FINAL REPORT

EVALUATION OF THE PERFORMANCE OF A PRESS-LAM TIMBER BRIDGE

Bridge Performance and Load Test After Five Years

by

Michael M. Sprinkel
Research Scientist

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

Virginia Highway & Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia Department of Highways & Transportation and the University of Virginia)

In Cooperation with the U. S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

June 1982
VHTRC 82-R56

BRIDGE RESEARCH ADVISORY COMMITTEE

J. M. MCCABE, JR., Chairman, Assist. Bridge Engineer, VDH&T

F. L. BURROUGHS, Construction Engineer, VDH&T

C. L. CHAMBERS, Division Structural Engineer, FHWA

H. L. KINNIER, Consultant, VH&TRC

J. G. G. MCGEE, Construction Control Engineer, VDH&T

W. T. MCKEEL, JR., Highway Research Scientist, VH&TRC

M. F. MENEFEE, JR., Structural Steel Engineer, VDH&T

L. L. MISENHEIMER, District Bridge Engineer, VDH&T

R. H. MORECOCK, District Bridge Engineer, VDH&T

F. L. PREWOZNIK, District Bridge Engineer, VDH&T

F. G. SUTHERLAND, Bridge Engineer, VDH&T

D. A. TRAYNHAM, Prestressed Concrete Engineer, VDH&T

SUMMARY

The report describes the results of the load tests and summarizes the collection of data on the dimensional changes and moisture content of the press-lam members and the number and types of vehicles using the bridge. The results of the load tests suggest that the AASHTO load distribution is conservative. Although the rails, wheel guards and posts are exhibiting a high moisture content and areas of delamination, the bridge is structurally sound and in excellent condition after five years of service.

FINAL REPORT

EVALUATION OF THE PERFORMANCE OF A PRESS-LAM TIMBER BRIDGE

Bridge Performance and Load Test After Five Years

by

Michael M. Sprinkel
Research Scientist

INTRODUCTION

This is the third of three reports to be issued during the scheduled five-year evaluation of the performance of the world's first press-lam timber bridge.[1] The report presents the results of the load tests conducted approximately five years after the bridge was constructed and summarizes data on the dimensional changes in the press-lam members, the moisture content of the members, and the number and types of vehicles using the bridge. The two reports issued earlier cover the installation and the first two series of load tests of the bridge.[2,3]

The press-lam timber bridge was installed on Rte. 610 over Little Stoney Creek in Shenandoah County by maintenance forces from the Virginia Department of Highways and Transportation during the week beginning April 18, 1977. The Douglas fir stringers and deck panels and the red oak rails, wheel guards, and posts used in the experimental bridge were fabricated at the U. S. Forest Products Laboratory of Madison, Wisconsin, using their recently developed press-lam process. In this process, thin sheets peeled from a log are glued together to produce lumber with dimensions restricted only by the size of the production line equipment.[4] There are no commercial manufacturers of the press-lam timber at the present; however, the process should become viable sometime in the future as trees of large diameter become scarce.

LOAD TESTS

Test Vehicle

On May 11, 1982, the rear tandem axle of a tractor connected to a trailer loaded with a D16 dozer was used to load test the press-lam bridge for the third time (see Figure 1). Prior to the tests, a crew from the Department's Traffic & Safety Division used scales to determine the load that would be provided by each of the four pairs of wheels on the rear tandem. The wheel spacing and the load produced by each wheel are shown in Figure 2. The scales indicated that, within 3%, each of the pairs of wheels supported 25% of the total load on the tandem. Therefore, for purposes of calculation it was assumed that each pair of wheels produced a load of 9,850 lb. (4,430 kg). The total load produced by the rear tandem axle was 3% less than the load used in the first series of tests on the bridge (May 4, 1977) and 2% less than the load used in the second series (April 26, 1978).

Tests of Interior Stringers

The theoretical flexural stress in the interior stringers produced at midspan by the test vehicle was 6.4% less than the theoretical flexural stress that would be produced by one 32,000 lb. (14,400 kg) AASHTO design axle.[5] Because of the short span length of the press-lam bridge, the AASHTO concentrated loading controls the moment design of the stringers. The tandem load of the test vehicle was 10.5% less than the 44,000 lb. (19,800 kg) tandem load permitted in Virginia. The theoretical midspan interior stringer deflection for the AASHTO loading placed at midspan is 0.378 in. (9.6 mm) as compared to the 0.431 in. (10.9 mm) deflection theoretically produced by the load test vehicle.

The stringer deflections for the 13 load test positions shown in Figure 3 are shown in Table 1. As anticipated, similar deflection data were obtained for each of the following pairs of equivalent loading conditions, 1-13, 2-12, 3-11, and 4-8. The maximum midspan deflection for an interior stringer with the test vehicle positioned in one lane was 0.28 in. (7.0 mm). With the test vehicle centered in both lanes (positions 3 and 11) simultaneously as simulated by position 14, there was a fairly uniform distribution of the load over the interior stringers, with a maximum deflection of 0.28 in. (7.0 mm). If loading position 9 were applied to both lanes simultaneously, stringer 6 would deflect 0.31 in. (8.0 mm) as simulated by loading position 15. Using 0.31 in. (8.0 mm) as the greatest live load deflection for an interior stringer, a distribution factor of S/X, where $X = (4)\ (10.9)/8 = 5.45$, could be applied to the design of the interior stringers. The AASHTO distribution factor of $S/4$ is conservative. The data in Table 2 indicate that five to six stringers support the wheel loads produced by the test vehicle.

