EVALUATION OF THE PERFORMANCE OF A PRESS-LAM TIMBER BRIDGE

Interim Report No. 1

Bridge Installation and Load Test

by

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Research Engineer

Virginia Highway and Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia
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SUMMARY

The report describes the installation and load test of the world's first press-lam timber bridge superstructure. A five-man crew replaced the substandard steel stringer-timber deck superstructure on Rte. 610 over Little Stoney Creek in Shenandoah County with the press-lam superstructure in about four work days, and the road was closed for only eight hours. Results of the load tests conducted two weeks after the 17.5 ft. (5.33 m) span, 2-lane bridge was constructed suggest that the stringer live load distribution specified by AASHTO is conservative. The Research Council will inspect and load test the bridge periodically over a five-year period.

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INTRODUCTION

The world's first 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. The Douglas fir stringers and deck panels and the red oak rails and posts used in the experimental bridge were fabricated at the U. S. Forest Products Laboratory (FPL) of Madison, Wisconsin, using their recently developed press-lam process which involves the rotary peeling of a log into thin sheets that are glued together to produce lumber of almost any desired dimension.(1) The stringers and deck panels were load tested at the FPL and all the press-lam bridge components were shipped to the Koppers Company in Orville, Ohio, for treatment with creosote and then shipped on to Virginia.(2) The Research Council is responsible for periodically load testing and evaluating the performance of the experimental bridge over a five-year period from its installation. This is the first of three reports which will be issued on the five-year evaluation.(3)

INSTALLATION

A five-man bridge crew from the Department of Highways and Transportation replaced an existing steel stringer-timber deck bridge with the experimental press-lam bridge superstructure in about four work days as shown in Table 1. The installation is illustrated in Figures 1-5 and the completed structure is shown in Figure 6.

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Table 1

Labor and Equipment Required to Install Press-Lam Bridge

Activity	Labor			Equipment	
	No.Days	No.Men	No.Man-hours	Type	No. Hours
Moving press-lam members and crane to bridge site	1	5	40	1 Bridge truck 1 Crane	16
Erecting press-lam* stringers and deck panels	2	6	96	1 Bridge truck 1 Crane 1 Pickup truck 1 Boom truck	64
Attaching press-lam rails and posts and general cleanup	1	5	40	1 Bridge truck 1 Crane	16
Totals	4		176		96
Totals/$ft.^2$			0.37		0.20
Construction Rate ($ft.^2$/hr.)			2.7		5.0

* Includes approximately 1 hour for removal of existing SS-TD superstructure. The hardware had been removed from the existing superstructure while the substructure was being widened.

1 $ft.^2$ = 930 cm^2

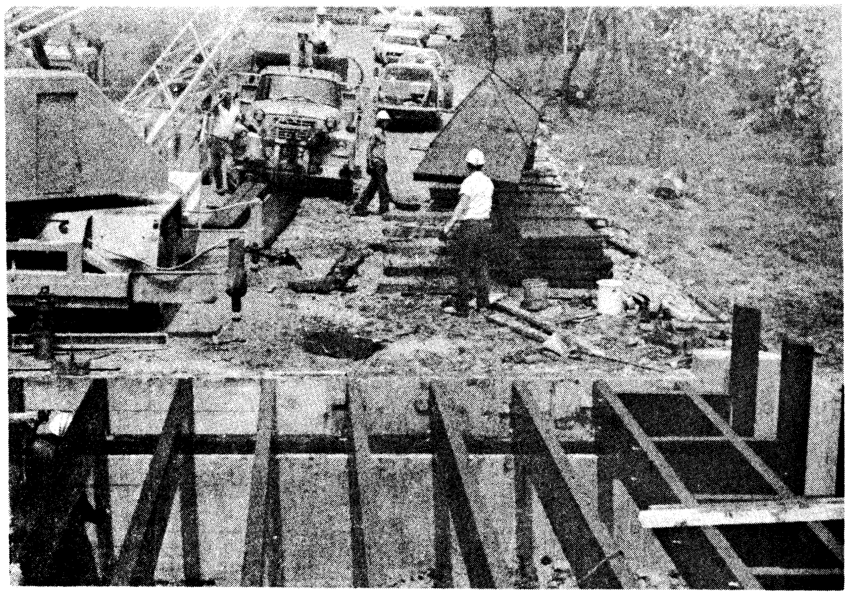


Figure 1. The press-lam stringers were placed and **connected** before the crane moved the first press-lam **panel.**

