface, so increasing the surface area should diminish the protective period provided by a given volatile chemical (Zahora and Morrell, 1989).

The risks of handling volatile, caustic liquids during remedial treatments encouraged the development of MITC as a wood fumigant. MITC is a solid at room temperature and sublimes directly to a gas, but it is also very caustic and must be encapsulated for safe handling (Zahora and Corden, 1986). Field tests in utility poles have shown that MITC is more effective than metham sodium but less effective than chloropicrin in terms of the length of the protective period (Figure 2)(Morrell et al., 1992c). In addition, MITC, as currently packaged, is more costly than either of the other materials, although the encapsulation does improve safety and permits application to wood well above the ground.

In addition to the registered formulations, efforts are underway to develop other, safer fumigants. The simplest strategy is to encapsulate an existing liquid fumigant to reduce the risk of spills and worker exposure. This strategy has recently been employed by encapsulating chloropicrin in various polymers to slow the release rate and reduce the risk of worker exposure during application. Preliminary field trials suggest that release may occur over a six to 10 year period (Love et al., 1996). When coupled with the tendency of wood to retain chloropicrin, this release rate creates the potential for longer protective periods than those afforded by current treatment technologies. Considerable effort remains to demonstrate the validity of these assumptions. This formulation is currently undergoing registration with the U.S. Environmental Protection Agency.

The other alternative to the currently registered fumigants is to identify solid chemicals which decompose to produce volatile fungicides in the presence of wood. There are a number of compounds which could potentially be used for this purpose, but the most likely candidate is Basamid (Forsyth and Morrell, 1995). Basamid is a crystalline material whose cyclic structure decomposes to produce MITC. Field trials have shown that this material decomposes too slowly to be of use as a remedial treatment (Highley and Eslyn, 1989), but the rate of decomposition can be accelerated by addition of buffers or metals. Field trials, again in utility poles, suggest that incorporating copper into Basamid prior to application produces decomposition at levels which would control fungi already present and limit the risk

Table 1- **Residual MITC content in Douglas-fir bridge stringers one or two years after metham sodium treatment as determined by gas chromatographic analysis of ethyl acetate extracts of wood samples.**

		µg MITC/OD g wood					
		Inner			Outer		
Structure #	Stringer Position	1 year	2 years	3 years	1 year	2 years	3 years
5	Top	4.3	52.3	9.7	0.00	27.6	3.3
	Bottom	59.7	34.7	31.1	24.5	112.4	84.1
10	Top	40.2	136.1	71.3	53.2	60.3	76.4
	Bottom	75.8	114.9	43.0	39.9	59.4	116.3
15	Top	27.3	66.1	46.4	37.4	59.5	145.4
	Bottom	16.0	99.7	17.8	24.3	112.9	43.4
20	Top	26.2	115.5	58.2	65.4	130.6	44.6
	Bottom	82.7	42.6	67.7	23.2	19.9	163.1
25	Top	26.5	80.2	40.7	13.1	44.4	52.5
	Bottom	33.4	83.3	86.0	65.5	95.4	32.1
30	Top	73.2	126.8	77.5	100.3	98.5	70.2
	Bottom	83.6	40.8	83.3	75.8	63.7	49.3
35	Top	44.1	74.1	108.7	60.6	120.8	56.5
	Bottom	14.0	75.1	19.2	9.2	42.4	8.8
40	Top		50.1			140.4	
	Bottom		92.1			56.7	
Average	Top	34.5	87.7	58.9	47.1	85.3	64.1
	Bottom	52.3	72.9	49.7	37.5	70.4	71.0

Table 2- **Levels of colonization by Douglas-fir timbers one to three years after application of metham sodium as measured by culturing increment cores.**

		Cores With Decay Fungi (%)[a]		
Structure #	**Stringer Position**	**1 year**	**2 years**	**3 years**
5	Top	0	0	0
	Bottom	0	0	0
10	Top	0	0	0
	Bottom	0	0	0
15	Top	17	0	0
	Bottom	0	0	0
20	Top	0	0	0
	Bottom	0	0	0
25	Top	17	0	0
	Bottom	0	0	0
30	Top	0	0	0
	Bottom	0	0	0
35	Top	17	0	0
	Bottom	0	0	0
Average	Top	7.3	0	0
	Bottom	0	0	0

[a]Values represent means of 6 cores/treatment

Figure 2-Residual levels of MITC in Douglas-fir and southern pine poles one to seven years after application of MITC-Fume.

of reinvasion (Forsyth and Morrell, 1993). One other advantage of this chemical is its existing registration for application to non-food crops, making it far easier to register for wood application.

Ultimately, strategies utilizing solid fumigants which can decompose slowly over a several year period can provide a safer method for preventing internal decay using volatile chemicals.

Alternative to Fumigants

While fumigants have proven to be highly effective, their handling properties have encouraged a search for less toxic decay control strategies. One alternative to fumigants are water diffusible fungicides including boron and fluoride. These compounds do not volatilize like fumigants, but they are able to diffuse from areas of high to low concentrations whenever free water is present in the wood. Both boron and fluoride have been used for many years for protecting a variety of products from decay, but their use for internal decay control in large timbers in North America is a relatively recent development (Becker, 1976). Boron is highly effective against most decay fungi and insects, although the levels required for control can vary quite widely. Typically, a target boron retention between 0.25 and 0.5% by weight is required for wood protection. Levels required for preventing wood attack where the Formosan termite is present are many times higher. Fluoride is generally only used for controlling decay fungi. In a number of studies, boron and fluoride have moved well through moist wood, but move very little when the moisture content falls below 30% (Smith and Williams, 1969). Proponents of these systems have pointed out that substantial fungal decay does not occur when the moisture content falls below 30%, therefore, it should not matter if the diffusible compound does not move in dry wood since no decay can occur under these conditions. However, this approach ignores the fact that wood moisture contents can vary widely along the length of large timbers. As a result, the boron or fluoride may be applied to a dry zone, where no movement will occur, while an adjacent wet area contained actively growing decay fungi. Judicious application can help overcome some of this limitation, but there remains the risk that the improperly placed chemical will not diffuse to the points where it is needed.

Two formulations of boron and fluoride are labeled for wood use in the U.S. Fused boron rods are produced by heating boron to high temperatures and pouring this molten material into molds. The boron cools and hardens into a glass-like rod which is applied to the same steep angled holes used for fumigant treatment. Boron diffuses from the rods in the presence of moisture (Morrell et al., 1990) and moves well through a variety of North American wood species (Morrell et al., 1992b). Sodium fluoride is available in rod form and has a long history of use in railroad ties, but has only recently been labeled for other wood uses. Field trials are currently underway to evaluate the performance of these materials in larger timbers. An additional formulation which is not currently labeled in the U.S. is composed of both fluoride and boron in a rod form (Preschem Ltd., Cheltenham, Australia). Field trials with this formulation suggest that the rate of chemical movement from the rods remains slower than that found with fumigants (Table 3).

Field trials of boron in fused boron rods have shown that boron diffusion away from the application point in Douglas-fir poles takes up to 3 years to achieve chemical levels which can provide effective fungal control (Table 4)(Morrell and Schneider, 1995). Since decay continues while this diffusion occurs, the user takes a risk that the timber will deteriorate to an unsafe condition before boron levels are sufficient to effect fungal control. Trials with southern pine poles have proven more successful, perhaps reflecting the more permeable nature of this wood species (Zahora et al., 1996). Trials with a fluoride/boron rod have shown that boron has moved more rapidly than the fluoride over a 2 year period. These results are interesting since a prior trial of groundline preservative pastes containing fluoride and boron showed the opposite effect in Douglas-fir posts (Morrell et al., 1994).

A final diffusible preservative system available for timber in bridges is a water soluble copper naphthenate/boron paste. Limited field trials with this formulation indicate that the boron moves well from the point of application, while the copper naphthenate moves to only a limited extent (Forsyth and Morrell, 1992). As a result, this treatment might be useful for treating the inner surface of large voids, where the copper naphthenate would coat the surface of the void, while the boron would diffuse further into the wood. This treatment, however, would be unlikely to completely eliminate established decay fungi.

Selecting Remedial Treatments

Bridge maintenance specialists have a variety of options for arresting internal decay in their bridges. Each chemical has certain pros and cons which may make it especially attractive for specific applications. For example, where decay is actively occurring in a bridge located away from inhabited structures, chloropicrin

Table 3- **Residual boron and fluoride at selected locations above or below the groundline in Douglas-fir poles one year after treatment with fluoride/boron rods.**

		Residual Chemical (%F or BAE) [b]											
		Distance from Treatment Zone											
		-300 mm				300 mm				600mm			
Dosage (g)	Application Pattern (Degrees)[a]	Outer		Inner		Outer		Inner		Outer		Inner	
		F	BAE	F	BAR	F	BAE	F	BAE	F	BAE	F	BAE
70.5	90	0.02	0.10	0.11	0.63	0.08	0.54	0.11	0.51	<0.01	0.03	0.01	0.02
	120	0.01	0.06	0.03	0.26	0.02	0.09	0.06	0.49	<0.01	0.04	<0.01	0.05
141.0	90	0.01	0.28	0.07	0.06	0.02	0.07	0.07	0.36	0.01	0.03	0.01	0.05
	120	0.04	0.09	0.12	0.67	0.03	0.10	0.04	0.20	0.01	0.04	0.01	0.05
0.00	-		0.01	-	0.08	-	0.04	-	0.01	-	0.01	-	0.01

[a] Values represent composite analyses of 5 poles/treatment. BAE represents boric acid equivalent.
[b] Application patterns were holes at 90 or 120" intervals around the pole.

Table 4 -- **Residual boric acid equivalent (BAE) at selected locations in Douglas-fir poles 1 or 3 years after treatment with borate rods with and without supplemental moisture.**

		Residual Boron Concentration (%BAE) by position[a]											
		Groundline				300 mm above Groundline				900 mm above Groundline			
Borate Dosage (%)	Water Added	Outer		Inner		Outer		Inner		Outer		Inner	
		Year 1	Year 3	Year 1	Year 3	Year 1	Year 3	Year 1	Year 3	Year 1	Year 3	Year 1	Year 3
120	-	0.02	0.17	0.02	0.34	ND[b]	0.20	ND	0.32	ND	0.02	ND	0.02
	+	0.02	0.49	0.02	0.72	ND	0.11	ND	0.16	ND	0.03	ND	0.04
240	-	0.02	0.45	0.02	0.75	ND	0.13	0.02	0.10	ND	0.05	ND	0.04
	+	0.01	0.38	0.02	0.54	ND	0.14	ND	0.22	ND	0.03	ND	0.04

[a] Values represent composite analyses of 5 pole sections.
[b] ND signifies boron levels <0.0l % BAE.

might represent the best option, while a similar bridge near houses might be better suited to treatment with metham sodium or MITC. In instances where there is no visible evidence of decay, the use of water-diffusible boron or fluoride may be appropriate since the risk of deterioration while the chemical diffuses through the wood is minimal.

Timber bridge inspectors contemplating the use of diffusible boron or fluoride must carefully weigh the benefits of safer chemical application against the need for rapid decay control. In instances where the timbers contain active decay fungi, fumigants may provide the fastest control, thereby preventing further deterioration of the bridge capacity. In some instances, inspection may show that a bridge has only minor decay problems. In these cases, the preventative application of diffusible chemicals may prevent the inception of decay. One advantage of boron or fluoride is the unrestricted classification of these compounds. Fumigants are generally restricted use pesticides and, even where they are not, considerable care must be taken during application. The diffusibles are more easily handled and may be more suitable in locations where extensive training of the inspection crew in chemical handling is not desirable or cost effective.

The long term protective effect of diffusibles remain under study, so users of these technologies would be strongly advised to consider some form of monitoring of the chemical levels in their structures to determine when retreatment is necessary.

Conclusions

The wide array of treatment options provide a variety of opportunities for prolonging the useful life of timber in bridges. Along with the obvious safety and economic benefits, these treatments also conserve our valuable forest resources.

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Proof Loading Closed Timber Bridges

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Abstract
Using proof loads and safety factors developed according to recently accepted AASHTO procedures, it is demonstrated that a closed timber structure can safely carry 3 tons and thus be reopened. A literature survey was conducted to provide information that was synthesized into a strategy for proof loading closed timber bridges. The resulting strategy applies the concept of structural shakedown to the rating of timber bridges. Load tests and material tests were conducted.

Key Words: Proof loading, load rating, shakedown, off-system bridges, closed bridges, residual deformation, timber bridges

Introduction
The Louisiana Department of Transportation and Development (DOTD) is responsible for the inspection and load rating of thousands of on-system bridges (owned by the state) and, as a special service, several thousand off-system (locally owned) timber bridges. There are approximately 150 off-system timber bridges in Louisiana that are currently closed.

The load-carrying capacity of bridges is usually very conservatively based upon an inspector's qualitative rating as well as quantitative results from a computer analysis of the structure. When a bridge is judged unable to safely carry 3 ton truck loads, it must be closed. Closed off-system bridges are likely to remain a hardship for local travelers for a long time, since local governments which own the bridges often lack the money needed to repair or replace such structures. The intent of proof loading is to augment the bridge authority's information concerning the closure of a structure; proof loading is not intended to replace the existing procedures for bridge inspection and rating.

The Louisiana Transportation Research Center of the DOTD conducted this proof loading research as State Project 736-99-0311. The primary objective of this research effort is to develop a strategy for proof loading closed off-system timber bridges typical to Louisiana. A static load test, laboratory material tests, and a proof load test were all conducted as part of the research effort.

Synthesis of Relevant Literature
A review of the available literature related to proof loading and rating timber bridges was conducted, including documents which outline federal and state policies concerning the load rating of bridges. While there is a considerable body of literature dealing with proof loading large steel and concrete bridges, the literature that deals with load rating and/or proof loading deteriorated timber highway structures is

limited.

Recent bridge research has focused on the reasonability of using proof loading as a means of rating bridges, and the latest version of the AASHTO bridge rating specifications (1) encourages the use of proof loading. The goal of this study was to develop a rational strategy to proof load a closed off-system timber bridge to determine if it could safely be reopened until repair or replacement could take place. Therefore, it was decided that since the structure was closed, it should not be fully reopened to the community. Instead, the proof loading will be directed at determining if the bridge is adequate to carry a single lane of traffic.

The specifications contained in the AASHTO and DOTD bridge rating manuals (1,2) state that an AASHTO H3 vehicle with a front axle of 0.6 tons and a rear axle of 2.4 tons should not create load effects that exceed operating stress levels in the bridge members. Every attempt is made to ensure that the general policies for inspecting and rating bridges outlined in these documents are followed when conducting proof loading.

When proof loading a structure, stresses are not necessarily measured; instead, it is generally quicker and easier to measure the load-induced deflections of the structure. In this case, the drafted proof loading manual developed by Lichtenstein (3) specifies that a safety factor of 1.6 for one lane of traffic applies for operating level rating. Thus for an AASHTO H3 proof load with its 4.8 kip rear axle, the target test load for short spans can be modeled by a 7.5 kip single axle trailer.

TxDOT conducted a thorough proof loading pilot study (4) wherein a car, a pickup, and then a water tank truck were used to load the structure while deflections were measured with the loads on and off the spans. Pre-established limits were placed both on the recoverable (elastic) deflections and the non-recoverable (residual) deflections However, no particular justification was found in the literature for the limits set by the TxDOT researchers. The target deflections were found to be too constrictive and difficult to measure.