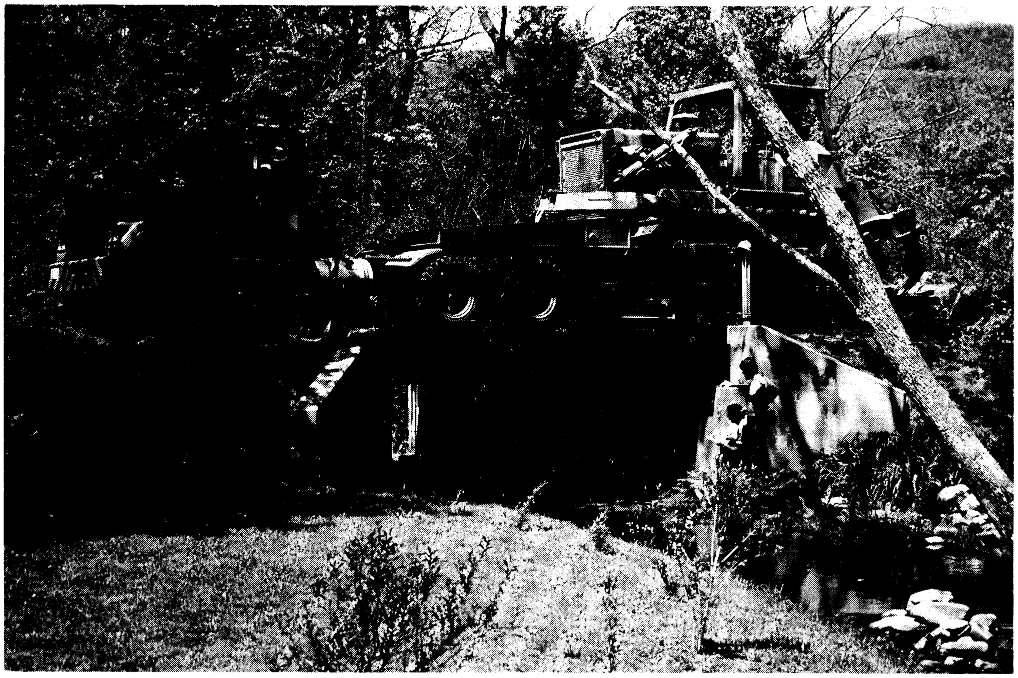


Figure 1. Tractor-trailer used for load test.

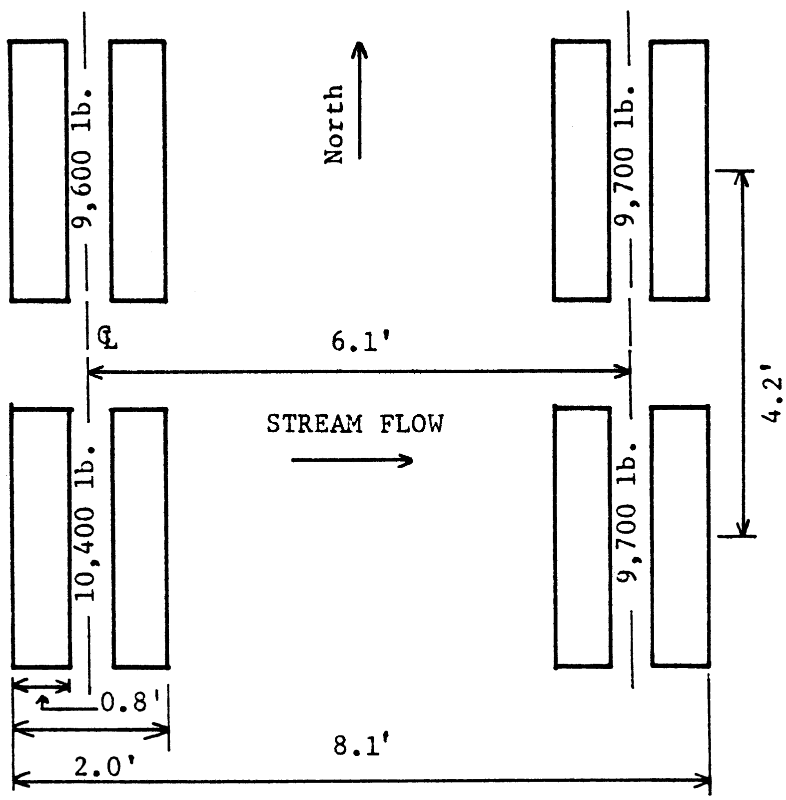
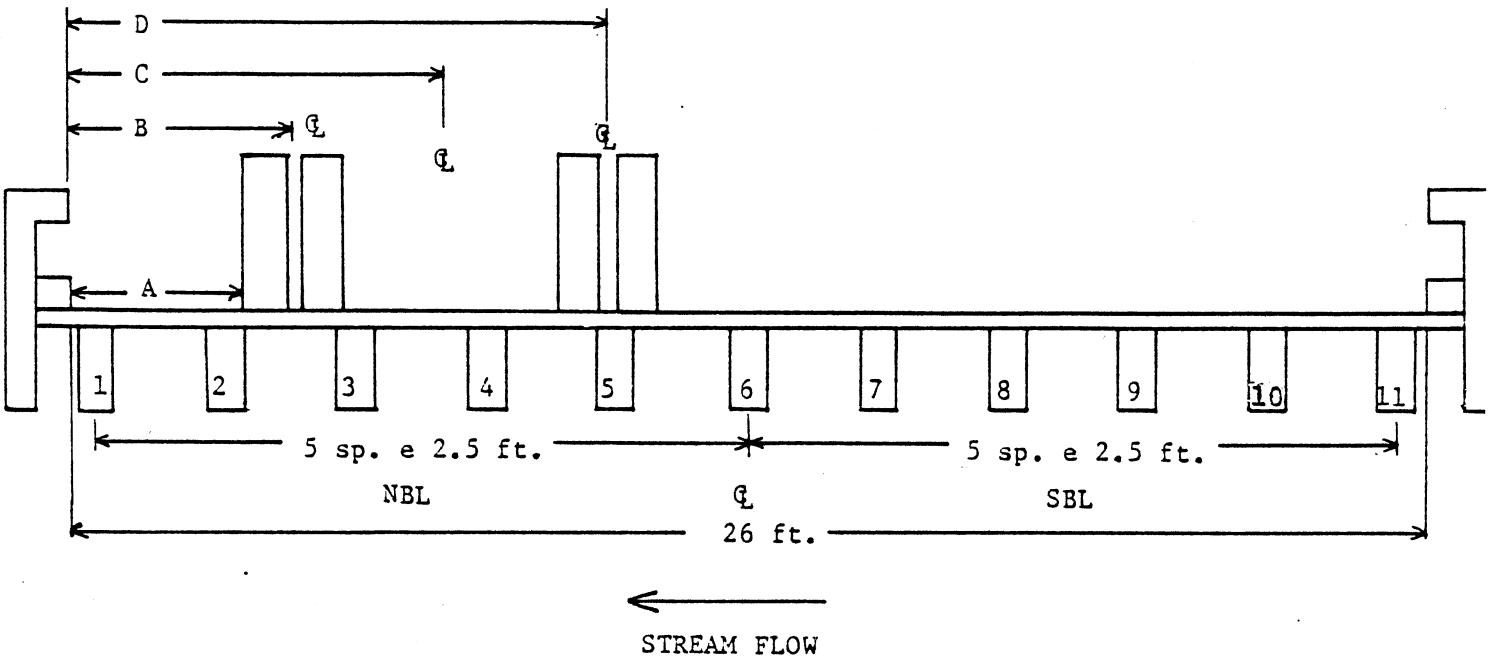


