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

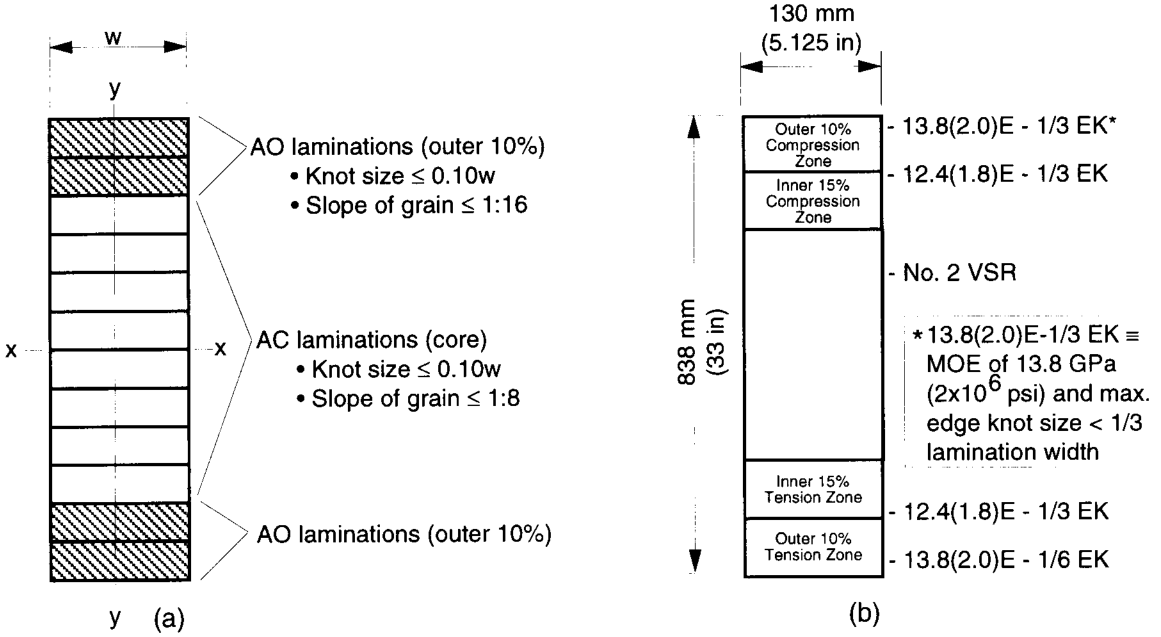

Figure 1--Glulam girder layups for (a) Combination A northern red oak and (b) red maple bridges.