The researchers of NCHRP Project 12-28(12) recommended (5) that structural shakedown should be adopted as the appropriate bridge rating limit state for steel bridges. Structural shakedown occurs when residual deflections from each successive passage of the factored target rating load are observed to diminish until no residual deflections are set up with the load's passage. After shakedown occurs and some accumulated residual deflection is created, the rating load will be resisted elastically with an assured level of safety provided by the load factor.

For the proposed proof loading strategy of this project, the recommendations of NCHRP 12-28 (12) will be accepted and extended to the proof loading of timber structures.

Static Load Test

In order to establish orders of magnitude for the proof load effects on typical off-system timber structures, a timber bridge in the process of being replaced was subjected to a static load test. The state-owned structure was a fifteen span timber bridge carrying LA 424 over the Pushapatapa Creek near Franklinton, Louisiana.

The 35 year old bridge was structurally deficient with a load posting of 20 ton single and 35 ton combination vehicles. The structure was also functionally deficient: it was straight with a narrow 19 foot roadway width occurring on a curved highway section. The bridge was being replaced with a 32 foot wide concrete structure.

The structure was constructed from No. 1 Southern Pine. Each of its 19 foot simple spans consisted of eleven 6 inch by 14 inch stringers spaced 2 feet apart. As a result of a statewide timber bridge rehabilitation project that was carried out 15 years ago, seven 6 inch by 12 inch stringers were inserted between the original stringers. The bents consisted of 12 inch square cap beams supported by four 12 inch diameter short piles spaced 6.5 feet apart. To better model typical off-system bridges, the existing 12 inches of overlay ballast and asphalt were removed from the 3 inches by 10 inches plank deck. The test load was 200 kips loaded onto two spans using four stacks of five 4 foot by 20 foot precast concrete panels weighing 10 kips each

Pairs of stacks simulated midspan point loads in each lane. A pair of panel stacks was first created to simulate one lane loaded, then the second pair of stacks was created. The timber pads supporting the panels were sized to simulate the footprint of a single axle load on one span and a tandem axle load to the other.

The elevation shots at the midspan locations of both spans were very similar. The maximum deflection was 1 inch under the 100 kip midspan loads. This

deformation was almost completely recovered upon removal of the load. Residual deflections were no more than 1/8 inch.

Prior to testing, a structural analysis of the simple span structure was carried out. It was assumed that the modulus of elasticity was 1,500,000 psi and that 50% of the deck was effectively acting as a composite flange with the stringers. On this basis, a center point load of 100 kips was calculated to create an elastic deflection of 1 inch.

Even though the test results matched the predicted behavior based on the structure modeled by assuming 50% effective composite deck action, this assumption had to be further proven correct -- it was necessary to ascertain that the modulus of elasticity was actually 1,500,000 psi as assumed.

Material samples were tested at the DOTD materials Laboratory. The lab personnel performed flexure tests on 11 clearwood beam samples. The average modulus of elasticity with the 1/0.94 span-depth ratio adjustment was determined to be 1,500,000 psi, as predicted for No. 1 Southern Pine.

The question of the order of magnitude of elastic and inelastic deformation was the primary interest of this test. For this structure, the midspan elastic deformation associated with 10 kips of load (a pair of panels) was about 1/8 inch. Under the full test load of 100 tons for this bridge which was posted for 20 tons, the residual deflections were less than 1/8 inch.

Proposed DOTD Proof Loading Procedure

The proposed DOTD proof load strategy requires a minimum of labor and equipment. The essentials include a minimum of two men, a surveying rod and level, and a light pickup truck towing a loaded single axle trailer with an axle weight of 7.5 kips. For the greatest safety to the pickup driver, the trailer should be long enough so that there is about 20 feet between the single axle and the rear axle of the pickup truck. For added safety, the tongue weight of the trailer should be kept low and the hitch loosened during the test.

The proposed DOTD proof load testing proceeds as follows:

1) Locate the midpoint of each span of the structure. At each of these midspan locations, mark three elevation points -- one at each of the two roadway edges and at one at the centerline of the bridge roadway. Note that for a bridge with N spans, there will be 3 X N marked elevation shot points.

2) Set up the level instrument somewhere off of the bridge structure and take the initial elevation shots to each of the marked points.

3) Drive the load across the structure along the roadway centerline. Stop the trailer axle over each midspan location for about two minutes. If any unusual visual or audible distressing is evident, the test should be terminated: it is unlikely that a bridge with visible deflections or audible stressing is in a condition suitable for opening. In any case, the pickup driver should be prepared to take quick action in the unlikely event that the loaded span behind the truck should completely collapse.

4) After the load has crossed and exited the bridge, take new elevation shots of each of the marked points. For each load crossing, compute the residual deflection values. Residual deflections for each load crossing are computed as the difference between each of the before and after elevation shots. If any of the residual deflections is 1/4 inch or more, the test should be terminated.

5) Repeat steps 3 and 4 several times. A load crossing may be accomplished by reversing the load back across the bridge.

Pass: If all of the computed residual deflections for a load crossing are zero, the structure is responding elastically, and it has reached a shakedown limit. This should be confirmed with another load crossing and another set of zero residual deflections.

Fail: If any of the computed residual deflections is greater than its previous value, the structure is exceeding the shakedown limit state. Additionally, based on field test data, if the total residual deflection should be 1/4 inch or more, the test should be terminated.

Proof Load Test

The proposed DOTD proof loading strategy was demonstrated on an off-system timber bridge near Springfield, Louisiana. The five span timber structure represents a standard Louisiana timber highway bridge and is similar in description to the structure of the static load test. The bridge is still open to traffic, though it is posted for the minimum 3 ton limit with an overall condition rating of 3.

A pickup was used to tow a fifth wheel trailer with a tandem axle weight of 11 kips. Since the tandem

axle trailer creates a slightly higher moment than the recommended 7.5 kip single axle trailer, the substitution is acceptable. For each of several load crossings, the computed residual deflections were zero. Though it was not required, midspan elevation shots were taken with the load on the spans; the maximum observed elastic deflection was only 1/8 inch.

The procedure proved to be quick and easy to follow. The rod and level deflection readings are accurate to 1/8 inch.

Conclusion

The impetus for this research was that there may be instances where a closed bridge can safely be reopened -- if only to one lane of traffic -- and thereby reduce inconvenience to local automobile traffic until such time as the deficiencies of the structure can be remedied. While not intended as a solution to the problems posed by lack of bridge replacement funding, proof loading closed bridges offers the potential for local residents to temporarily continue to use a bridge.

Proof loading is likewise not intended to replace long established AASHTO procedures for inspecting and rating bridges. Proof loading is merely intended to supply additional information when making decisions about appropriate posting levels.

Several conclusions can be drawn:

- Load capacity ratings for timber highway bridge structures appear to be very conservative.
- It is reasonable to assume that 50% of the timber deck acts compositely with the stringers.
- Proof loads in the 10 kip range create elastic deformations of no more than 1/8 inch and generally little or no residual deformation.
- If residual deformations of 1/4 inch are observed, the bridge should not be reopened.
- A simple procedure to proof load off-system closed timber bridges based on a shakedown limit state has been developed and successfully demonstrated.

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Load Rating of Single Span, Glulam Beam Bridges using Two Computer Applications

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Abstract

The National Bridge Inspection Standards requires bridges be inspected biannually and when conditions warrant that a load rating must be conducted. There are more than 575 thousand bridges in existence today. As these structures age, load rating should be conducted when a significant change in condition occurs. Load ratings are time consuming and costly. With ever decreasing maintenance budgets, an economical and versatile load rating worksheet or a computer program would be of benefit to many local, state and federal agencies. With the broad availability of computers, this process can be accomplished quickly and accurately with computer spreadsheets or programs. This paper will discuss two different computer programs developed to load a rate single span, one lane, glulam beam bridges for a standard AASHTO HS20 vehicle.

The two methods developed, used a Mathcad® 6+ worksheet and a QuickBASIC program to calculate load ratings for a Glulam Beam bridge. Mathcad® 6+ is a powerful mathematical tool that allows equations to be written in both real math notation and in a worksheet. Two additional functions were developed for calculating moments and shears using a Dynamic Linked Library (DLL) in Mathcad® 6+. QuickBASIC is a computer language that is widely used and is easy to learn.

The single span, one lane bridge type was used because it is standard type structures for the U.S.D.A. Forest Service and many local municipalities. Alterations may be made with ease for load rating other bridge types and multiple lane structures. Load rating bridges using a Mathcad® 6+ worksheet and a QuickBASIC program turned out to be a very economical and viable solution for this bridge type.

Load Rating

The load rating procedure used in this paper will follow the method detailed in the Manual for Condition Evaluation of Bridges, 1994 and the standard design specifications from AASHTO Standard Specifications for Highway Bridges, 15th ed. Load rating involves structural inspection and calculations based, in part, on inspections of the structure. Structure inspection should comply with the requirements of the National Bridge Inspection Standards with respect to frequency of inspections, type of inspections and qualifications of inspection personnel. The types of inspections are initial, routine, damage, in-depth and special. The normal frequency for routine inspections is biannual with additional inspections being conducted when the structure has been damaged or when conditions suggest a more in-depth inspection be needed. Load ratings should be conducted for damage, in-depth and special inspections and for routine inspections, when changed

conditions warrant it. The Manual for Condition Evaluation of Bridges, 1994 states the qualifications for "the individual charged with the overall responsibility for load rating bridges should be a licensed professional engineer and preferably have a minimum of five years of bridge design and inspection experience." They should also have experience and knowledge with the type of bridge that is to be load rated.

Structures should be load rated for inventory and operating levels. Inventory rating is the stress level at which the bridge was designed and the live load that can safely use the structure for an indefinite period or indefinite repetitions. Operating rating is the stress level at the maximum permissible live load that the bridge may be subjected, but allowing unlimited vehicles at this stress level may shorten the bridge life. Use of the bridge at operating stress level requires the governing agency to issue special use permits for that vehicle.

Rating Procedure

This paper follows the rating procedure outlined in Section Six (6) of the Manual for Condition Evaluation of Bridges. 1994 published by the American Association of State Highway and Transportation Officials. This procedure only addresses ratings of interior beams.

Nomenclature

The following nomenclature will be constant for the worksheet and program:

A- Area

A_v- Area for shear.

A_T- Total area of beam.

Bridge_Span - Span of bridge in feet. Measured from the centerline of bearing to the centerline of bearing.

C_M- Moisture content factor.

C_D- Duration of load factor.

C_f- Form factor.

C_V- Volume factor.

C_L- Lateral stability of beams factor.

DL_{Deck}- Dead load due to deck.

DL_{Beam}- Dead load due to Beam.

$DL_{RunningSurface}$- Dead load due to running surface.

DL_{Total} - Total dead load.

F'_b- Allowable bending stress.

F'_V- Allowable shear stress.

F_b- Tabulated bending stress.

Fv - Tabulated shear stress.

Inv_Rating - Bridge member rating in tons for inventory stress level.

I_X - Moment of Interia

M_{R_inv} - Moment Capacity of the member for inventory stress level.

M_{R_oper} - Moment Capacity of the member for operating stress level.

M_{DL} - Moment due to dead load.

M_{LL} Moment due to live load.

Num_of_Beams - Number of glulam beams.

Oper_Rating - Bridge member rating in tons for operating stress level.

R_{DL}- Reaction due to dead load.

RF- Rating factor for the live load carrying capacity with subscripts for a combinations of moment, shear, inventory and operating.

S_X - Section Modulus

$V_{R\text{-}inv}$- Shear Capacity of the member for inventory stress level.

V_{R_oper}- Shear Capacity of the member for operating stress level.

V_{DL}- Shear due to dead load.

V_{LL}- Shear due to live load.

Wt_{Wood} - Weight of wood, 50 lb/ft^3.

b - Width of beam.

$b_{Reduction}$ - Reduction in beam width.

$b_{effective}$-Effective beam width.

d - Depth of beam.

$d_{Reduction}$ - Reduction in beam depth.

$d_{effective}$ - Effective beam depth.

Allowable Stresses

Table 13.5.3A Structural Glued Laminated Softwood Timber of AASHTO Design Specifications gives the allowable stresses.

Dead Load

The dead load is the weight of the member and any other permanent weight carried by the member. The unit weights for materials in this calculation follow AASHTO 3.3 Dead Load. Use a weight of wood is 50 lb/ft^3.

Live Load

The live load is the weight of a moving vehicle that is traveling across the bridge. The typical vehicle used in these load rating worksheets is a standard AASHTO HS20 vehicle per Manual for Condition Evaluation of Bridges, 1994. Mathcad® 6+ functions were developed for calculating maximum moment and shear using mvldStdMaxM(span, type) to calculate the maximum moment and mvldStdMaxV(span, type) to calculate the maximum shear for any simple span length due to five vehicle types. Span is the clear length of bridge in feet. The QuickBasic program calculates the maximum moment and shear using a program subroutine. Any of the following five vehicles maybe used to load rate the bridges:

(1) Standard AASHTO HS20 Vehicle

(2) Standard Forest Service U54 logging truck
(3) Standard Forest Service U80 logging truck
(4) Standard Forest Service U102 logging truck
(5) Standard Forest Service L90 yarder

The programs compute the live load shear at a distance three times the beam depth or the quarter point, whichever is less. A shear envelope approach is used to approximate the shear at this point.

Distribution of Live Load

The distribution of live loads to individual stringers is determined by AASHTO 3.23 Distribution of Loads. The column labeled Bridge Designed for One Traffic Lane from Table 3.23.1 Distribution of Wheel Loads in Longitudinal Beams was used for calculation of live load distribution for Glulam Bridges. The distribution of live load shear follows AASHTO 13.6.5 and the given formula:

$$V_L = 0.5*[(0.6*V_{LU}) + V_{LD}]$$

where V_{LL} is the distributed live load vertical shear, V_{LU} is the maximum vertical shear from an undistributed wheel line, and V_{LD} is the maximum vertical shear from the vehicle wheel lines distributed laterally as specified for moment.

Impact Factor

Timber structures do not require an Impact Factor (I).

Rating Formula

The Rating Factor (RF) equation (6-1a) for Allowable Stress Design is

$$RF = \frac{\text{Member Capacity- Member Dead Load}}{\text{Member Live Load*(1+Impact Factor)}}$$

where the load effect being the effect of the applied loads on the member. The bridge rating equation (6- lb) is

$$\text{Bridge Member Rating} = RF * \text{Nominal Weight}$$

with the bridge member rating and nominal weight in tons. The lowest rated structural member controls the load rating of the bridge. This worksheet rates interior beams for shear and bending. Other elements of this bridge such as decking and substructure need rating.

Mathcad® 6+ and DLL'S

Mathcad® 6+ computer software allows placing of formulas, numbers, graphics and text into a worksheet, that is easy to follow and professional in appearance. The equations are shown in real math notation and the changing of variables update answers instantly. Modifying equations is simple and results are obtained on the spot.

Mathcad® 6+ supports Dynamically Linked Libraries (DLL's), development of customized functions, which simplify Mathcad® 6+ worksheets. These functions are included as a built-in function in Mathcad® 6+. One DLL developed for the load rating worksheets, contained two functions: mvldStdMaxM(span, type) and mvldStdManV(span, type). MvldStdMaxM(span, type) stands for the Maximum Moment due to Standard Moving Loads. The function calculates the maximum moment of a single span beam due to standard moving vehicles. MvldStdMaxV(span, type) stands for the Maximum Shear due to Standard Moving Loads. The function calculates the maximum shear of a single span beam due to standard moving vehicles. The span is the clear span of the bridge and type is either 1 - AASHTO HS20 truck, 2 - Forest Service U54 logging truck, 3 - Forest Service U80 logging truck, 4 - Forest Service U102 logging truck or 5 - L90 yarder.