Figure 2. Rear axle dimensions and wheel loads.
1 lb. = 0.45 kg; 1 ft. = 0.30 m.



Loading Position	Wheel Positions, feet Relative to East Wheel Guard A	B	C	D	Condition of Load
1	17.5	18.5	21.6	24.6	Max. load stringer 11
2	15.9	16.9	20.0	23.0	Max. load stringer 10
3	15.2	16.2	19.3	22.3	Center of southbound lane
4	12.2	13.2	16.3	19.3	Midspan stringers 8 and 9; stringer 6
5	10.7	11.7	14.8	17.8	Midspan stringers 5 and 6; stringer 8
6	8.9	9.9	13.0	16.0	Center of bridge
7	8.1	9.1	12.2	15.2	Midspan stringers 4 and 5
8	5.7	6.7	9.8	12.8	Midspan stringers 3 and 4; stringer 6
9	4.4	5.4	8.5	11.5	Stringer 3
10	3.2	4.2	7.3	10.3	Midspan stringers 2 and 3; stringer 5
11	2.7	3.7	6.8	9.8	Center of northbound lane
12	1.9	2.9	6.0	9.0	Stringer 2
13	0.4	1.4	4.5	7.5	Max. load stringer 1

Figure 3. Loading positions used to measure stringer deflections. (1 ft. - 0.30 m).

Table 1

Stringer Deflections at Midspan, mm

Loading Position	Stringer Number 1	2	3	4	5	6	7	8	9	10	11
1	0.0	0.0	0.0	0.0	-1.0	0.0	3.0	5.0	6.0	6.0	8.0
2	0.0	0.0	0.0	0.0	0.0	1.0	4.0	6.0	6.0	7.0	5.0
3	0.0	0.0	0.0	0.0	1.0	2.0	6.0	6.0	7.0	6.0	4.0
4	0.0	0.0	0.0	0.0	3.0	5.0	6.0	6.0	6.0	2.0	1.0
5	-1.0[a]	0.0	0.0	4.0	5.0	5.0	6.0	6.0	4.0	1.0	1.0
6	0.0	0.0	2.0	4.0	6.0	5.0	6.0	4.0	2.0	1.0	1.0
7	-1.0	1.0	3.0	5.0	6.0	5.0	6.0	3.0	2.0	1.0	0.0
8	0.0	2.0	5.0	6.0	5.0	5.0	3.0	1.0	0.0	0.0	-1.0
9	1.0	3.0	6.0	6.0	6.0	4.0	2.0	0.0	0.0	0.0	0.0
10	1.0	5.0	6.0	6.0	6.0	3.0	2.0	0.0	1.0	0.0	-1.0
11	2.0	5.0	6.0	6.0	6.0	2.0	1.0	0.0	0.0	0.0	0.0
12	3.0	6.0	6.0	6.0	5.0	2.0	1.0	0.0	0.0	0.0	0.0
13	6.0	6.0	6.0	6.0	3.0	0.0	1.0	0.0	0.0	0.0	0.0
14[b]	2.0	5.0	6.0	6.0	7.0	4.0	7.0	6.0	7.0	6.0	4.0
15[c]	1.0	3.0	6.0	6.0	8.0	8.0	8.0	6.0	6.0	3.0	1.0

(a) Negative sign implies upward deflection.