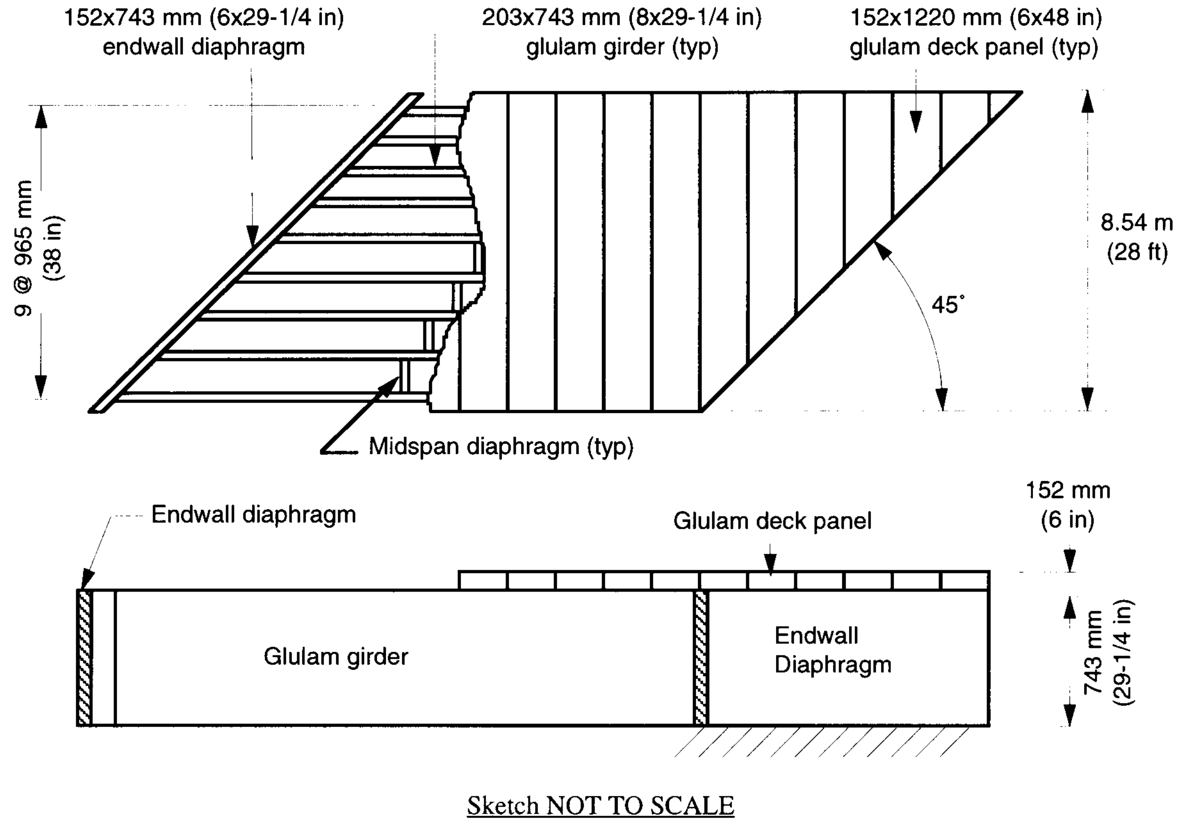

Figure 2--Sketch of superstructure for the northern red oak glulam bridge.

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

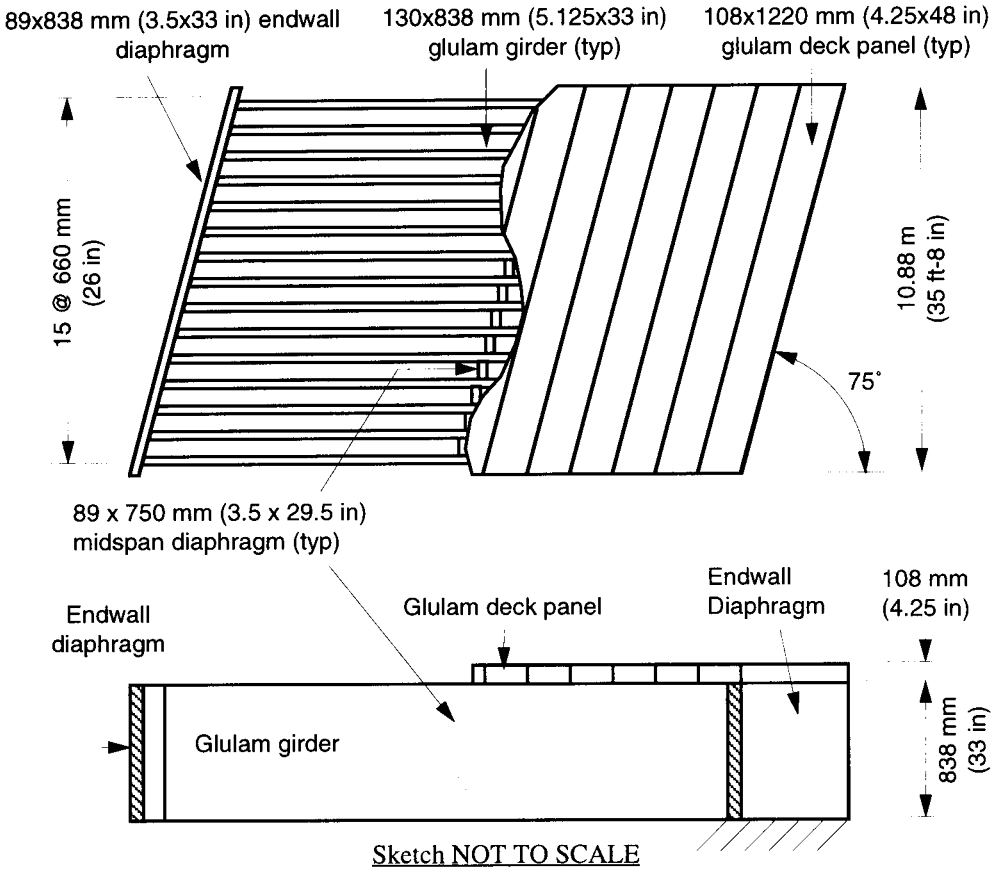

Figure 3--Sketch of the superstructure for the red maple glulam bridge.