The procedure for developing a DLL is relatively simple and requires Mathcad® 6+ and a 32-bit C++ compiler. The steps in creating a DLL are 1) writing the source code in C or C++, 2) compiling the source code with a 32-bit compiler, 3) linking the object files with the **mcaduser.lib** file to create a DLL, and 4) placing the DLL in the directory **C:\mcad\userefi.** Mvldstd.dll is the name of the DLL created. Mathcad® 6+ allows for error checking in the DLL and will return user friendly error messages. The errors checked for in the above DLL are: 1) span must be positive, 2) span must be real number and 3) type must be 1,2,3,4 or 5. See Figures l-4 for an actual Mathcad® 6+ worksheet.

QuickBasic Program

The basic program discussed in this paper was developed by Charles Milne, PE for the Stikine Area of the Tongass National Forest. The program was written in a computer language called basic and was developed for a specific bridge type. The program has tried to take into account most of the major attributes for load rating glulam bridges. A subroutine was written for calculating a maximum moment and shear for the five types of vehicles listed above. See Figure 5 for an actual QuickBasic program output.

GLULAM BEAM BRIDGE RATING

The following program rates single span, one lane Glulam bridges using the procedure described in the Manual For Condition Evaluation of Bridges published by the American Association of State Highway and Transporation Officials, 444 North Capital Street, N.W., Suite 249, Washington, D.C. 20001. This worksheet considers interior stringers only.

General Information

Bridge_Span := 48 in feet. Measure from center to center of bearing. kips := 1000·lb

Treated Timber Glulam Bridge Typical

$d := 50.875 \cdot in$ $b := 8.5 \cdot in$ $Num_of_Beams := 3$ $S_{Beam} := 5.68 \cdot ft$ $Wt_{Wood} := 50 \cdot \frac{lb}{ft^3}$

$d_{Reduction} := 0 \cdot in$ $b_{Reduction} := 0 \cdot in$ $t_{Plank} := 6.75 \cdot in$ $t_{Running_deck} := 4 \cdot in$

Section Properties

$b_{Effective} := b - b_{Reduction}$ $b_{Effective} = 8.5 \cdot in$ $d_{Effective} := d - d_{Reduction}$ $d_{Effective} = 50.87 \cdot in$

$I_x := \frac{b_{Effective} \cdot d_{Effective}^3}{12}$ $I_x = 93271.93 \cdot in^4$ $S_x := \frac{I_x}{\left(\frac{d_{Effective}}{2}\right)}$ $S_x = 3666.71 \cdot in^3$

$A_V := (b - b_{Reduction}) \cdot (d - d_{Reduction})$ $A_V = 432.44 \cdot in^2$

$A_T := b \cdot d$ $A_T = 432.44 \cdot in^2$

Allowable Stresses

from AASHTO Table 13.5.3A for Structural Glued Laminated Softwood Timber

$F_b := 2400 \cdot \frac{lb}{in^2}$ $F_v := 200 \cdot \frac{lb}{in^2}$

Moisture_Condition := 1 1 for Wet Use & 0 for Dry Use

$C_{mv} := if(Moisture_Condition = 1, .875, 1)$ $C_{mv} = 0.875$ (AASHTO 13.5.4.1)

$C_{mb} := if(Moisture_Condition = 1, .8, 1)$ $C_{mb} = 0.8$

$C_D := 0.9$ (AASHTO 13.5.4.2)

$C_f := 1.0$ (AASHTO 13.6.4.5)

Figure 1-Page 1 of 4 of Mathcad® 6+ Worksheet

$x = 10$ for all other species $x = 20$ for Southern Pine

$$C_V := \left(\frac{21}{Bridge_Span}\right)^{\frac{1}{x}} \cdot \left(\frac{12 \cdot in}{d_{Effective}}\right)^{\frac{1}{x}} \cdot \left(\frac{5.125 \cdot in}{b_{Effective}}\right)^{\frac{1}{x}} \qquad C_V = 0.76 \qquad \text{(AASHTO 13.6.4.3)}$$

$C_L := 1.0$ Beam must be adequately braced with no danger of laterial buckling. (AASHTO 13.6.4.4)
Use the lesser of CV or CL in the equation given in AASHTO 13.6.4.1

$$F'_b := F_b \cdot C_{mb} \cdot C_D \cdot C_f \cdot C_V \qquad F'_b = 1308.99 \cdot \frac{lb}{in^2} \qquad \text{(AASHTO 13.6.4.1)}$$

$$F'_v := F_v \cdot C_{mv} \cdot C_D \qquad F'_v = 157.5 \cdot \frac{lb}{in^2} \qquad \text{(AASHTO 13.6.5.3)}$$

$$F'_{b_Inv} := F'_b \qquad F'_{b_Inv} = 1308.99 \cdot \frac{lb}{in^2} \qquad F'_{b_Oper} := F'_b \cdot 1.33 \qquad F'_{b_Oper} = 1740.96 \cdot \frac{lb}{in^2}$$

$$F'_{v_Inv} := F'_v \qquad F'_{v_Inv} = 157.5 \cdot \frac{lb}{in^2} \qquad F'_{v_Oper} := F'_v \cdot 1.33 \qquad F'_{v_Oper} = 209.48 \cdot \frac{lb}{in^2}$$

Calculation of Dead Load Moments and Shears

$$DL_{Deck} := S_{Beam} \cdot t_{Plank} \cdot Wt_{Wood} \qquad DL_{Deck} = 159.75 \cdot \frac{lb}{ft}$$

$$DL_{Beam} := b \cdot d \cdot Wt_{Wood} \qquad DL_{Beam} = 150.15 \cdot \frac{lb}{ft}$$

$$DL_{Running_Surface} := t_{Running_deck} \cdot S_{Beam} \cdot Wt_{Wood} \qquad DL_{Running_Surface} = 94.67 \cdot \frac{lb}{ft}$$

$$DL_{Total} := DL_{Deck} + DL_{Beam} + DL_{Running_Surface} \qquad DL_{Total} = 404.57 \cdot \frac{lb}{ft}$$

$$M_{DL} := \frac{DL_{Total} \cdot (Bridge_Span \cdot ft)^2}{8} \qquad M_{DL} = 116.52 \cdot ft \cdot kips$$

Horizontal Shear

The location of the maximum vertical shear occuring at the lesser of 3d or L/4 per AASHTO 13.3.1

$$Dist := if\left(3 \cdot d < \frac{1}{4} \cdot Bridge_Span \cdot ft, 3 \cdot d, \frac{1}{4} \cdot Bridge_Span \cdot ft\right) \qquad Dist = 12 \cdot ft$$

$$R_{DL} := \frac{DL_{Total} \cdot (Bridge_Span \cdot ft)}{2} \qquad R_{DL} = 9.71 \cdot kips$$

$$V_{DL} := R_{DL} - DL_{Total} \cdot Dist \qquad V_{DL} = 4.85 \cdot kips$$

Calculation of Live Load Moments and Shears

This worksheet is set up for calculating HS20 live load moments and shears for simple span bridges. The worksheet can also be set up for four other standard vehicle Truck-Types: 1 - HS20,2 - U54,3 - U80,4 - U102 & 5 - L90. (see distribution factors for live load below.) The live load shear has been computed for a distance 3 times the beam depth or the quarter point, which ever is less. The shear at this point has been approximated by using a shear envelope approach.

Truck-Type : = 1

Figure 2-Page 2 of 4 of Mathcad® 6+ Worksheet

$M_{HS20} := mvldStdMaxM(Bridge_Span, Truck_Type) \cdot ft \cdot kips$ $\quad M_{HS20} = 296.08 \cdot ft \cdot kips$

$R_{HS20} := mvldStdMaxV(Bridge_Span, Truck_Type) \cdot kips$ $\quad R_{HS20} = 29 \cdot kips$ At support

$x := if(Dist + 28 \cdot ft < Bridge_Span \cdot ft, 36 \cdot kips, if(Dist + 14 \cdot ft < Bridge_Span \cdot ft, 32 \cdot kips, 16 \cdot kips))$ $\quad x = 36 \cdot kips$

$V_{HS20} := R_{HS20} - Dist \cdot \left(\frac{x}{Bridge_Span \cdot ft} \right)$ $\quad V_{HS20} = 20 \cdot kips$

Impact Factor

AASHTO 3.8.1.2 - None required for timber members.

I :=0

Distribution of Live Load Moment

The following is the live load distribution factor for bridges designed for one traffic lane from AASHTO Table 3.23.1. Beam Spacing in feet.

Kind Of Floor	Distribution Factor
Timber planks on timber beams	Beam Spacing / 4.0
Nail laminated 4" thick on timber	Beam Spacing / 4.0
Nail laminated 6" thick or greater on timber	Beam Spacing / 4.0
Glued Laminated 4" thick on glulam stringers	Beam Spacing / 4.5
Glued Laminated 6" thick or greater on glulam stingers	Beam Spacing / 6.0

Note: if beam spacing is greater than 6 feet. Assume flooring between stringers acts as a simple beam.

$DF_{LL} := \frac{S_{Beam}}{6.0 \cdot ft}$ $\quad DF_{LL} = 0.95$

$M_{LL} := M_{HS20} \cdot (1 + I) \cdot DF_{LL}$ $\quad M_{LL} = 280.29 \cdot ft \cdot kips$

Load Rating - Moment

$M_{R_inv} := F'_b \cdot S_x$ $\quad M_{R_inv} = 399.97 \cdot ft \cdot kips$

$M_{R_oper} := F'_{b_Oper} \cdot S_x$ $\quad M_{R_oper} = 531.97 \cdot ft \cdot kips$

$RF_{M_inv} := \frac{M_{R_inv} - M_{DL}}{M_{LL}}$ $\quad RF_{M_inv} = 1.01$

$RF_{M_oper} := \frac{M_{R_oper} - M_{DL}}{M_{LL}}$ $\quad RF_{M_oper} = 1.48$

$HS20_Inv_Rating_M := RF_{M_inv} \cdot 36 \cdot ton$ $\quad HS20_Inv_Rating_M = 36.41 \cdot ton$

$HS20_Oper_Rating_M := RF_{M_oper} \cdot 36 \cdot ton$ $\quad HS20_Oper_Rating_M = 53.36 \cdot ton$

Distribution of Live Load Vertical Shear

The following is the live load distribution factor from AASHTO 13.6.5. The distribution shall be determined by the following equation:

$$V_{LL} = 0.50 * [(0.6 * V_{LU}) + V_{LD}]$$

Figure 3-Page 3 of 4 of Mathcad® 6+ Worksheet

$V_{LU} := V_{HS20}$ $\qquad$ $V_{LD} := DF_{LL} \cdot V_{HS20}$

$V_{LL} := 0.5 \cdot \left[\left(0.6 \cdot V_{LU}\right) + V_{LD} \right]$ $\qquad$ $V_{LL} = 15.47 \cdot kips$

Load Rating - Shear

$V_{R_oper} := \frac{2}{3} \cdot A_V \cdot F'_{v_Oper}$ $\qquad$ $V_{R_oper} = 60.39 \cdot kips$

$RF_{V_inv} := \frac{V_{R_inv} - V_{DL}}{V_{LL}}$ $\qquad$ $RF_{V_inv} = 2.62$

$RF_{V_oper} := \frac{V_{R_oper} - V_{DL}}{V_{LL}}$ $\qquad$ $RF_{V_oper} = 3.59$

$HS20_Inv_Rating_V := RF_{V_inv} \cdot 36 \cdot ton$ $\qquad$ $HS20_Inv_Rating_V = 94.39 \cdot ton$

$HS20_Oper_Rating_V := RF_{V_oper} \cdot 36 \cdot ton$ $\qquad$ $HS20_Oper_Rating_V = 129.26 \cdot ton$

Final Load Rating

This final step compares the capacity of the bridge in shear and bending. The result is the less of the two.

```
Inventory_Load_Capacity:= | HS20_Inv_Rating_V   if   HS20_Inv_Rating_V<HS20_Inv_Rating_M
                          | HS20_Inv_Rating_M   otherwise
Operating_Load_Capacity:= | HS20_Oper_Rating_V  if  HS20_0per_Rating_V<HS20_Oper_Rating_M
                          | HS20_Oper_Rating_M   otherwise
```

This **bridge has a HS20** Inventory_Load_Capacity = 36.41 •ton

This bridge has a HS20 Operating_Load_Capacity = 53.36 •ton

Figure 4-Page 4 of 4 of Mathcad ® 6+ Worksheet

Summary

The Mathcad® 6+ worksheet and QuickBASIC program have worked well for load rating this type of structure. Alterations to the Mathcad® 6+ worksheet may be made with ease for load rating other bridge types, but are not so easy for the basic program. Both load rating applications have their downside. The Mathcad 6+ worksheets are quite lengthy, but all equations used in load rating maybe viewed and checked. The basic program has a smaller output, but acts as a "black box". All equations are hidden and cannot be checked. The Mathcad 6+ worksheet maybe tailored to fit any specific structure and saved for future use. The basic program does not have this function at this time, but maybe modified in the future to allow for saving a file.

Load rating bridges using Mathcad® 6+ or the QuickBASIC program turned out to be a very economical and viable solution for this bridge type and with minor effort could be implemented for other types of bridges. In my opinion, the Mathcad® 6+ worksheet is the most versatile and viable application for load rating bridges.

Conversion Table

1 inch = 2.54 centimeters
1 foot = .3048 meters
1 pound = .454 kilograms
1psi = 6.895 kiloPascals
1 ton = 907.185 kilograms

```
*****************************************************************
                    Glue-Laminated Bridge Rating
*****************************************************************

Location:          Zarembo Island
Bridge :           McCormick Creek
Last Inspection :   7/15/95
Operator:          JSG
Date        :      7/15/95
-------------------------------------------------------------------------------
        Bridge Clear Span, Feet.......................... 48
        Bridge Width, Feet............................... 16
        Number of Glulam Beams........................... 3
        Location of Beam being analyzed.................. Interior
        Beam Height, Inches.............................. 51
                Assumed Height Reduction, Inches.........  0
        Beam Width, Inches............................... 8.5
                Assumed Width Reduction, Inches..........  0
        Deck Thickness, Inches........................... 6.75
        Running Surface Thickness, Inches................ 4
        Total Other Dead Load, lb/ft..................... 0
        Wood Type & Grade................................ Douglas Fir
                Basic Allowable Bending Stresses, psi.... 2400
                Basic Allowable Shear Stress, psi........ 200
        Assumed Moisture Condition....................... Wet
        Load Duration Factor............................. .9
        Deck Type ....................................... Glulam Panels
- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
Without considering impact and load distribution factors:

        Max.  Allowable  Inventory  Moment  =  400.0  ft-kips
        Max.  Allowable  Operating  Moment  =  532.0  ft-kips
        Max.  Allowable  Inventory  Shear   =  45.4   kips
        Max.  Allowable  Operating  Shear   =  60.4   kips

Considering appropriate impact and load distribution factors:

        HS20 (32.66 Metric Tons) Inventory Rating  =  36.4 Tons - controlled by moment
                                 Operating Rating  =  53.4 Tons - controlled by moment
*****************************************************************
```

Figure 5 - Basic Program Output

References

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Perception versus Reality: An Analysis of Timber Bridge Performance in the United States.