(b) Data for position 14 were simulated from data for positions 3 and 11.

(c) Date for position 15 were simulated from data for position 9 assuming vehicle in both lanes simultaneously.

$$1 \text{ mm} = 3.9 \times 10^{-2} \text{ in.}$$

1240

Table 2

Deck Panel Deflections at Midspan
Relative to Adjacent Stringers

Loading Position	Adjacent Stringers	Relative Panel Deflections, mm 1977	1978	1982
2	9, 10	0.15	0.18	0.01
2	10, 11	0.15	0.19	0.10
3	7, 8	0.10	0.10	0.19
3	9, 10	0.18	0.22	0.19
5	5, 6	0.05	0.19	0.10
7	5, 6	0.08	0.17	0.20
8	5, 6	0.10	0.22	-0.20
11	3, 4	0.13	—	0.09
11	4, 5	0.13	0.13	0.06
11	5, 6	0.18	0.10	-0.33
12	3, 4	0.20	—	0.11
12	4, 5	-0.03(a)	0.15	-0.72
12	5, 6	0.20	0.28	0.25

(a) Negative sign means upward deflection relative to adjacent stringers.

1 mm = 3.9×10^{-2} in.

Test of Exterior Stringers

The test vehicle was positioned as close to the curb as possible for load positions 1 and 13. Assuming the deck acts as a simple span between the stringers as required by AASHTO design specifications for these positions, the theoretical load produced on the exterior stringers by each pair of wheels is 6,076 lb. (2,730 kg). The theoretical deflection for two wheel loads of 6,076 lb. (2,730 kg) symmetrically positioned with respect to midspan and 4.2 ft. (1.3 m) apart is 0.425 in. (10.8 mm). The maximum deflection produced in the exterior stringers was 0.31 in. (8.0 mm) for loading position 1. Therefore, the AASHTO design requirement is conservative. A more realistic value for the deflection of the exterior stringer can be obtained by assuming that the deck is fixed over the interior stringer and simply supported over the exterior stringer. Because the AASHTO specification requires that the exterior stringers have the same carrying capacity as the interior stringers, it would not help to change the method of determining the load on the exterior stringer, unless a less conservative distribution factor is used for the interior stringers.

Test of Deck Panels

A dial gage was used to measure the deflections of the center deck panel midway between selected stringers for selected loading positions. The deflections of the deck panel with respect to the adjacent stringers are reported in Table 2. As was the case with the first two series of load tests, the magnitudes of the deflections were extremely small; therefore, no attempt was made to interpret the data. Since one pair of tires on the test vehicle distributed the load over a width of 2.0 ft. (610 mm) and the clear span between two stringers was 2.13 ft. (649 mm), a negligible relative panel deflection would have been expected for a pair of test wheels centered between two stringers.

To determine the ability of the steel dowels to transfer wheel loads between adjacent panels, a series of deck panel deflection readings were recorded with the test vehicle in load positions 11 and 12 and again with the vehicle positioned approximately 1 ft. (305 mm) south of positions 11 and 12. In moving the test vehicle southward 1 ft. (305 mm), the rear wheels of the tandem moved from the center panel to the adjacent panel. The relative panel deflections were small because one pair of tires was 2.0 ft. (610 mm) wide. Because the deflections followed a linear pattern across the panel joints, it can be concluded that the steel dowels provide a satisfactory load transfer between adjacent panels. Taken in their entirety, the load test data suggest that the press-lam deck behaves as a flat plate.

Comparison of Results of Three Load Tests

An effort was made to duplicate the loading conditions for each of the three series of load tests. Table 3 shows the test loads and the theoretical and actual deflections for the three series. The test load was slightly less for each successive test and the deflections were the same or slightly greater. A comparison of the ratios of total deflection to test load for the three series indicates a 9% increase in deflection after one year of service and a 17% increase in deflection after five years of service relative to the initial tests. Obviously the bridge has become more flexible with age, but even after five years of service the measured deflections for the interior and exterior stringers are 25.9% and 26.6% less, respectively, than the theoretical deflections based on the AASHTO load distribution. The increase in live load deflection with age may be related to the increase in the moisture content of the stringers with age, which will be discussed later.

Table 3

Comparison of Results of Three Series of Load Tests

Test date	5/04/1977	4/26/1978	5/11/1982
Total test load in lb.	40,780	40,280	39,400
Test load as a percent of 44,000 lb.	92.7	91.5	89.5
Theoretical max. test moment as a percentage of moment produced by 32,000 lb. AASHTO axle	97.6	96.4	93.6
Theoretical max. midspan deflection in in. (interior stringers)	0.446	0.440	0.431
(exterior stringers)	0.440	0.435	0.425
Measured max. midspan deflection as a percentage of the theoretical (interior stringers)	62.5	63.6	74.1
(exterior stringers)	70.8	71.4	73.4
Interior stringer load distribution factor based on deflection data	5.65	5.60	5.45
Total deflection for 13 loading positions from Table 1 in mm	335.5	362.5	378.0
$\frac{\text{(Total Deflection/Test Load)}}{\text{(Total Deflection/Test Load)}_{5/4/77}}$	1.00	1.09	1.17

1 in. = 25.4 mm;   1 lb. = 0.45 kg

DIMENSIONAL DATA

To determine the dimensional changes in the press-lam members over the five-year period of evaluation, selected members were measured at selected locations immediately following the installation of the bridge and again at three, six, and twelve months, and at annual intervals during the following four years. Calipers and a framing square were used to measure the thickness or depth of the deck panels, stringers, rails, wheel guards, and posts. Metal tacks were installed at selected locations in the deck panels, stringers, and posts, and the distance between each pair of tacks was determined using a dial gage. A steel tape was used to measure the length and width of the deck panels, and a framing square was used to determine the distance between the bottom of the deck panels and the bottom of the stringers.