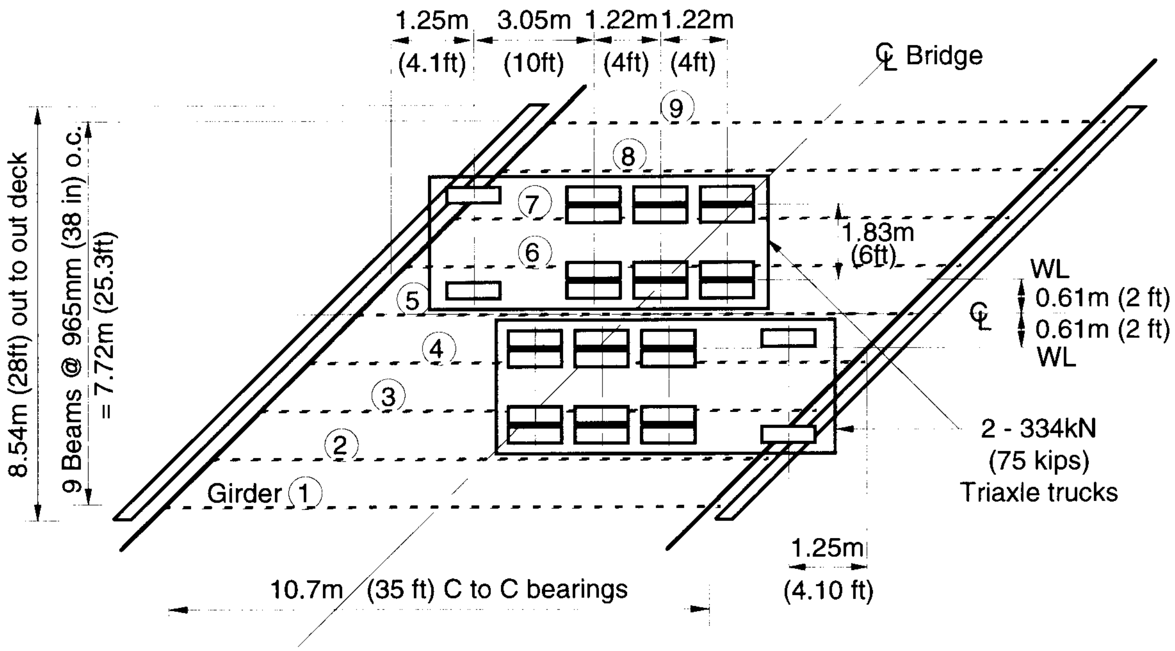

Figure 4--ML-8O vehicle location for maximum deflection of the red oak glulam bridge.

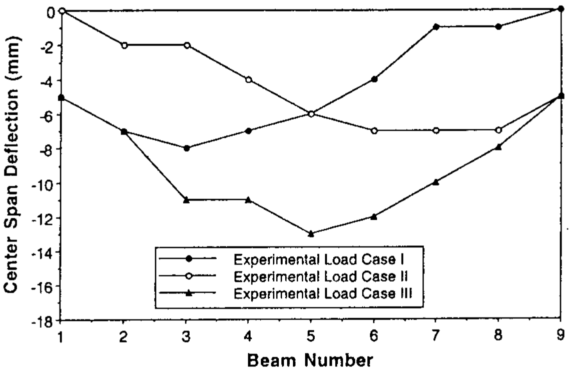

Figure 5--Centerline live load deflections for the red oak bridge.

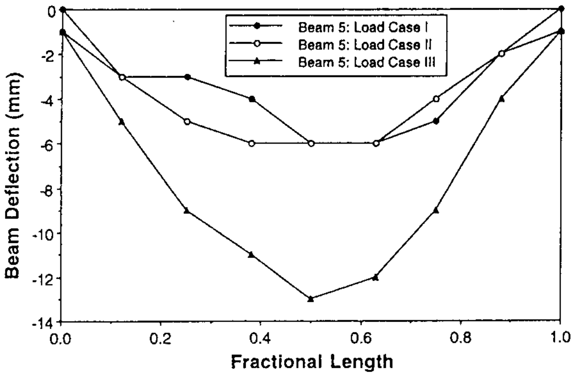

Figure 6--Live load deflection profile for beam 5 of the red oak bridge.

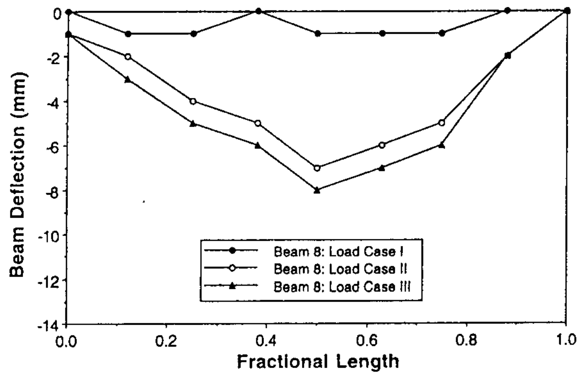

Figure 7. Live load deflection profile for beam 8 of the red oak bridge.

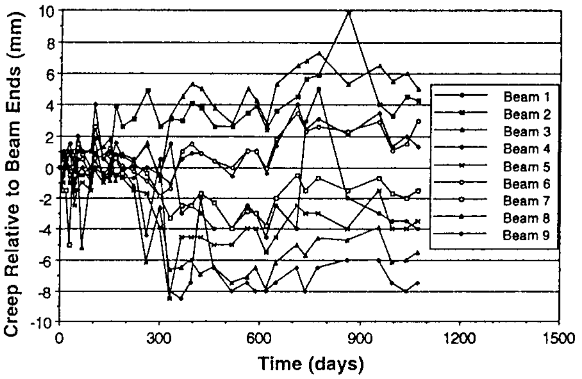

Figure 8--Creep deflection of girders 1 to 9 of the red oak glulam bridge.

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 10^6psi), 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 10^6psi) 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 10^6psi). 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)

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