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Kim Stanfill-McMillan, Forest Products Laboratory, USDA Forest Service

Abstract

Bridge material selection is one of the most difficult decisions an engineer has to make. Many factors and individuals are often involved in choosing the proper bridge material for a given site and location. Not only physical factors such as strength or lifespan of material, but also site specific factors like roadway alignment and traffic count play important roles in material selection. It is not uncommon for state Department of Transportation engineers, private consulting engineers, and local highway officials all play roles in the material selection process. Each individual may have his/her own perception of bridge materials based upon past experience and education. And little is known how these perceptions influence the choice of materials. In this study perceptions of engineers and highway officials toward timber as a bridge material were compared to the actual performance of timber as reported in the National Bridge Inventory. To accomplish this case studies were conducted in four selected states. Highway officials and engineers in Mississippi, Virginia, Washington, and Wisconsin were surveyed by mail and personally interviewed to capture their perceptions toward timber as it compared to other major bridge materials (prestressed concrete, steel, and reinforced concrete). This information was compared with the actual performance data obtained from the National Bridge Inventory. The results indicate that there is a strong correlation between highway officials' perceptions towards bridge materials and the reported performance of these materials.

Keywords: Perceptions, National Bridge Inventory, timber, steel, concrete, performance.

Introduction

The need for bridge replacement has been well documented (Brungraber et al. 1987, Cheney 1986, USDA 1989). Over 40% of our Nation's bridges are in need of repair or replacement. According to the FHWA (1992) four major structural materials make up over 98% of all bridges in the United States. These include prestressed concrete (15%), steel (36%), reinforced concrete (40%), and timber (8%). However, since 1982 over seventy percent of the replacement bridges have been prestressed or reinforced concrete, while timber and steel were used in less than thirty percent of replacement structures. This suggests that perceptions toward prestressed and reinforced concrete by highway officials are better than that of competing materials.

The United States has more than 3.9 million miles of roadway and 575,000 bridges. In 1967, in response to the collapse of the Silver River bridge over the Ohio River, Congress mandated the implementation of National bridge inspection standards. The individual bridge inspection records constitute the National Bridge Inventory (NBI). The purpose of the NBI is to provide a uniform base of bridge information that can be used to identify those bridges that are most in need of repair and to serve as a basis for allocating Federal Highway Administration (FHWA) funding for bridge replacement or rehabilitation. The NBI is administered by the FHWA in Washington, D.C. Data are updated continuously based on the latest bridge inspection, which are usually completed on a two year cycle.

All too often the evaluation of competing products is limited to tangible characteristics such as price and physical attributes, disregarding intangibles such as consumer perceptions and attitudes (Dickson 1974). However, it is these perceptions that determine a belief about a material, which may influence the design decision. Although the importance of perceptional variables is well established in marketing literature (Green and Carmone 1970, Johnson 1971, Lehmann and O'Shaughnessy 1974), little information is available concerning perceptions of competing bridge materials or actual performance.

Clapp (1990) and Luppold (1990) conducted qualitative studies concerning highway official's perceptions of timber in the Northwest and South, respectively. Both concluded that timber was not perceived well by these officials as a bridge material. Dunker and Rabbat (1992) conducted an extensive analysis of the National Bridge Inventory to compare the performance of prestressed concrete to other bridge materials since 1950. They concluded that prestressed concrete out-performed all other materials in the past 40 years, with timber being the poorest performing material.

Nationwide, the number of timber bridges is declining. Between 1982 and 1992 the number of timber bridges declined from 71,200 to 45,863, an average of 2500 timber bridges per year (Stanfill-McMillan and Hatfield 1992). With a thorough understanding of how timber has performed and how timber is perceived by highway officials, reasons for this decline can be identified.

Objectives

1. Determine highway official's perception of timber as a bridge materials in four states.

2. Evaluate the performance of timber as a bridge material based upon the National Bridge Inventory.

3. Compare the performance of bridge materials to the perceptions of highway officials toward these materials in four states,

Methodology

Perception

To measure the perceptions of highway officials across the U.S., a mail survey was conducted in the winter of 1993. Over 1200 questionnaires were mailed to highway officials in twenty-eight states. These officials were segmented into three groups: State Department of Transportation (DOT) engineers (401 sampled), private consulting engineers (419 sampled), and local highway officials (406 sampled).

To determine if differences exist between geographic regions, five segments were identified. These included the Northwest, South, Midatlantic, Northeast, and Midwest. These five regions accounted for 70% all of the bridges built between 1982 and 1991 (FHWA 1992). These regions were chosen due to their different timber bridge usage, different bridge selection protocol, and different timber resources. The four states studied in depth included Mississippi (South), Virginia (Midatlantic), Washington (West), and Wisconsin (Midwest).

The questionnaire consisted of three sections. The first section used rating scales to collect data concerning perceived overall bridge material performance and the past experiences with various bridge materials. The second section of the questionnaire used rating questions to collect data concerning how did timber compare with prestressed concrete, steel, and reinforced concrete on eight preselected attributes. The third section consisted of multicotomous questions designed to gather information about the respondents. In particular, individuals were asked about past exposure to timber engineering.

The questionnaire was reviewed by knowledgeable civil engineers and university personnel to test its face validity, clarity, and to ensure that no important bridge material selection factor was overlooked. A pretest was then conducted with bridge officials in various groups in Virginia, Wisconsin, and Minnesota. The responses of the pretest were used to clarify question wording and revise the set of material attributes and factors used in the decision making process.

After tabulation of data, in depth personal interviews were conducted in Mississippi, Virginia, Washington, and Wisconsin to further identify highway official's perceptions toward timber bridges and to better understand their material decision making process. Highway officials in each group were interviewed to access their opinions on the use of timber in bridges.

Reality

To identify how the various bridge materials actually performed, the 1992 National Bridge Inventory data were obtained from the Federal Highway

Administration. The NBI is a comprehensive database that maintains inventory records on all bridges in the United States that are over 20 feet or greater in length located on public roads. The NBI is primarily used by individual states and the Federal government for assessing bridge condition and assigning priorities for future projects. It also contains much useful information on the historical trends in bridge construction, adequacy, and longevity. The NBI contains an inventory of over 575,000 bridges, of which approximately 100,000 are culverts. These culverts were not included in this study.

The inventory contains current detailed data on individual bridges which includes such topics as bridge location, configuration, material, age, and condition. Based on inspection data, each bridge in the NBI is assessed a performance rating of structurally deficient (SD), functionally obsolete (FO), or satisfactory. A structurally deficient bridge must meet one of three criteria: the condition of the deck, superstructure, or substructure is rated poor; the structural evaluation or overall bridge condition is rated below a 3; or the waterway adequacy is not sufficient. To a great extent, criteria for rating a bridge structurally deficient are associated with material performance of the superstructure, substructure, or deck.

Structural deficient bridge data were examined to determine the reasons a bridge was classified as structurally deficient (SD). This was done to determine if a relationship exists between the bridge material and the reason for the deficiency. Bridges are classified as SD if they received a low rating in at least one of the five following categories: deck, superstructure, substructure, waterway, or structural evaluation.

After evaluating a bridge against the criteria for a structural deficient rating, the nonstructurally deficient bridges are evaluated against criteria for a rating of functionally obsolete (FO). The criteria for a FO rating require that at least one of five conditions is met: the deck geometry is too narrow for traffic; the vertical or horizontal underclearance is not adequate; the roadway alignment requires substantial reduction in vehicle operating speed; the structural evaluation rating is below a 3; or the waterway adequacy is such that the frequency of flooding is less than that for SD bridges, but still unacceptable. With the exception of the structural evaluation rating, FO ratings are based more on geometry and less on material performance.

Of the approximately 45,800 timber bridges classified by the FHWA, over 37,000 (81%) are timber bridges with timber decks. The remaining 19% have other deck materials on timber bridges. There also exists over 42,000 non-timber bridges with timber decks. These consist primarily of timber decks on steel bridges. For the purpose of this study, only timber bridges with timber decks are considered, because they represent the largest number in this category.

Results and Discussion

Forty percent of all respondents to the mail survey were from state DOT offices, 30 percent were private consultants, and 30 percent were local highway officials. Nearly 70 percent of the respondents reported that their state had standard bridge plans, but only one-third of these respondents mentioned that the plans included designs for timber.

Highway officials were asked to state the materials they had used in the past five years in bridge design or replacement. Eighty-four percent had used prestressed concrete, 79 percent had utilized reinforced concrete, 68 percent had used steel in bridges, and 46 percent had experience with timber during the past five years. Approximately 40 percent of all respondents had taken a formal course in timber design, with one-third saying it was mandatory. Fifty-four percent said they were aware of the recent changes in timber design, with one-half saying these changes had improved their impression of timber as a bridge material.

Figure 1. Average rating of bridge material from all 28 states.

Nationwide, prestressed concrete was rated the highest, followed by reinforced concrete, steel and timber (Figure 1). The four states studied also followed this trend. Each state rated prestressed concrete the highest. Steel was rated the highest in Washington (5.1), timber was rated the highest in Wisconsin (4.3). and reinforced

concrete was rated the highest in Virginia (6.3) (Table 1). One-way analysis of variance indicated that perceptions by highway officials toward bridge materials do differ significantly by state.

Table 1- Average material performance ratings by highway officials in selected states.

	MS	VA	WA	WI
Pres.	6.2	6.4	5.5	5.3
Steel	4.6	4.3	5.1	4.4
Timber	3.9	3.4	3.3	4.3
Reinfc	5.9	6.3	5.1	5.0

* Rating scale from 1, below average, to 7, above average.

The three decision groups responding from the four states rated prestressed concrete the highest, followed by reinforced concrete, steel, and timber. Steel was rated the highest by state DOT engineers, timber was rated the highest by local highway officials, and reinforced concrete was rated the highest by state DOT officials. One-way analysis of variance indicated that their perceptions of bridge materials were similar across decision groups (Figure 2).

Figure 2. Average material performance rating by decision group in four states.

Eight material attributes were identified by civil engineers across the country as important to material selection for bridges. Timber was compared on a scale from 1 (low) to 7 (high), to other bridge materials on these attributes which included: *Long life, high strength, low maintenance, pleasing aesthetics, environmentally safe, low cost, easy to design, and easy to construct.* These ratings identified areas in which timber must improve its performance to gain increased market acceptance (Table 2).

Wisconsin rated timber highest on five of the attributes: *Long life, high strength, low maintenance, pleasing aesthetics and easy to construct.*

Table 2 - Material attribute rating by state

Attribute	WA	MS	VA	WI	Overall
Long life					
Pres	6.3	**6.4**	5.3	4.5	5.9
Steel	4.9	5.4	**5.6**	4.5	5.0
Timber	3.8	3.7	3.5	**4.4**	4.0
Reinfc	6.0	**6.3**	5.6	5.3	5.8
High strength					
Pres	5.9	**6.5**	5.7	5.6	5.9
Steel	5.8	**6.0**	5.9	5.4	5.7
Timber	3.7	4.0	3.1	**4.3**	3.9
Reinfc	5.1	**6.3**	4.6	5.3	5.4
Low maintnce					
Pres	**6.2**	6.1	5.6	5.5	5.9
Steel	4.0	**4.9**	3.9	3.8	4.1
Timber	3.5	3.7	3.1	**4.7**	3.9
Reinfc	5.8	**6.2**	5.0	5.4	5.6
Pleasing aesthetics					
Pres	5.0	**5.6**	5.3	4.8	5.1
Steel	4.6	**5.4**	5.3	4.7	4.8
Timber	4.8	4.7	4.7	**5.4**	4.9
Reinfc	4.9	**5.8**	5.1	4.7	5.1
Environ. safe					
Pres	5.5	**6.2**	5.7	5.8	5.8
Steel	4.4	4.9	**5.2**	4.1	4.6
Timber	4.5	**4.7**	4.6	4.6	4.6
Reinfc	5.1	4.9	**5.2**	4.6	5.5
Low cost					
Pres	**5.1**	4.6	4.8	4.9	4.8
Steel	3.9	4.2	**4.7**	3.7	4.0
Timber	4.6	**5.0**	3.9	3.7	4.3
Reinfc	4.8	**4.9**	4.5	4.8	4.7
Easy to design					
Pres	5.1	5.1	4.5	**5.2**	5.0
Steel	4.3	4.9	**5.8**	4.6	4.8
Timber	4.8	**4.9**	4.4	4.7	4.7
Reinfc	5.0	**5.4**	5.2	5.0	5.2
Easy to construct					
Pres	**5.6**	5.3	4.9	5.2	5.3
Steel	4.3	4.8	**5.3**	4.6	4.7
Timber	4.8	5.2	4.5	**5.2**	5.0
Reinfc	4.5	**5.3**	4.3	4.3	4.6

Mississippi, which have the most timber bridges of states interviewed rated timber highest in 3 attribute categories: *environmentally safe, low cost, and easy to design.* It can be noticed that Wisconsin was basing their decisions on material attributes, while Mississippi appears to be basing their decision on cost and ease of design.

Table 2 further illustrates that timber is perceived as the poorest bridge material in terms of the following attributes: *long life, high strength, low maintenance, environmentally safe, and easy to design.* Prestressed or reinforced concrete was rated the highest in terms of perceived performance on every attribute. Only steel was rated lower than timber on the attributes of *easy to design, pleasing aesthetics, and low cost.* Analysis of Variance (ANOVA) indicated that no statistical differences existed between the four states on the attributes of *pleasing aesthetics, environmentally safe, easy to design, and easy to construct.* On the remaining attributes measured, differences did exist among the states highway officials.

National Bridge Inventory data were used to determine the actual performance of bridges. Table 3 illustrates the performance rating of bridges by material type and state. Mississippi not only has the most number of timber bridges, but also the most deficient number of timber bridges. Personal interviews in Mississippi revealed that this state does not have standard bridge plans for timber. To receive federal cost sharing revenues, the state only allows bridge projects using standard plans. So the timber bridges in Mississippi may not be being built to HS-20 loading requirements and this may be one reason for the high number of deficient timber bridges in the state.

Virginia has the fewest number of timber bridges (67) and 91% of these were classified as deficient. Virginia also rated timber lower in over-all perceived performance. Virginia does not have standard plans for timber bridges, however it has some designs for the use of timber on steel bridges. Personal interviews indicated that much of the perceptions of timber are based on this use of timber as a decking material on steel stringers or trusses.

Washington, which has 75% of its 912 timber bridges classified as deficient, rated timber the lowest in over-all perceived performance. Washington does not have standard bridge plans for timber and state engineers stress standard plans when allocating federal highway funding.

Wisconsin is the only state that has standard timber bridge plans of those interviewed. More than 80% of its 519 timber bridges have a satisfactory rating. The state also rated timber the highest on perceived performance. Wisconsin also rated timber high on every attribute but *easy to design, low cost, and environmentally safe.* This data indicates that properly designed and constructed timber bridges have a great impact on highway official's perceptions material performance in the bridge application.

Table 3 - National Bridge Inventory ratings of bridge materials by state 1992.