Dimensional changes based on the data collected during the first year are reported in Table 4. A dial gage was used to measure the distance between a pair of reference tacks to provide the data in Table 4 which follow the reference point numbers, and the other data are based on the average of four or more random measurements made with the indicated piece of equipment. There should be more error in the data taken at random than in the data based on the reference tacks, because the random measurements are based on the exterior condition of the members and may be influenced by surface irregularity and warpage due to delamination. However, there appears to be reasonable agreement between the two.

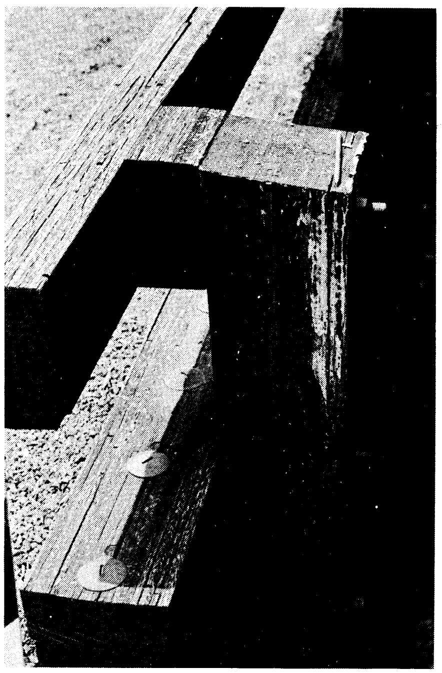
Based on the data collected during the five-year study it appears that the largest dimensional changes are positive (expansion) and in the directions perpendicular to the glue planes. The data indicate that the rails have increased in thickness about 6.0%, the wheel guards about 3.5%, the posts about 2.5%, and the stringers about 2.0%. A 2.0% increase in the thickness of a stringer is only 1/16 in. (1.6 mm) and may be within the precision that can be expected for data collected by different operators. The 6.0%, 3.5%, and 2.5% increases in the thickness of the rails, wheel guards and posts, respectively, are easily understandable considering the amount of delamination that has occurred in these members. Also, part of the 3.5% increase in the depth of the rails and wheel guards may be explained by the fact that because of the delamination, these members have warped slightly and the upper and lower surfaces have become irregular. Some of the areas of delamination are exhibited in the photos in Figures 4 and 5, taken after two years of service life.

All members have exhibited dimensional changes in the depth or thickness direction, with the greatest increases being exhibited during the first two years of service life. The dimensional changes were probably caused by most of the changes in the moisture content of the members as discussed next.

Table 4

Percent Change in the Dimensions of Selected Members
Based on Original Data Collected on May 4, 1977

Type Member	Type Dimension	Equipment	Reference Point No.	Percent Change in Dimensions at Indicated Date						
				8/11/77	11/15/77	4/24/78	6/13/79	9/24/80	10/29/81	5/10/82
Deck Panel	Depth	Dial gage	9,10,12,13	0.29	0.85	0.46	1.51	1.99	2.13	1.72
Deck Panel	Depth	Calipers	None	0.00	1.69	0.57	1.14	1.14	1.71	2.29
Deck Panel	Width (parallel to traffic)	Tape	27,28	0.06	0.18	0.23	0.32	0.11	0.28	0.28
Deck Panel	Length (perpendicular to traffic)	Tape	None	0.04	0.00	-0.04	-0.08	-0.12	-0.08	-0.08
Exterior Stringers	Depth	Dial gage	1,2,3,4	-0.05	-0.05	0.03	0.19	0.30	0.30	0.43
Interior Stringers	Depth	Dial gage	5,6,7,8	0.01	0.05	0.28	0.45	0.73	0.97	0.85
All Stringers	Depth	Square	None	-0.05	-0.05	-0.15	0.35	0.75	0.55	0.50
All Stringers	Thickness	Calipers	None	0.88	0.44	1.95	1.31	1.97	1.53	2.84
Post	Thickness	Dial gage	19,20,21,22	0.44	1.26	1.10	2.76	3.22	3.17	3.04
Post	Thickness	Calipers	None	-0.12	1.35	0.37	2.33	2.33	2.46	2.58
Post	Width	Calipers	None	0.00	0.89	0.13	1.40	2.29	2.16	2.16
Rail	Thickness	Calipers	None	2.74	4.74	4.24	5.74	6.48	5.74	5.74
Rail	Depth	Calipers	None	0.51	1.15	2.16	3.18	3.69	3.18	3.56
Wheel Guard	Thickness	Calipers	None	2.43	2.80	2.92	3.41	3.65	3.77	3.41
Wheel Guard	Depth	Calipers	None	0.00	2.39	2.13	4.26	4.52	4.26	3.19



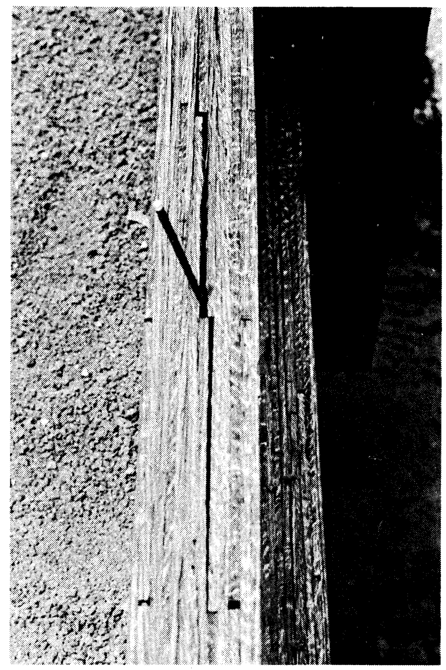


Figure 4. Areas of delamination were obvious in post, rail, and wheel guard on southeast corner of bridge after two years of service life.