State	Material	# Bridges	Sat. %	#SD %	#FO %
MS	Reinfoc	5387	3922 (73%)	700 (13%)	765 (14%)
	Steel	2940	761 (26%)	1537 (52%)	642 (22%)
	Timber	4045	138 (3%)	3803 (94%)	104 (2%)
	Pres.	2398	2069 (86%)	23 (1%)	306 (13%)
VA	Reinfoc	2712	1538 (57%)	376 (14%)	798 (29%)
	Steel	6527	3885 (60%)	1031 (16%)	1611 (24%)
	Timber	67	6 (9%)	40 (60%)	21 (31%)
	Pres.	845	711 (84%)	16 (2%)	118 (14%)
WA	Reinfoc	3039	1345 (44%)	284 (9%)	1410 (47%)
	Steel	840	233 (28%)	185 (22%)	422 (50%)
	Timber	912	227 (25%)	150 (16%)	535 (59%)
	Pres.	1968	1152 (59%)	150 (8%)	666 (33%)
WI	Reinfoc	3192	2250 (70%)	813 (25%)	159 (5%)
	Steel	5223	2799 (54%)	2046 (39%)	378 (7%)
	Timber	519	419 (81%)	85 (16%)	15 (3%)
	Pres.	2281	1971 (86%)	239 (11%)	71 (3%)

Source: Stanfill-McMillan and Hatfield 1995 from FHWA 1992.

The average age of satisfactory timber bridges in Mississippi, Virginia, and Washington exceeds the average age of other bridge materials. Only in

Wisconsin is the average age of timber less than steel and reinforced concrete. With the exception of prestressed concrete, the average age of structural deficient bridges appears to be similar among material types. Prestressed concrete has lower average age in every state because it is the newest bridge material developed. Mississippi has the lowest average age of structural deficient bridges of all materials, being approximately 34 years. This compared to 46 years in Virginia, 57 years in Washington, and 48 years in Wisconsin. The average age of functionally obsolete bridge materials appear to be independent of material type. (Table 4).

Table 4. Average age (years) of bridges in selected states.

State	Material	# Bridges	Age Sat. (yrs)	Age SD (yrs)	Age FO (yrs)
MS	Reinfoc	5387	21.3	34.0	34.2
	Steel	2940	33.9	39.2	47.2
	Timber	4045	34.3	32.2	31.8
	Pres.	2398	16.7	30.3	28.1
VA	Reinfoc	2712	35.7	57.7	52.0
	Steel	6527	26.2	50.8	41.8
	Timber	67	40.7	53.4	46.1
	Pres.	845	23.3	24.3	26.6
WA	Reinfoc	3039	41.6	48.9	44.2
	Steel	840	35.6	73.4	52.5
	Timber	912	47.9	82.3	51.2
	Pres.	1968	19.5	25.6	20.7
WI	Reinfoc	3192	25.6	56.7	48.3
	Steel	5223	37.9	55.4	48.1
	Timber	519	23.4	48.4	45.8
	Pres.	2281	18.3	29.8	23.4

Source: Stanfill-McMillan and Hatfield 1995 from FHWA 1992.

Timber bridge performance varied more according to route class than other materials. Route class refers to the road classification by the FHWA. Satisfactory timber performance is lower on county highways and city streets. However, timber ranked significantly higher on U.S. Numbered highways, where bridges will be usually designed for the state minimum load capacity (Table 5). The average age of satisfactory timber bridges on State highways is 53 years, which is higher than the average age of all satisfactory timber bridges. More satisfactory timber bridges were found on State and Federal lands than on county and city highways. Again, it can be expected that bridges located on major roads are designed to HS-20 load ratings than bridges built on local road systems.

The design of the timber bridges appears to have an impact on its performance. Throughout the U.S., the most common type of timber design is a stringer/beam/girder, representing nearly 90% of the 37,000 complete timber bridges. However, this design has only 29% satisfactory rating. The second most common design is the timber slab. Nearly 2000 timber bridges fall in this category, but over 85% of these are rated satisfactory. The third most common design is the frame style, which there are over 800 in service. Only 8% of these are rated satisfactory. Although there are numerous other designs, these three types make up over 97% of the timber bridges in service (Stanfill-McMillan and Hatfield 1992).

Table 5. Performance ratings and number of timber bridges by route class throughout the U.S.

		Number of Bridges (%)		
Route Class	Total	Satisfactory	SD	FO
Interstate	66	27 (44%)	28 (46%)	6 (10%)
U.S. Numbered	708	540 (76%)	97 (14%)	71 (10%)
State Hwy.	5117	2177 (43%)	2100 (41%)	840 (16%)
County Hwy	36701	10665 (29%)	21041 (57%)	4995 (14%)
City Street	1580	377 (24%)	874 (55%)	329 (21%)
Federal Land	215	127 (59%)	53 (25%)	35 (16%)
State Land	205	93 (45%)	66 (32%)	46 (22%)
Other	1272	634 (50%)	449 (35%)	189 (15%)
Total	45859	14640 (32%)	24708 (54%)	6511 (14%)

Source: Stanfill-McMillan and Hatfield 1995 from FHWA 1992.

The basis for classification of the bridge as structural deficient (SD) is a good indicator of material performance. Nationwide, the main reason that prestressed concrete is rated SD is due to low ratings on the deck. Reinforced concrete is rated lower on the substructure, steel is rated low on the deck and substructure, while timber is rated low on substructure and load or waterway capacity (Table 6). Nationally, timber bridges had the lowest percentage of deficiency

rating for the deck and superstructure. Table 6 shows that 39% of timber bridges were rated SD as a result of inadequate load capacity or waterway adequacy, not as a result of material condition.

Timber bridge data were divided into three categories according to design load ratings. The categories were: design loads of (1) H-20, HS-20, or HS-25, (2) H-10, HS-10 or H-15, and (3) other. Nationwide 58% of

Table 6. Basis for bridge classification as structurally deficient by material (No. & %).

Rating Criteria	Reinfc Concrete	Steel	Pres Concrete	Timber
Deck (1)	1361 (7.3%)	8407 (12.%)	924 (25.8%)	1199 (5%)
Super-structure (2)	1823 (9.7%)	7214 (10.1%)	433 (12.1%)	1462 (6%)
Sub-structure (3)	4976 (26.5%)	11773 (16.8%)	988 ((27.6%)	5112 (20%)
Two of three locations (1,2, or 3)	4358 (23.2%)	13309 (19.0%)	417 (11.6%)	3433 (11.6%)
All three locations (1,2, and 3)	3117 (16.6%)	9054 (12.9%)	164 (4.6%)	3906 (16%)
Load or waterway capacity	3116 (16.6%)	20419 (29.1%)	659 (18.4%)	9597 (39%)
Total	18,751	70,176	3585	24,709

Source: Stanfill-McMillan and Hatfield 1995 from FHWA 1992.

timber bridges with timber decks are not coded for a standard design load and of these 28% are satisfactory (Stanfill-McMillan and Hatfield, 1995).

Of the four states studied, nearly all bridges in Mississippi were not designed to the current HS-20 load rating as required by the FHWA for the use of Federal funding (Table 7). Less than 1% of these bridges hold this classification. Nearly all of Mississippi's bridges are classified as structurally deficient. One of the reasons for this SD classification is that the bridges were never designed for the HS-20 loading or traffic on the system has increased to now warrant a higher rating.

In Virginia all timber bridges hold an unknown or less than a HS-20 load rating. Only 5 bridges in the state are classified as satisfactory. Again, since the bridge may never have been designed for the HS-20 rating, it is classified as deficient. Virginia recently built one timber bridge in each highway district under a state timber bridge initiative program. They are currently evaluating these structures to measure cost effectiveness and long term performance.

Table 7. Distribution and rating of timber bridges

State Rating	H-10, HS-10, HS-15 No. (%)	H-20, HS-20, HS-25 No. (%)	Other No. (%)
Mississippi	2644 (70%)	2 (<1%)	1143 (30%)
Satisfactory	74 (3%)	0	42 (4%)
SD	2497 (94%)	1 (50%)	1078 (94%)
FO	73 (3%)	1 (50%)	23 (2%)
Virginia	25 (38%)	0	41 (62%)
Satisfactory	2 (8%)	0	3 (7%)
SD	11 (44%)	0	29 (71%)
FO	12 (48%)	0	9 (22%)
Washington	347 (64%)	120 (22%)	71 (13%)
Satisfactory	9 (3%)	7 (6%)	11 (15%)
SD	49 (14%)	18 (15%)	15 (21%)
FO	289 (83%)	95 (79%)	40 (56%)
Wisconsin	23 (24%)	49 (52%)	23 (24%)
Satisfactory	14 (61%)	46 (94%)	7 (30%)
SD	8 (35%)	2 (4%)	14 (61%)
FO	1 (4%)	1 (2%)	2 (9%)

Source: Stanfill-McMillan and Hatfield 1995 from FHWA 1992.

In Washington 120 of the states 538 timber bridges were designed for an H-20 or higher rating. Of these structures, nearly 80% are classified as functionally obsolete. In general, functional obsolete bridge ratings are based more on geometry and less on material performance. Again, although Washington has a large number of deficient timber bridges, it may not be due to the type of material, but the original design of the bridge.

Wisconsin has the most timber bridges designed to the HS-20 rating of those states interviewed. Of those bridges designed to HS-20 or higher, 94% are classified as satisfactory and those with a lower bridge rating had over 60% satisfactory rating. Wisconsin is a state where counties have an active bridge maintenance program. These results suggest that timber bridges that are designed to current standards and maintained properly, will perform well. And this performance will have a direct result on the perceptions of highway officials.

Perception vs. Reality

The results of this research indicate that there is a strong correlation between highway official's perceptions of timber bridges in the four states studied and reported performance of bridges. In Mississippi where nearly all timber bridges are deficient in one area, perceptions of timber were quite low. However, personal interviews in Mississippi reveal that most timber bridges are built to no design criteria. The state DOT requires that Federal funds be allocated only to those bridges that state standard plans exist. There are no standard plans for timber in Mississippi. Although numerous timber bridges are being built in the state, highway officials are basing their decision on low cost and easy construction, not on longevity or good design. Highway officials rated timber the highest on the attributes of *environmentally safe, low cost, and easy to design.* Highway officials interviewed stated that timber bridges were only lasting 25 to 30 years and timber is a high maintenance material. These officials believe that prestressed concrete is the most cost effective material (averaging $30 sq./ft.) and timber offers no competitive advantage in bridge construction.

Virginia had the fewest number of timber bridges of the states studied, but their engineers still had a poor perception of timber. Most timber is used as a decking material on steel bridges, and engineers' perceptions have been based primarily on this factor. Highway officials in Virginia rated timber lowest on six of the eight material attributes measured. Of the 67 timber bridges in the state, only 6 are classified as satisfactory. However, none of these bridges currently have a HS-20 rating. And over 60% have no rating on the structure at all. Virginia was the only state interviewed where the state DOT controlled over 97% of all the roads. The state does not have standard plans which include timber. Most bridges are designed by state engineers in one of nine highway districts. When interviewed, engineers stated that maintenance problems and poor performance have been problems with timber.

Washington has the second largest number of timber bridges of those states interviewed. However, their highway official's perception of timber was the poorest. Only 25% of Washington's timber bridges are rated satisfactory. Of these bridges, nearly 60% are classified as functionally obsolete. And the average age of Washington's timber bridges exceeds 50 years. The average age of satisfactory timber bridges exceeds that of all other bridge materials. Over 75% of their bridges have less than an H-20 rating. Although state highway officials have a poor perception of timber and its attributes, the bridges that are in service appear to have performed well and are rated low because of design criteria, not material performance.

The state with the most satisfactory timber bridges is Wisconsin. Over 80% of Wisconsin's timber bridges are rated satisfactory. This state had the highest perceptional rating of timber also. It is the only interviewed state that has standard bridge plans for timber and regularly builds timber bridges using FHWA funding. This states' highway officials rated timber the highest of all states on the attributes of *longlife, high strength, low maintenance, pleasing aesthetics, and easy to construct.* The average age of Wisconsin's deficient bridges was quite similar to reinforced concrete and steel. Over 90% of the timber bridges that have a HS-20 or above rating are rated as satisfactory. This states' counties are in charge of rural bridge maintenance. Wisconsin is the only one of the four states that have seen an increase in timber bridge use since 1986.

Conclusion

This study sought to determine how the performance of timber as a bridge material as reported by the FHWA's National Bridge Inventory compared to highway official's perceptions of timber in four selected states. Nationally, timber was rated the poorest in perceived performance by all highway officials. It was also rated lowest by officials in the four states studied.

Prestressed concrete was rated highest, followed by reinforced concrete, steel and finally timber.

Timber bridges were rated higher in structural deficiency and as functionally obsolete in three of the four states. However, a closer look at these ratings indicate that most of the timber bridges were rated low since they were not designed to HS-20 loading requirements. Nearly 40% of the timber bridges rated structurally deficient were based upon inadequate load capacity or waterway adequacy. In other words, the reported performance of timber in the NBI is based on factors which may not be a result of the material performance, but more on initial design practices.

In Wisconsin, the only state which has standard bridge plans, over 80% of timber bridges were rated satisfactory. This state rated timber higher in perceived performance that the other three states. Timber bridges on Federal and State Highways were timber bridges are more likely to be built to higher standards. The average age of satisfactory timber bridges was quite similar to other materials. The average age of deficient bridges was also similar, independent of material type.

In conclusion timber bridges are perceived not to perform well, but the NBI data indicates that in those areas where perceptions are poor, timber bridges have not been designed to current standards. Where timber has been designed properly (Wisconsin) perceptions and performance are rated as higher than the other states measured. This research indicates that future timber bridges built to current HS-20 loading requirements should perform better and influence the perception of highway officials in a positive manner. It is recommended that highway officials base their material choice decision not only on their perception of timber, but also on why timber is rated low in the NBI.

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Wood In Transportation Program, Superstructure Costs Report for Vehicular Timber Bridges, 1989-1995

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Abstract

The Wood In Transportation Program, formerly called the National Timber Bridge Initiative, began funding demonstration timber bridges in Fiscal Year 1989. Since the beginning of the Program, 349 demonstration vehicular and pedestrian timber bridges and 58 special projects have been funded in 48 states. The purpose of this report is to provide information on the superstructure costs of vehicular timber bridges funded through the Wood In Transportation Program. This report focuses on the final cost data that the Timber Bridge Information Resource Center (TBIRC) has for vehicular timber bridges. The final cost data is supplied by our cooperators, and the cost averages in this report are presented by region, bridge type, length, and species.

Keywords: timber bridges, superstructure costs, wood transportation structures.

Introduction

All costs in this report are made up of final cost data from completed projects from fiscal years 1989 through 1995. The source of the final cost data is from individual cooperators. As of April 1996, the TBIRC has final cost information on 112 vehicular timber bridge projects throughout the United States. A listing of states with the number of projects in which we have cost information is as follows: Alabama-2, Arkansas-2, Colorado-3, Florida-1, Georgia-2, Idaho-1, Illinois-3, Kansas-3, Maryland-2, Michigan-2, Minnesota-1, Mississippi-2, Montana-4, Nebraska-1, Nevada-1, New Mexico-1, New York-9, North Dakota-1, Ohio-4, Oklahoma-2, Oregon-2, Pennsylvania-10, South Dakota-4, Washington-1, West Virginia-46, Wyoming-2.

Cost information is provided to the TBIRC by cooperators in three formats. The most common is bid tabulation sheets; second is material, labor, design, and engineering receipts; and third is a summary form provided by the TBIRC. At times the TBIRC has requested cost information and has provided the cooperator with a Final Costs and Specifications form for summarizing the cost data. The form is divided into five major sections: preliminary, which includes design, survey, and other related costs; substructure, which includes excavation, abutment, and demolition; superstructure, which includes structure, deck, railing, fabrication, and erection; other costs, which includes site work, approaches, and detours; and surfacing. A small percentage of our cooperators have returned cost data using the Final Cost and Specifications summary form.

Much of the data is provided by cooperators with bid tabulation sheets or project receipts. When cost data is provided with bid tabulation sheets or with project receipts, the costs are summarized by TBIRC using the Final Costs and Specifications form. In previous reports published in *Crossings*, the newsletter of the Timber Bridge Information Resource Center, estimated cost information was included in the calculations. This report uses only final cost information.

Average Regional Cost

In Figure 1, the numbers in the large boxes represent the average dollar superstructure cost for that area. The numbers in the smaller boxes represent the number of bridges for which the TBIRC has final cost information for that region and is used in the calculation average.

Figure 1

The total number of bridges included in the cost data calculation is 112, with overall average cost at $58.28 per square foot of superstructure. The Program has funded 310 vehicular bridge projects from 1989 through 1995, and from that the TBIRC has 254 bridge projects that are completed or are in the process of being completed. The TBIRC has final cost information for 44 percent of our completed projects. We have no final cost information for vehicular bridge projects in Forest Service Region 5 (California) or Region 10 (Alaska).

Average Cost By Type

Through the Wood In Transportation Program, 17 types of vehicular timber bridges have been funded from 1989 through 1995. The types are general classifications for analysis purposes. The bridges are classified as closely as possible to the general type. In Figure 2, average costs have been calculated for 9 of the 17 types of bridges. Seven of the 17 types have no more than two bridges in the database and have not been included in Figure 2. The number in the small box on the right hand side of the chart represents the number of bridges used in the calculation. The most expensive type of bridge is the Mozingo steel/wood composite design developed from research performed at Pennsylvania State University. The Mozingo bridge average cost is $113.42 per square foot of superstructure. The stressed-t and stressed-box designs follow, with average superstructure costs of $68.18 and $64.83 respectively. The average cost of a timber box culvert is $56.23 per square foot of superstructure. Next in cost is the transverse glulam deck with glulam stringers, with a cost of $53.02 per square foot. The stress laminated deck bridge type is also over $50 per square foot, with a cost of $51.07 per square foot of superstructure.

Figure 2

The least costly bridge is the dowel-laminated structure, with a cost of $43.97 per square foot of superstructure. Next are the laminated veneer lumber/parallel strand lumber t-system bridges, with an average cost of $44.15 per foot of superstructure. Following are longitudinal glulam bridges, with a cost of $47.57 per foot of superstructure.

The data indicates that the dowel-laminated structures are the least expensive along with structures using glulam material and engineered wood products, such as laminated veneer lumber (LVL) and parallel strand lumber (PSL).

Average Cost By Length

In Figure 3, average cost by length is divided into seven categories. The categories begin with greater than 50 feet in length and end with less than 20 feet in length, with five divisions between. The number in the small box on the right hand side of Figure 3 represents the number of bridges used in the calculation.

It should be understood that many of the longer span bridges are experimental demonstration bridges and that the average cost does not take into consideration the number of spans involved in the bridges. Of the 31 bridges greater than 50 feet in length, 16 are single span structures, with an average cost of $68.41 per square foot of superstructure. Fifteen bridges are multiple span bridges, with an average cost of $42.64 per square foot of superstructure. Forty-eight percent of the bridges in the greater than 50 foot category are multiple-span.

Figure 3

In the 41 to 50 foot category, only one bridge is multiple-span, and its cost per square foot is $36.37. The balance of the 18 bridges in this category average $59.82 per square foot of superstructure. One of these bridges was a deck replacement on a 252-foot structure.

Average Cost By Species

The data in Figure 4 indicates that the species of wood used to construct a bridge influences the cost of bridge superstructures. Table 1 expands on the data given in Figure 4. Additional information is given for the number of bridges by length, width, and spans for bridges by species.

Figure 4

The 14 bridges built with Douglas fir are the least expensive structures to date. Of these, five are laminated veneer lumber (LVL) t-sections, two are dowel-laminated bridges, two are logitudinal glulam, and two are transverse glulam decks with glulam stringers. All of these bridge types are among the least expensive to build.

Table 1. Number of bridges by length, width, span, by species

Species	No. Bridges	Length <32	Length >=32	Width <=18	Width >18	Single Span	Multiple Span
Douglas Fir	14	3	11	3	11	9	5
Northern Hardwoods	4	1	3	0	4	4	0
Ponderosa Pine	3	1	2	1	2	3	0
Red Maple	6	5	1	0	6	5	1
Red Oak	21	9	12	5	16	20	1
Red Oak/ Southern Pine	30	2	28	7	23	28	2
Southern Pine	12	4	8	3	9	6	6

The three bridges constructed of Ponderosa pine are the second in the Average Cost by Species graphic, with a superstructure cost of $49.81 per square foot of superstructure and are all longitudinal glulam structures.

Third in cost are 21 bridges constructed of red oak. Fourteen of these bridges are logitudinal stress-laminated structures built in West Virginia, with one being a deck replacement. It should be noted that 13 of the red oak structures are a West Virginia Division of Highways design that is no longer used. Five bridges are in Pennsylvania. Four of the Pennsylvania bridges are the Mozingo steel/wood composite design and one is a transverse glulam deck with glulam stringers. One of the 21 red oak bridges is an A-frame design in Maryland, and one is a logitudinal stress-laminated bridge in Ohio.

The 12 structures built with southern pine vary in bridge type and are fourth in the cost graphic. Four of the bridges are transverse glulam panels over glulam stringers, two are box culverts, two are logitudinal glulam, two are logitudinal stress-laminated, and two are not classified because they do not fit the bridge types listed on the Final Costs with Specifications form. Eight of the bridges are in the south, and four are in the northeast.

Red maple species are fifth in cost. All of these bridges are in the northeast, with structure type being three longitudinal stressed decks, one logitudinal stress-laminated and, one transverse glulam panels over glulam girders. One bridge is not classified, because it does not fit the bridge types listed on the Final Costs with Specifications form.

The red oak/southern pine species classification contains the largest number of projects with thirty, of which all are in West Virginia. The classification contains two bridge types; t-stressed and box-stressed, two of the three bridge types used in West Virginia. The square foot superstructure costs are $68.07.

The species classification of northern hardwoods shows the greatest cost in this report, with a cost of $77.49 per square foot of superstructure. The Mozingo design bridge make up two of the four bridges with this classification. One structure is a longitudinal stress-laminated deck over steel girders, and one is a transverse glulam deck over steel girders.

Conclusion

This reports indicates that bridge design type, species of wood, and the region where the bridge is constructed influence the superstructure cost of timber bridges. Regional costs indicate that timber bridges constructed in the northwestern states are less expensive to construct than in the south and east. Superstructure costs for bridge types such as dowel-laminated types, along with bridges using engineered wood products such as laminated veneer lumber, are less expensive structures. The five longitudinal glulam bridges in Figure 2 also cost less than $50 per square foot of superstructure.

Length of timber bridges does not seem to affect costs per square foot of superstructure. Shorter span bridges are sometimes more expensive than longer span bridges. Average cost by species indicates a clear trend of western softwood species costing less than eastern hardwoods and southern pine. The data indicates similar average costs when comparing southern pine structures with red oak structures. Structures constructed of red maple also showed an almost identical cost per square foot of superstructure as compared to southern pine, although bridge design varied greatly between the two species.

Several conclusions may be drawn from this data: regions may make a difference in cost of timber bridges, bridge type makes a difference in cost of timber bridges, and, most importantly, species may not be as important as once thought in the cost of timber bridges. Additional analysis of the data is needed to conclusively answer questions as to relationships between costs, region, species, and bridge type.

Questions concerning this report should be referred to Chuck Coole, Program Analyst, USDA Forest Service, Timber Bridge Information Resource Center, 180 Canfield Street, Morgantown, WV 26505: Phone: 304-285-1591.

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National Cost Study of Timber Bridges

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Abstract

A study is underway to determine the initial cost of timber bridges compared to those of steel, concrete, and prestressed concrete bridges. This report discusses the early results of the timber bridge portion of the data set. To this end, timber bridge owners, as identified in the June 1994 National Bridge Inventory (NBI), were sent a specially designed questionnaire to survey cost information on timber bridges under their ownership. In order to establish a comparative basis, timber bridges were selected under the requirements that they be built no earlier than 1980 and be load rated according to American Association of State Highway and Transportation Officials (AASHTO) specifications. No private or government demonstration bridges were included in this study, Given these requirements, 1604 timber bridges were identified as survey bridges. This paper summarizes the analysis of data collected on the cost of timber bridge superstructures throughout the country. The results of such analysis suggest that unit costs were highest for both the longest and shortest bridges considered and tend to increase with higher load ratings. Additionally, it was noted that questionnaire responses were lower than expected for shorter, narrower bridges that were designed to carry lighter loads.

Keywords: Timberbridge(s) superstructure cost.

Purpose And Background

According to Smith and Bush (1994), there are approximately 200,000 deficient bridges throughout the country with a projected replacement cost of $84 billion. As Wolchuk (1988) indicates, concrete decks account "for about two-thirds of the deficiency cases." In the face of such staggering figures, there has been a renewed interest in the prospect of timber bridge utilization.

Throughout much of the 19th century, timber structures accounted for the majority of the bridges and railroad trestles in the United States. These were simple structures constructed of sawn lumber. Many timber bridges of the period even lacked preservative treatments that would allow them to withstand exposure to moisture and decay. Additionally, the older timber bridges were often crudely designed with little or no input by engineers. For example, it was not until 1840 that a complete stress analysis of a timber bridge design was included with the bridge designer's patent. In the 20th century, timber bridges were first replaced by steel. Steel competed with timber as a bridge construction material on a first-cost basis by 1910 and came to dominate the bridge market by 1930 (Ritter 1990).

The failure of older, primitive timber bridges and their eventual replacement by newer steel and, later, concrete designs is the likely source of a general perception held by some today that timber bridges are of inferior quality. Over time, however, the limitations of steel, concrete, and prestressed concrete have become apparent and range from susceptibility to corrosion to costly maintenance and replacement.

Beginning in the mid-1940s engineers began to reconsider timber for bridge construction. The development of such techniques as glue- and, later, stress- lamination demonstrated the strength of timber as a construction material and led to a renewed interest in timber bridge utilization (Ritter 1990). The rationale for this interest is three-fold. First, timber offers a potentially low-cost

alternative to modern bridge construction materials such as steel, concrete, and prestressed concrete. Second, recent research indicates that timber bridges may be more durable than those constructed from other materials, particularly in cold climates where salts and other de-icing agents are frequently used. Third, it is hoped that the creation of a viable timber bridge market will encourage economic growth in rural areas with under-utilized timber resources.

The Timber Bridge Initiative program is a formal representation of the resurgent interest in timber bridges. Since its introduction, the program has resulted in over $17 million in congressional funding for bridge research, construction, and promotion (Smith and Bush 1994). As a result of such efforts, modern timber bridge designs, construction techniques, and preservative treatments have made it possible to better utilize local wood species that were previously viewed as undesirable.

Despite this interest, however, little is actually known regarding the initial and life-cycle costs of timber bridges relative to those of competing bridge construction materials. Data limitations have restricted the number and scope of studies in this area. Of the studies that have been conducted, most have been limited to a particular geographic region and a relatively small number of bridges. The latter shortcoming has made tests for statistical significance of cost comparison results virtually impossible. Accordingly, it is difficult to convince transportation agency officials that timber bridges are a viable alternative.

The main objective of this study is to determine the cost characteristics of timber bridges and to present them in an easily-comprehensible format to a wide-range of transportation officials. A secondary aim of this study is to identify and quantify the components of bridge superstructure costs.

Background Literature

Several studies have been conducted on the subject of timber bridge economics in recent years. The stated impetus for the bulk of these projects is the need for cost-effective alternatives to traditional infrastructure components of the national highway system.

In the majority of such work, five characteristics predominate. First, the studies tend to focus only on bridge superstructure, *i.e.* the deck, beams, girders, wearing surface, and periphery such as guard rails. Researchers have chosen this approach to avoid complications involved in attempting to identify and uniformly evaluate the cost components of bridge substructures. Specifically, they have found that substructure construction costs are more likely to vary with respect to site due to differences in geological formations, soil types, and other site-specific characteristics that are difficult to quantify. For example, Behr *et al.* (1990) consider superstructures only in their cost comparison of several bridge designs in the New England area. Similarly, Verna *et al.* (1984) limit their treatment of bridge replacement costs to major superstructure components, *e.g.* the deck and structural members.

Second, past research efforts tend to deal with initial as opposed to life-cycle costs. There is some question as to the need for initial cost information, since such costs only represent a portion of bridge costs over time. According to Wolchuk (1988), the immense task of rebuilding the nation's bridges "should call for planning based on sound economic principles, with due consideration of the total cost of structures over their entire projected service lives." Researchers cite difficulty in obtaining accurate, complete maintenance and replacement cost figures as the main reason for omitting life-cycle cost analyses.

A third characteristic of timber bridge cost research is that studies tend to be area-specific, *i.e.* they are limited to a particular state or geographical region. For example, the 1990 "Cost Comparison of Timber, Steel, and Prestressed Concrete Bridges" by Behr *et al.* applies only to potential bridge projects in New England and, thus, its results may not be wholly applicable in other regions. A study by Sarsley (1990) considers a single prototype stress-laminated timber bridge in the State of Connecticut. Studies by Verna *et al.* (1984) and Hill and Shirole (1984) deal with the similarly small geographic areas of the States of Pennsylvania and Minnesota, respectively. This limitation makes it difficult to apply the conclusions of past research to current bridge project proposals.

Fourth, some past research efforts rely on hypothetical as opposed to historical cost information. This is true of cost studies by both Behr *et al.* (1990) and Verna *et al.* (1984). In the former study, cost information was obtained by supplying participating contractors with a bridge design and asking each to provide an estimated bid. In the latter research effort, bids were supplied for competing deck replacement materials and designs. Despite its appeal from a data collection standpoint, this approach does not allow for the possibility of cost overruns and, thus, may serve to skew cost comparison results.

A final, critical characteristic of past research is a lack of statistical sophistication necessary to make valid comparisons between the costs of various bridge projects. As Behr *et al.* note in regard to their own study, this problem stems from the consideration of sample sizes that are too small for the application of any meaningful tests of statistical significance.

The results of past studies are varied and, at times, contradictory. Studies by Behr *et al.* (1990), Verna *et al.* (1984), and Hill and Shirole (1984) suggest that timber bridges are cost competitive with bridges composed of other materials for certain applications or within limited specifications, *e.g.* length, load-rating, *etc.* However, there appears to be no consistent pattern between studies as to the limits or characteristics of timber bridge feasibility. For example, Behr *et al.* (1990) find a positively-sloped relationship between unit cost and structure length, while Verna *et al.* (1984) find an inverse correlation between the two. A need exists for further clarification of timber bridge cost relationships.