Figure 5. Delamination in east rail after two years of service life.

MOISTURE CONTENT DETERMINATIONS

A Delmhorst portable, battery-powered, probe type moisture meter supplied by the Forest Products Laboratory was used to measure the moisture content of the press-lam members at selected points. The data collected are shown in Table 5, where each value is the average of four or more readings.

As can be seen from Table 5, the moisture content of the press-lam members has increased with time. However, during the first year the moisture content of the members, with the exception of the rails and wheel guards, did not change significantly. The rails and wheel guards were saturated (moisture content above 30%) when checked after 6 months of service life. Measurements were not made at the time these members were delivered to the bridge site. The stringers exhibited the lowest moisture content, which would be expected since they are the most protected members in the bridge. On the other hand, the rails, wheel guards, posts, and deck are readily exposed to rain and to temperature changes. Although moisture contents exceeding 16% were noted in the interior stringers in 1981, it is believed that a dry stress condition was otherwise maintained. The deck panels, however, entered into a wet stress condition sometime after one year of service life. Evidently, the moisture content of the members increases until it reaches equilibrium with the environment. Seasonal flucuations may explain the relatively lower readings in the spring and the higher readings in summer and early fall.

Figure 6 shows the relationship between the moisture content and the change in the thickness or depth of the members based on the data in Tables 4 and 5. It's interesting to note that the members exhibiting the highest moisture content also exhibited the greatest delamination and dimensional changes.

For comparison with the moisture content values obtained with the meter, 12 cores were removed from the structure on 5/11/1982 and analyzed for moisture content by the Forest Products Laboratory.(6) The six cores obtained from the posts and rails exhibited an average moisture content of 30.0% as compared to an average meter reading of 35.3%. Incidently, the manufacturers of the meter indicate that it is not very accurate for moisture contents above 30%, which is the approximate fiber saturation point. The six cores obtained from the stringers and deck panels exhibited an average weight loss of 18.3% due to drying and to the evaporation of creosote. It is reasonable to expect that no more than 2% to 3% of the weight loss was due to the evaporation of creosote. The average meter reading for these core areas was 15.5%. The moisture content determination on the cores indicate that the meter functions properly up to the fiber saturation point. The accuracy of the meter had been verified in 1978, when the results of laboratory tests on cores removed from selected areas of several glulam panels were found to agree with the high moisture contents detected with the meter.(7)

Table 5

Moisture Content Data in Percent

Reference No.	Reference Locations	Moisture Content at Indicated Date						
		8/11/77	11/15/77	4/24/78	6/13/79	9/24/80	10/29/81	5/10/82
1 - 4	Stringers (exterior)	10.5	7.6	9.8	12.5	13.0	13.4	11.6
5 - 8	Stringers (interior)	10.8	9.3	10.3	14.0	14.6	16.8	12.7
9 - 14	Deck (bottom exterior)	13.7	10.9	14.0	18.3	23.7	25.2	20.1
15 - 18	Deck (bottom interior)	16.4	12.6	16.0	22.0	22.8	21.3	18.6
19 - 22	Posts	16.4	13.5	19.5	33.0	50.6	46.8	32.9
23 - 24	Rails	35.5	29.0	35.0	45.0	56.5	66.9	38.8
25 - 26	Wheel guards	37.5	26.0	40.0	65.0	68.8	67.9	38.6

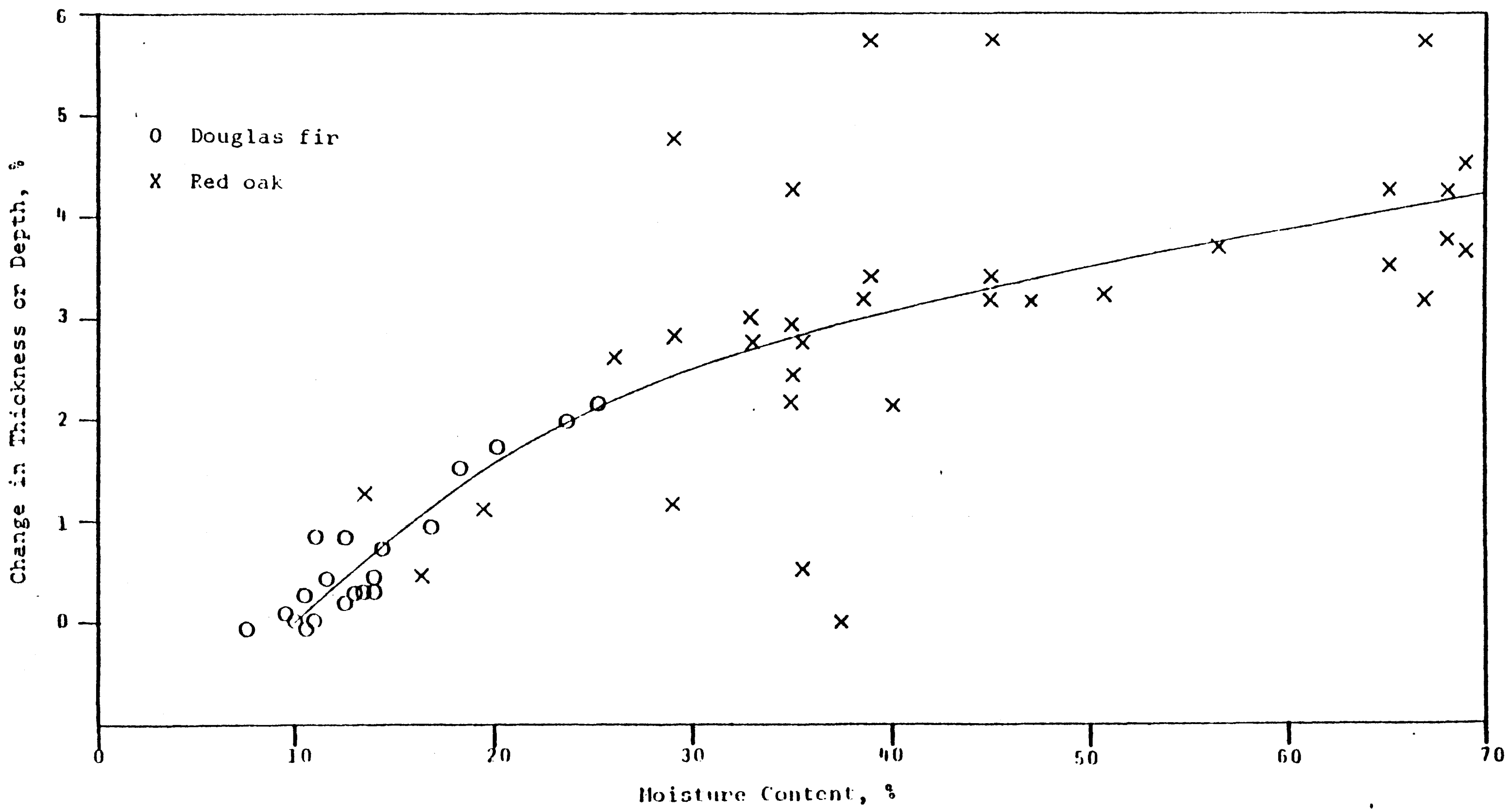


Figure 6. Relationship between moisture content and change in thickness or depth of Pres-lam members.

Therefore, the data in Table 5 can be considered as reliable, with the exception of the values greater than 30%, and these can be assumed to be 30% or greater.

WEARING SURFACE

Maintenance forces from the Edinburg Residency applied a 1-in. (25 mm) thick bituminous wearing surface to the press-lam bridge on September 27, 1977, which was approximately six months after the bridge was constructed. The six-month waiting period allowed excess creosote to leach from the deck panels, and thereby the bond between the panels and the wearing surface was better than it otherwise would have been.

Hairline cracks, most of them directly above the joints between the deck panels, were noted in the wearing surface during the inspection of the bridge after two years of service life. Subsequent inspections revealed that the cracks had become wider and more numerous. However, even after five years of service life, the overlay had not spalled away and could be considered to be in good condition, with the exception that water was able to reach the deck panels directly through the cracks. The condition of the overlay after three years of service life is shown in Figure 7.

LOADING HISTORY

Estimates of the number and types of vehicles using the bridge were made (1) with traffic counting equipment at selected times, and (2) from observations of the number and types of vehicles using the bridge during each site inspection. The data are reported in Table 6. From the table, it is apparent that several heavy vehicles and approximately 100 smaller vehicles use the secondary road bridge each day.

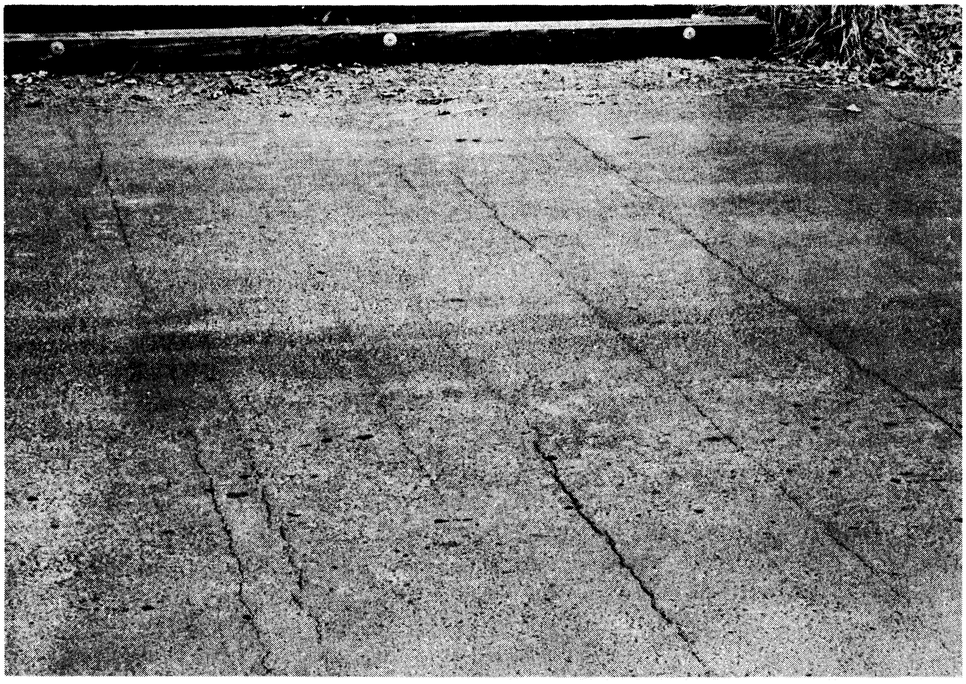


Figure 7. Wearing surface after three years of service life.