Methods

It is the intent of this project to overcome some, but not all of the limitations of past research efforts. The study is unique in many regards, not the least of which is its sheer magnitude. The project covers bridges throughout the United States and from a variety of locations. This comprehensive approach should help to control for variable factors such as climate, labor, and transportation costs and will allow for the establishment of practical, generally-applicable results. Furthermore, the project utilized cost information from completed, non-demonstration bridge projects as opposed to cost estimates from hypothetical bridge sites. Finally, a detailed comprehensive data set was generated and statistical tests completed in order to assess the validity of study conclusions.

Some of the limitations of past research apply to the current study as well. Due to the potential variability of substructure costs with respect to site specific conditions, *e.g.* differences in soil composition and terrain between bridge sites, the study focuses primarily on superstructure costs. In addition, only initial costs are evaluated, since there are inadequate data for a meaningful comparison of life-cycle costs. This is probably due, in part, to the fact that modern-design bridges have needed little repair. Long-term cost data are often only available for bridges of antiquated design and construction.

Project methodology consists of two major tasks, data collection and compilation, and statistical analysis. The following two sections explain the project methodology in greater detail.

Data Collection and Compilation

Timber bridge selection was based on the U. S. Department of Transportation Federal Highway Administration National Bridge Inventory (NBI), a database that contains structural and inspection statistics for the nation's 668,433 bridges and other highway structures greater than 20 feet in length (FHWA 1988). The NBI includes such information as bridge identification or structure number, location, ownership, length, width in feet, the number of lanes, and the year built, as well as inspection information regarding each bridge's structural condition. The NBI database is updated continually and is available on magnetic computer tape.

For this project, a copy of the June 1994 NBI was obtained in magnetic tape form and downloaded onto a computer. Due to an error in storage or data entry, bridge information for the State of Texas was improperly aligned and had to be parsed into a usable format by software developed specifically for this task. Three bridge records were deemed beyond repair and lost during this procedure. At this point, unnecessary fields were removed from the database in an effort to reduce file size. Of 116 total NBI database items, only 36 were retained. Among these 36 were the state, owner, county, place, structure number, location, feature intersected, year constructed, number of lanes on,

design load, deck width, and structure length. Some items, *e.g.* structure number, owner, county, *etc.*, were maintained to identify each bridge and assign it with its owner-agency. Others, such as structure length, deck width, year constructed, *etc.*, provide data on vital bridge characteristics and were selected for purposes of comparison and statistical analysis. The resulting database was then loaded into Microsoft Access version 2.0 to aid in data storage and management.

Once in this format, the database was again filtered to eliminate records, *i.e.* bridges, that fell outside the scope of the current project. Records were maintained only for load rated bridges constructed during or after 1980. This year was selected because many modern timber bridge innovations such as glue and stress lamination were unavailable or unknown in prior years. The database was also filtered to remove records representing pedestrian and railroad bridges. The resulting database contained 68,500 records.

At this point, a sub database was developed containing only timber bridges and meeting the above filtering requirements. The resulting database contained 1604 timber bridge records, each containing identification, design, and structure information as outlined above.

Here, efforts were directed towards obtaining cost, bid, and supplier information not found in the NBI. Using ownership information and state and county code numbers supplied in the NBI database, the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (FHWA 1988), and the *Codes for Named Populated Place, Primary County Divisions, and Other Locational Entities of the United States and Outlying Areas* (national Bureau of Standards 1987), an owner-agency was identified for each bridge record.

A form letter was sent to Department of Transportation or other central transportation office in each state that requested a list of names and addresses for county or transportation district officials and their agencies. The resulting lists were then merged to provide a contact person and an address for each timber bridge record in the project database. Similar lists were obtained for National Park Service and USDA Forest Service offices and the appropriate agency contacts were linked with each bridge.

Once a contact and address were established for each timber bridge record, a detailed two-page questionnaire was developed to obtain cost, bid, and contractor/supplier information from each respective bridge owner-agency. Additionally, owner-agency officials were asked to verify the accuracy of NBI identification information, location, length, and the year constructed for each bridge. These questionnaires were then mailed along with a return envelope to respective bridge owner-agencies across the country.

Upon receipt of completed timber bridge questionnaires, the new bridge data obtained from owner-agency officials were entered into the database by record. Those records containing superstructure cost information or a final contractor or bid worksheet were flagged for later identification as “valid’ or usable bridges. The newly obtained information was then used for a preliminary timber bridge cost analysis. Questionnaire response information was compared to that of the total number of timber bridges to ascertain whether the chosen sample exhibited the same characteristics as the population as a whole.

Data Analysis

Upon receipt of questionnaires from transportation owner-agencies, the characteristics of the bridges for which responses had been returned were compared with those of the entire population of timber bridges. The former group of bridges is referred to here as the *response* set, while the latter, the *total* population. Here, it was assumed that the return of bridge questionnaires is random if the number of bridges from the response set with a given characteristic, for example an HS 20 load rating, is proportional to the total population of bridges possessing that same trait. This analysis was completed for bridge structure length, deck width, number of lanes, load rating, and year constructed. Similarly, data from bridges for which usable superstructure cost information had been returned was compared to those of the total population. This smaller subset of bridges from the response set with complete cost information is referred to here as the *valid* set. The results of these comparisons are summarized graphically below. A chi-square test was also performed to determine whether the observed number of bridges

for each characteristic differed significantly from the expected number for that characteristic. The expected number for a given category was based on the proportion of the number of bridges from that category to the total population. The resulting chi-square value is then compared to an accumulative distribution table to assess whether noted differences are statistically significant.

Structure Length - Of the 1604 timber bridges considered, 1591 possessed complete structure length information, and 553 of the 556 response set had usable length data. Of the response set, 223 were considered valid, since they included both

Figure 1 - Number of Timber Bridges versus Bridge Length.

complete structure length and superstructure cost information. As seen in Figure 1, the number of bridges from the response and valid sets appear to be roughly proportional to the total number of timber bridges from the original database for each ten-foot interval of length. Perhaps the single largest discrepancy is the lower ratio of response and valid set bridges between twenty to thirty feet in length to the total number of bridges in that range. The results of chi-square analysis suggest, however, that differences between the response and valid set bridges and the total population are significant, with chi-square values of 39.34 (at just above the 2.5 percent level of significance) and 71.47(at the 0.5 level of significance), respectively with twenty-four degrees of freedom. This is probably due to an under representation of bridges in the twenty to thirty foot category and an overrepresentation of bridges in the ninety to one hundred foot category. Removing these categories reduces the chi-square values to 36.30 for response set bridges and 59.08 for valid set bridges, with 22 degrees of freedom. The under-representation of bridges shorter than thirty feet may stem from a lack of reliable record-keeping for short, relatively inexpensive timber bridges in transportation agency offices. This is supported by questionnaire comments and is supported below as other bridge characteristics are considered.

Width - Of the initial 1604 timber bridges, 1570 possessed complete width information, as did 556 of 556 response bridges. All 223 bridges of the valid set had usable width information.

Figure 2 - Number of Timber Bridges versus Bridge Width.

While the response and valid set characteristics, as seen above in Figure 2, appear to match those of the total population, the results of a chi-square test suggest that the differences between observed and expected values are statistically significant at the 0.5 percent level of significance, with chi-square values of 77.03 and 56.61, respectively, and 12 degrees of freedom. Again, a large discrepancy exists for the range of values between twenty-five and thirty-five feet in width. When this range is removed, the values drop to 30.99 for the response set and 17.96 for the valid set, with ten degrees of freedom, making the difference between the valid set and the total population significant only at the ten percent level. Again, one finds that the smaller or, in this case, narrower bridges tend to be under-represented by the valid and response sets, with one exception noted in the latter. Likewise, wider bridges tend to be over-represented. As mentioned

above, this may stem from a deficiency in record-keeping for smaller, less-sophisticated bridges.

Number of Lanes - Of the total data set, 1571 bridges had information regarding the number of lanes that cross each bridge. Of the 556 response bridges, 551 listed the number of lanes. 222 of these included complete cost data as well. Only one and two lane bridges are present in the population. At first glance, the lane characteristics from the response and valid sets appears to follow those of the total population, as seen below in Figure 3.

Figure 3 - Number of Timber Bridges versus the Number of Lanes on Bridge.

As experienced above, however, a chi-square test reveals that the differences between the population set and the response and valid sets are statistically significant at below the ten percent and 2.5 percent levels, respectively, with one degree of freedom. Again, most of the discrepancy appears to be the result of an under-representation of smaller, single-lane bridges.

Load Rating - Load ratings were absent for several bridges in the initial population. Only 1517 of 1604 bridges had complete data for this category. Similarly, only 527 of 556 response bridges listed a load rating, and 216 of these included complete cost information. Figure 4 illustrates that the above trend towards under-representation for smaller bridges was again apparent for load rating. This observation was supported by the results of a chi-square test which suggest that the differences between the total

Figure 4 - Number of Timber Bridges versus Load Rating.

population and the response and valid sets are statistically significant at the 0.5 percent level, with chi-square values of 41.89 and 31.30, respectively (with seven degrees of freedom). Here, it was noted that H10 bridges, the lightest load classification in the total data set and probably that with the smallest bridges, were under-represented in the response and valid sets. When this category was removed, chi-square values improved to 6.50 for response bridges and 10.82 for valid bridges, with six degrees of freedom. Differences between the valid set and the total population were only statistically significant at the ten percent level. The response set was not significantly different from the population.

Location - The initial population of 1604 timber bridges was spread across the country among forty-seven of the fifty states. The response set demonstrated similar characteristics as the total population, as did the group of response bridges for which cost information was available. However, a chi-square test indicates that the differences between the total population and the response and valid sets were statistically significant at the 0.5 percent level. Based by comments provided on bridge questionnaires by owner-agency transportation officials, the reasons for failing to provide cost information for bridges ranged from inadequate record-keeping systems, to short-staffed or busy offices.

Year Constructed - Of the 1604 bridges in the initial data set, 1590 included information regarding the year each bridge was constructed, as did 555 of 556 response set bridges. Of the response set, 222 possessed complete cost

information. As noted in Figure 5, no perceptible difference exists between the characteristics of the total population and those of the response and valid sets.

Figure 5 - Number of Timber Bridges versus Year Constructed.

A chi-square test supports this finding and suggests that differences between the population and the response and valid sets are not statistically significant, with chi-square values of 10.01 and 11.43, respectively (with thirteen degrees of freedom). It is interesting here to note that a category that is not size-related turns up no significant difference between the population and the response and valid sets. This observation indirectly supports the assumption that questionnaire response is a random process, when smaller bridges are excluded.

Results

Unit Cost versus Structure Length

Of the studies conducted on timber bridge costs, most focus on the relationship between bridge length and superstructure cost per square foot. It, thus, seems appropriate to address this factor first.

After indexing the unit cost figures to 1982 dollars using a construction sector producer price index (PPI) from the *Economic Report of the President* (Council of Economic Advisers 1996), the relationship between superstructure unit cost and structure length takes on a somewhat parabolic shape with high variability (see Figure 6). Thus it appears unit cost is at its highest at both short bridge lengths, roughly between 20 and 50 feet, and longer lengths above 150 feet.

Figure 6 - Unit Cost (indexed) versus Structure Length.

Higher costs at lower structure lengths may exist because of fixed costs, such as that for equipment rental, that are required regardless of length. These costs may only begin to be recovered on larger projects. In addition, there may be more variability of design at shorter lengths that leads to higher costs when untried bridges are constructed. At the other end of the spectrum, higher costs at relatively high structure length are probably associated with the increase in construction material volume necessary to maintain a given carrying load for longer bridges. In addition, a lack of standardization implicit in rare, longer lengths may serve to increase costs as untried designs and construction practices are advanced.

Unit Cost versus Load Rating

Because bridges built to carry higher loads may require more of the primary construction material per square foot of deck, it is likely that load rating is an important cost factor. There are seven different load ratings represented in the total data set: H10, H15, HS15, H20, HS20, HS20 + mod, and HS25. All of these load classifications were present in the valid data set, with the exception of HS15. In addition, there were few data points available for the H10, H15, and HS25 load ratings. This shortage follows the general trend in the population as a whole, with the majority of bridges falling in the H20, HS20, and H20 + mod classifications.

The number of observations available and mean unit cost for each load rating are listed below in

Table 1. Indexed unit costs for each load rating were sorted in ascending order and divided into

Table 1 - Number of Observations Available and Mean Unit Cost by Load Rating.

Load Rating	Number of Observations	Mean
H10	4	52.41
H15	8	13.51
HS15	1	46.92
H20	9	21.67
HS20	154	30.34
HS20 + Mod	37	40.44
HS25	3	37.83
Total:	216	31.67

quartiles. These quartiles were then used to develop the box plot shown in Figure 7. Because of the limited number of data points for the HS15 and HS25 load ratings, these categories were not included. The boxes in Figure 7 represent the

Figure 7 - Unit Cost (indexed) versus Load Rating.

middle 50 percent of the data for each load rating. The whiskers that extend from the boxes show the range of cost data for each category. The bold, horizontal lines show the mean for each load rating.

An initial look at the box plot suggests an upward trend in unit cost as load ratings increase. It should be noted, however, that load ratings H10, H15, and H20 are each based on between four and nine data points. In addition, an unusually high unit cost exists for one bridge in the H10 category, which dramatically alters the mean for that load rating. The elimination of this outlying observation lowers the mean unit cost for the H10 load rating to $20.05.

Unit Cost versus Region

Adopting regional boundaries used in a USDA Timber Bridge Information Resource Center (TBIRC) report (TBIRC 1993), superstructure unit cost was again evaluated, this time by region. One region, Northeast, was actually a hybrid of two TBIRC regions and includes states from along the Atlantic coast from Maryland north. The number of observations available and the mean unit cost for each region are shown in Table 2. Within the valid

Table 2 - Number of Observations Available and Mean Unit Cost by Region.

Region	Number of Observations	Mean
Southeast	5	$36.96
Northeast	54	$42.21
Northern	2	$28.79
Northwest	7	$30.06
Midwest	148	$27.81
Central	4	$41.40
Intermountain	2	$27.22
Total:	222	$31.84

set, the two largest regions were the Northeast with 54 data points and the Midwest with 148 data points. The remaining regions all had between two and seven valid bridges each within their boundaries. The mean unit cost for the Northeast region was $42.21 while that for the Midwest was $27.81.

Like those for the load ratings above, unit costs for each region were sorted in ascending order and divided into quartiles. These quartiles were then incorporated in the box plot in Figure 8. The Northern and Intermountain regions were not included due to data limitations. The means for each region are again represented by the bold, horizontal lines, while the boxes include the middle 50 percent of the observations for each category. The whiskers represent the range for each region. The boxes in Figure 8 roughly follow the regional means reported above, with the exception of that

Figure 8 — Unit Cost (indexed) versus Region.

for the Southeast region. The mean for this region appears to have been skewed upward by the same unusually high unit cost noted earlier for the H10 load rating. Removing this outlying data point reduces the mean unit cost for the Southeast region from $36.96 to $8.82.

Concluding Remarks

The dual purposes of this paper were to describe the cost characteristics of timber bridge superstructures throughout the country and to identify the factors that comprise bridge costs. Based upon the above analyses, the following generalizations can be made. There appears to be a roughly parabolic relationship between the unit cost of timber bridge superstructures and structure length in feet, with higher costs at both the shortest and longest lengths. Additionally, there is a somewhat positive relationship between indexed unit cost and load rating, Finally, despite differences in mean unit costs among regions, there is no identifiable trend associated with this geographical breakdown. These findings are limited, however, due to the wide variability in the available data and, in the case of load rating and region, by the limited number of data points for some categories. This circumstance may relate to one of three causes: unspecified cost factors, a lack of standardization in timber bridge construction, or a realization of the market niche for timber bridges.