Table 6

Loading History Data
(1 lb. = 0.45 kg)

Date	Time	No. Hours	>10,000 lb.	<10,000 lb.	Total	Hour	Day
4/18/77	10:00 a.m.- 3:00 p.m.	4	0	14	14	3.5	84
5/03/77	4:00 p.m.- 6:00 p.m.	3	2	15	17	5.7	136
5/04/77	9:00 a.m.- 4:00 p.m.	7	3	18	21	3.0	72
6/01-02/77	—	24	-	—	121	5.0	121
8/11/77	11:00 a.m.- 4:00 p.m.	5	7	15	22	4.4	106
11/15/77	9:15 a.m.- 12:45 p.m.	3.5	3	14	17	4.9	117
4/24/78	10:30 a.m.- 4:00 p.m.	5.5	4	12	16	2.9	70
6/13/79	11:00 a.m.- 2:30 p.m.	3.5	3	15	18	5.1	123
2/26-27/80	—	24	-	—	77	3.2	77
9/24/80	10:30 a.m. 2:45 p.m.	4.3	7	7	14	3.3	79
10/29/81	11:30 a.m.- 3:00 p.m.	3.5	3	32	35	10.0	240
4/26-27/82	—	24	-	—	163	6.8	163
5/10/82	11:00 a.m.- 3:00 p.m.	4	1	15	16	4.0	96
5/11/82	8:45 a.m.- 2:00 p.m.	5.3	1	19	20	3.8	91
					Average 24 hr.		120
					Average on-site		110

CONCLUSIONS

1. The press-lam timber bridge was quickly assembled and road closure time was limited to eight hours (from Interim Report No. 1).

2. The results of load tests conducted after the bridge was constructed, after one year of service life and after five years of service life indicate that the AASHTO load distribution is conservative by approximately 25%.

3. The load test results indicate that the bridge has become more flexible with age.

4. The press-lam members have swelled and their moisture content has increased with age, with most of the change occurring during the first two years of service life. The in-service moisture content exceeds 16% in all members but the stringers.

5. The bridge is structurally sound and in excellent condition after five years of service life. Only the rails, wheel guards, and posts show areas of delamination.

6. Press-lam timber is suitable for use in bridge construction.

ACKNOWLEDGEMENTS

Many persons deserve to be acknowledged for their contributions to this project.

The author acknowledges with appreciation the contributions of John A. Youngquist, David S. Gromala, and others at the Forest Products Laboratory, who were responsible for the design, fabrication, and treatment of the press-lam members used in the bridge. Mr. Gromala also instrumented the bridge. Fred Sutherland and Larry Misenheimer of the Department participated in and approved the design of the bridge.

A bridge crew from the Staunton District supervised by Lloyd Derrer constructed the bridge, and Edinburg Residency personnel under John Chiles, and later W. H. Bushman, provided traffic control.

Staunton District Equipment Engineer Jim Dunkum provided the load test vehicle and driver, while personnel from the Traffic and Safety Division under J. P. Mills, Jr., and later A. L. Thomas, Jr., provided scales to weigh the load test vehicle. Michael Burton, Jimmy French, Clyde Giannini, and Bobby Marshall, technicians at the Council, assisted with the collection of data.

Arlene Fewell handled the secretarial responsibilities, and the Report Section under Harry Craft edited and reproduced the three reports on the project.

The study originated as a four-party cooperative agreement between the U. S. Forest Service, the Forest Products Laboratory, the Virginia Department of Highways & Transportation, and the Research Council. It was initiated under the direction of Jack Dillard, and Harry Brown provided administrative guidance and support.

The study was financed with HPR funds as an implementation study.

REFERENCES

1. Sprinkel, M. M., "Working Plan — Evaluation of the Performance of a Press-Lam Timber Bridge," Virginia Highway and Transportation Research Council, February 1977.

2. _______, "Evaluation of the Performance of a Press-Lam Timber Bridge — Interim Report No. 1, Bridge Installation and Load Test," Virginia Highway and Transportation Research Council, VHTRC 78-R2, July 1977.

3. _______, "Evaluation of the Performance of a Press-Lam Timber Bridge — Interim Report No. 2, Bridge Performance and Load Test After One Year," Virginia Highway and Transportation Research Council, VHTRC 78-R57, June 1978.

4. Youngquist, John A., and David S. Gromala, "Press-Lam Timbers for Exposed Structures," Preprint, ASCE Spring Convention, Pittsburgh, Pennsylvania, April 24-28, 1978.

5. American Association of State Highway & Transportation Officials, Standard Specifications for Highway Bridges, 11th ed., 1973.

6. Personal correspondence, David Gromala, Forest Products Laboratory, Madison, Wisconsin, May 19, 1982, File 29.7.1.

7. Personal correspondence, William Henry, Koppers Product Development Department, Orville, Ohio, December 11, 1978, File 29.18.