First, it is likely that still other cost components play an important role in determining bridge costs. For example, such factors may be site-specific cost determinants relating to site preparation and material and equipment transportation. If this is indeed the case, a similar characteristic of an unexplained variation in bridge cost for a given length should be observable for bridges constructed of other materials. Future research efforts will focus on this possibility and will also include a cost comparison between material types. The acquisition of cost data for bridges of other material types will enable the use of a paired *t* test to analyze variability in bridge cost by type of construction material, even if both sets of data have high variability in unit cost versus other factors.

An additional explanation for cost variability may relate a lack of standardization in timber bridge design and construction resulting in ad hoc assembly practices by various transportation agencies. If so, the implementation of standardization efforts by Lee, Ritter, and Triche (1995) and others will likely lead to a reduction in timber bridge costs. Again, future research efforts on other bridge construction materials will help assess the validity of this assumption, since standards are already in place for other popular bridge materials.

Finally, there exists the possibility that timber bridges have found their market niche in the form of primarily small-crossing, rural, and, most importantly, non-traditional applications that lead to a wide-range of construction practices and design concepts unique to timber alone. The limited number of timber bridges above 100 feet in length and absolute lack of bridges of more than two lanes in width support this notion. Whatever the case, it seems that future research is necessary to help identify the observed variation in timber bridge costs.

Given the size and scope of this data set, it is easy to understand the limitations of past studies to explain the nature of timber bridge costs. While the cost components considered above appear relevant, there remains a largely unexplained variability in superstructure cost. This finding heightens the need for an examination of cost data for timber bridges versus those constructed of steel, concrete, and prestressed concrete. Only after a thorough comparison between material types will the large-scale feasibility of timber bridges be recognized.

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Engineered Wood Products for Transportation Structures-An Overview of the Obstacles and Opportunities

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Abstract

The opportunities for engineered wood products in the transportation field are numerous. Innovative bridges, geotechnical structures, poles, and sight/sound barriers are some of the existing opportunities. In all probability, more opportunities will emerge in the future as engineered wood products become more versatile and durable. Unfortunately, some very real obstacles exist as well. Engineer's perception of wood as a second-class material, poorly executed designs and construction, and intense competition are only a few of the obstacles facing the industry. A workshop of industry, government and research experts from the field of engineered wood products was held in Morgantown, West Virginia in December 1994 to discuss the opportunities and obstacles facing the industry. The record created from the workshop provides the basis for this paper.

Keywords: Engineered wood products, bridges, transportation structures

Background

The use of wood for transportation structures is as old as human history. Certainly our first bridges were logs felled across streams. Readily available throughout most of the world and easily worked, wood has served mankind well for thousands of years.

In 18th and early 19th century America, our rural economy was heavily dependent on wood for fuel, building, furniture, machinery, and probably thousands of other uses. Even then transportation was an essential portion of the economy, and wood was the material of choice for most transportation structures. Some of our early wood bridges still stand today as a testament to the durability and strength of wood structures when designed and protected well. Later in our country's history, railroads were the dominant force in the economy- wood products such as crossties, poles, and bridge timbers allowed the railroads to transport heavy loads economically and quickly across the country.

The development of cast iron, steel, and eventually prestressed concrete in the late 19th and 20th centuries made wood products less desirable for some transportation structures. In particular, bridges built of these new materials were longer, stronger, and more durable than traditional wood bridges. In only a few decades the dominance of these newer materials was virtually complete. Not only was the use of wood for transportation structures decreased, but the skills required to use wood effectively diminished also. Engineers and constructors were trained to work primarily with concrete and steel and the few wood

bridges that were built were often of poor design and quality.

Many less visible markets for wood products in the transportation field were retained, however. Wood railroad crossties were made much more durable with the development of preservative treating techniques. Poles for the transport of electrical power and communication lines sprung up in every comer of the country. Timber piles continue to be used extensively for the support of bridge abutments and piers. Formwork and falsework are necessary for the construction of almost all transportation structures and use large quantities of wood products.

A Renaissance of Wood?

With concrete, steel and a variety of structural plastic materials now available for the construction of transportation structures, why would today's engineers choose wood? The answer is in both the old and the "new" properties of wood. Wood, as always, is light-weight, strong, and easily worked-properties which attract an engineer's attention. Wood also is in plentiful supply and is likely to remain so. Our forests are remarkably resilient and, despite public opinion to the contrary, "forest growth nationally has exceeded harvest by 37% and the volume of forest growth was 350% greater than it had been in 1920". (MacCleery, 1995) The "new" properties of wood are a result of years of technological advances to meet market needs. Newly developed wood-based composites and engineered wood products have strength, stiffness, durability, and reliability properties vastly superior to their sawn-lumber ancestors. The engineer's choice of materials is no longer as clear-cut as it once was.

The Workshops

In an attempt to expedite the use of new wood products engineered specifically for transportation structures, a workshop for interested engineers, manufacturers, researchers, and policy-makers was sponsored by the USDA Forest Service, State and Private Forestry. The Constructed Facilities Center of West Virginia University administered and hosted the *Engineered Wood for Transportation Workshop* in December, 1994. Workshop participants formed several working groups and were asked to generate lists of opportunities for and obstacles to the use of engineered wood for various types of transportation structures. The discussions of that meeting were recorded to generate a document, the *Engineered Wood for Transportation Structures Workshop Record* (CFC, 1996). In order to organize the experts into productive groups for discussion, the meeting was divided into ten workshops. Each workshop was assigned a topic and the experts were assigned to the workshops where their expertise would be most useful. On the first day of the meeting, workshops were held to discuss the opportunities for and obstacles facing engineered wood in five applications-bridge superstructures; guide rails, posts and poles; noise/sight barriers and formwork/falsework; retaining walls, piles, culverts, and drop inlets; and repair and rehabilitation. On the second day, the experts were asked to form five different groups and confine their discussions to the issues related to material characterization; evaluation and design; construction; marketing and technology transfer: and environmental concerns.

Each of the workshops' leaders maintained a record of the group discussions and made a brief presentation to the entire meeting to summarize the groups' findings. The following is a synopsis of the group leaders' presentations.

Workshop 1- Bridge Superstructures

Opportunities for the expansion of engineered wood are abundant in the area of bridges. The USDA Forest service's Timber Bridge Initiative and the Intermodal Surface Transportation Efficiency Act of 1991 have allowed several states to test the newest wood bridge types. Innovative designs have been given opportunities that they otherwise might never have had. The performance of the new bridges has been mixed - some of the new wood bridges are performing well and are reasonably cost-competitive, but others are not. The TBI has had a sufficient life-span to revise and improve many of the original innovative designs. However, in order for this potentially large market to be fully realized, the quality will have to be more consistent and the costs more competitive.

Other opportunities for the expanded use of engineered wood for bridges include:

- Newly developed engineered wood materials which have improved structural performance may be well-suited to bridge construction
- Improved prefabrication methods for wood bridge components have the potential to reduce costs
- The public retains a preference for wood structures in specific settings
- Stress-laminated ballasted deck bridges for railroad trestle bridges

Obstacles limiting the expanded use of engineered wood for bridges include:

- A history of poorly engineered wood bridges which

has reduced many engineer's confidence in wood
- Environmental concerns resulting from improper creosote treatment
- Lack of comprehensive inspection methods and accurate inspection devices
- Well-established competition from other materials

Workshop 2- Guide Rails, Posts, and Poles

The best properties of wood have long been utilized for posts and poles and, more recently, for guide rail components. These properties include high strength and energy absorbing characteristics. The greatest opportunities for expansion of the market for guide rails, posts, and poles exist in the timber producing areas where costs would be most competitive with other materials. The expanding number of "Greenways" (environmentally sensitive highways) also should be a natural market for engineered wood guide rails, posts and poles.

Other opportunities for guide rails, posts, and poles include:
- Composite posts and poles which combine wood and plastics
- New wood bridge rail designs that have been crash-tested and are now available for general use

Some of the obstacles cited to expanded use of engineered wood for guide rails, posts, and poles include:
- A lack of consistent design guides and design details (such as steel rail to wood rail transitions)
- The large size of wood posts requires larger equipment to drive posts than is required for competing materials
- A history of widely fluctuating prices for wood posts and poles

Workshop 3 & 6- Noise/Sight Barriers and Formwork/Falsework

Potentially large markets exist for wood visual and acoustic highway screens. Recycled materials, combined with wood products, could be used for many of the components which form noise and sight barriers. The formwork/falsework field is currently the largest application area of engineered wood in the transportation market. The newly adopted LRFD code for formwork and falsework will bring the design process to modem standards and could increase the use of engineered wood products for this application.

Other opportunities for noise/sight barriers and formwork/falsework include:
- New wood-based composite materials like cement bonded wood
- "Permanently" marked form materials that can be re-used more often than current products
- Recently completed FHWA publications for construction and design of bridge temporary works

Some of the obstacles cited to expanded use of noise/sight barriers and formwork/falsework include:
- A history of poorly designed noise and sight barriers has reduced engineers confidence in wood
- Proprietary forming and falsework systems which have inconsistent design methods

Workshop 4 Retaining Walls, Piles, Culverts, and Drop Inlets

The need for improved preservative methods and consistent quality products were cited as the major obstacles to expanded use of engineered wood for retaining walls, piles, culverts, and drop inlets (or other wood-based geotechnical structures). Opportunities of which the industry could take advantage are in the areas where the properties of wood, such as its non-corrosive nature, exceed those of competing materials. An engineered wood culvert used in highly acidic streams is one example.

Other opportunities for the expanded use of engineered wood for geotechnical structures include:
- Innovative prefabrication techniques resulting from the Timber Bridge Initiative which could decease first costs and reduce construction time
- New wood-based composites are becoming available which may increase the durability of geotechnical wood structures which must survive in very harsh environment

Some of the obstacles cited which limit the use of geotechnical structures include:
- The lack of standard plans
- The difficulty of constructing durable connections and the cost of durable connections
- The perception that wood products cannot be made to be durable in harsh environments

Workshop 5 Repair and Rehabilitation

The basic challenge facing engineers whose responsibility includes repair and rehabilitation of highway structures is to convert the connotation of repair and rehabilitation from the negative to the positive.

Opportunities for repair and rehabilitation using

engineered wood include:

- Hardwood structural glued-laminated wood panels have been tested for bridge deck applications and design guides are now available.
- Repair of transportation structures using wood and plastic composites

Some of the obstacles cited which have limited the expanded use of engineered wood for repair and rehabilitation include:

- Connecting wood or wood-based composites to other materials for repair is often difficult
- Very little design information or standard planning is available

Workshop A- Material Characterization

An accurate and consistent characterization of wood and wood-based composites and the components made of wood or wood based-composites is essential to the expansion of wood markets. The combination of engineered wood with other materials such as fiber reinforced plastics (FRP's) may produce a material possessing exceptionally useful properties. The low cost, light weight, and availability of wood combined with high strength and reliable properties of FRP could be applied to numerous highway structural applications.

Opportunities for expanded use of engineered wood based on advances made in material characterization include:

- In-grade testing of various species and sizes of wood may add material choices for engineers
- Recent advances made to machine stress-rating and stress wave grading techniques should provide more efficient use of wood resources
- More accurate design values for many hardwood species are now available
- Developments made in engineered wood for larger markets (such as housing) can often be converted to applications in transportation

Some of the obstacles which limit the use of engineered wood include:

- The continued difficulty of defining long-term durability and structural performance of wood products
- Inherent properties of wood such as dimensional instability related to moisture changes

Workshop B- Evaluation, Analysis, and Design

Although timber design has kept pace with the advances made by other materials, the need for continued design and evaluation improvements was seen by the workshop group as essential to expanded use of engineered wood. Specifically, the group noted the need for "proof loading" methods to evaluate wood structures, new non-destructive inspection methods, expanded education of engineers, and development of standardized plans for engineered wood products.

Opportunities for expanded use of engineered wood based on advances made in evaluation, analysis and design include:

- Recently completed standard plans for stress-laminated bridges and hardwood glued laminated bridges
- New design specifications in the American Association of State Highway and Transportation Officials *Standard Specifications for Highway Bridges* for stress-laminated wood bridges
- The recent completion of several FHWA crash-tests for wood rail systems

Some of the obstacles related to evaluation, analysis and design which have limited the expanded use of engineered wood for transportation include:

- The lack of standard plans for pedestrian bridges
- The need for more empirical data on the energy adsorbing characteristics of wood so that this feature can be used more advantageously
- The need for more accurate inspection methods which will allow more precise evaluation of in-service wood structures

Workshop C- Construction

Because wood is a light-weight and relatively inexpensive material and most construction crews are familiar with it, the use of wood as a highway structures material is generally accepted. The major obstacle facing the expanded use of engineered wood is the lack of industry-wide quality control standards. The inherent variability of wood quality makes the need for quality control even more necessary for wood products than for other materials.

Opportunities for expanded use of engineered wood based on advances made in construction include:

- Recent innovations in the prefabrication of wood structures as a result of the Timber Bridge Initiative and the ISTEA of 1991
- Many new engineered wood products are now available which reduce construction time and which are of higher quality than traditional sawn lumber

Some of the construction-related obstacles which limit

the use of engineered wood include:

- The lack of contractors with experience in heavy timber construction
- The lack of training aids for constructors specifically for wood transportation structures

Workshop D and E- Economics, Marketing and Technology Transfer

Marketing efforts for engineered wood products exist for many applications. Engineered wood products such as bridges, noise/sight barriers and formwork are already commonplace but could be expanded. A marketing strategy for these and other engineered wood products should be based on a unified approach. Achieving a unified marketing strategy will require a unification of the industry-something that has been impossible to accomplish thus far.

Opportunities for expanded use of engineered wood based on advances made in marketing and technology transfer include:

- Establishment of the USDA Forest Service's National Timber Bridge Information Resource Center
- Studies of current market conditions for wood bridges are now available
- Large increase in the amount of technical support information available

Some of the obstacles which limit the use of engineered wood include:

- The lack of consistent wood terminology hampers clear communication of technical information
- The belief on the part of many engineers in the "second-class" nature of wood for structural applications

Workshop F-Environmental

Like the repair and rehabilitation group, the environmental group found that the most pressing challenge was to change the perception that preservative treated wood products are an environmental liability. Providing preservative treated wood products users as well as the general public with the facts about wood preservatives should be the highest priority for the engineered wood products industry.

Opportunities for expanded use of engineered wood based on advances made environmental include:

- The perception of wood as an "environmentally friendly" material can be used advantageously to promote wood
- Wood has the potential to be re-used or recycled after its original service life

Some of the environmental and recycling obstacles which limit the use of engineered wood include:

- Poor quality designs and specifications for wood structures have frequently led to environmental problems and diminished engineer's confidence
- Composites manufactured of wood and other materials may be more difficult to recycle than conventional wood products

The Next Step

Either explicitly or implicitly, each of the groups recognized the need for actions to be taken to take advantage of opportunities available or to eliminate obstacles. The need for actions requires another step in the process begun by this meeting. At the nearly-unanimous suggestion of the experts, a strategic planning committee comprised of 15 to 20 key persons in the engineered wood community was formed. The strategic planning committee will has as its mission the task of developing an action plan to implement the resolutions of the expert group.

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