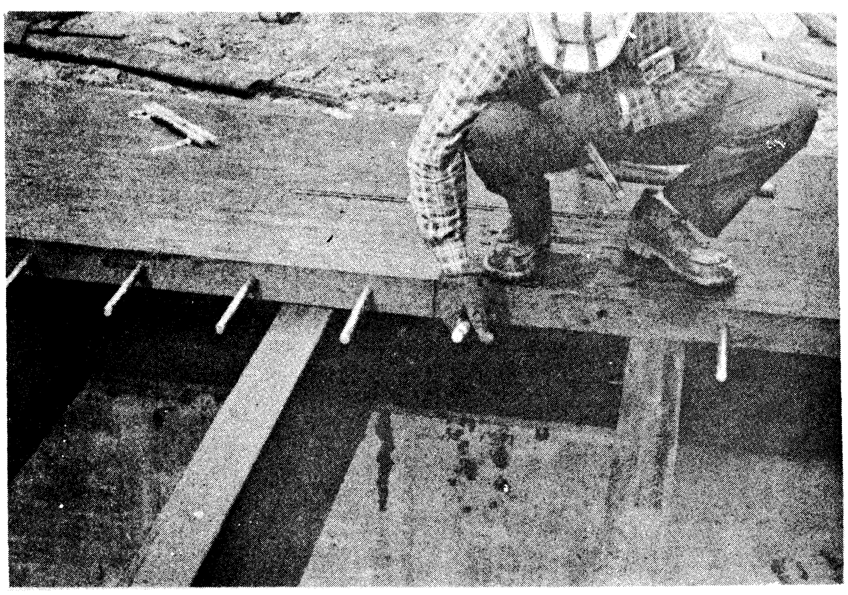


Figure 2. Galvanized coated steel dowels (0.88 inch [2.2 cm] diameter x 13 inches [33 cm] long) are inserted **in** the 3.5 inch (8.9 cm) thick panels.

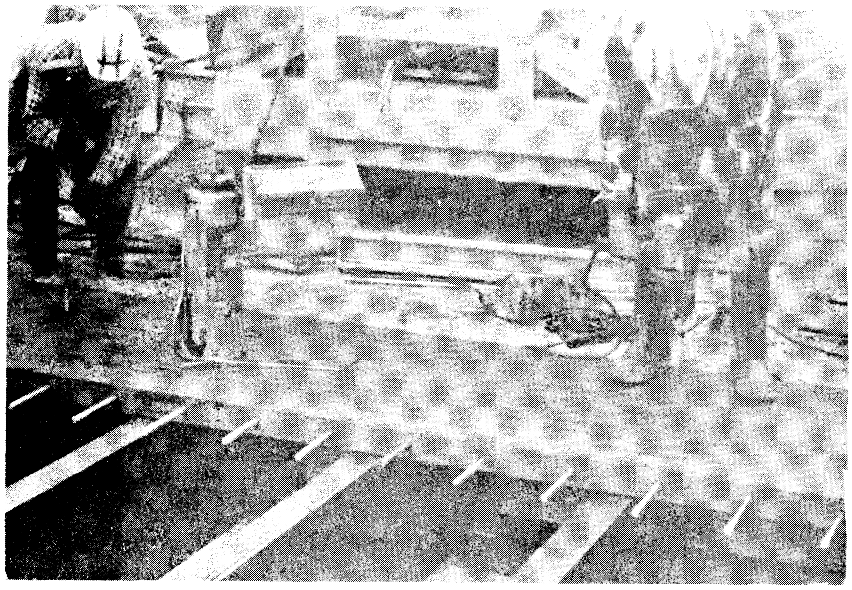


Figure 3. Holes are drilled in the panels prior to connecting them to the stringers with steel spikes.

Figure 4. Spikes are driven to connect each panel prior to positioning adjacent panel.

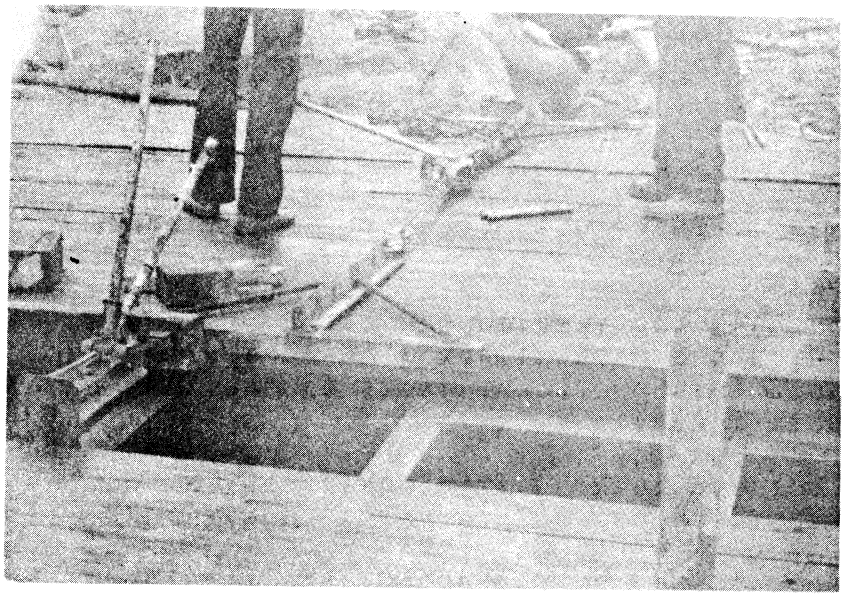


Figure 5. First two panels are jacked together.

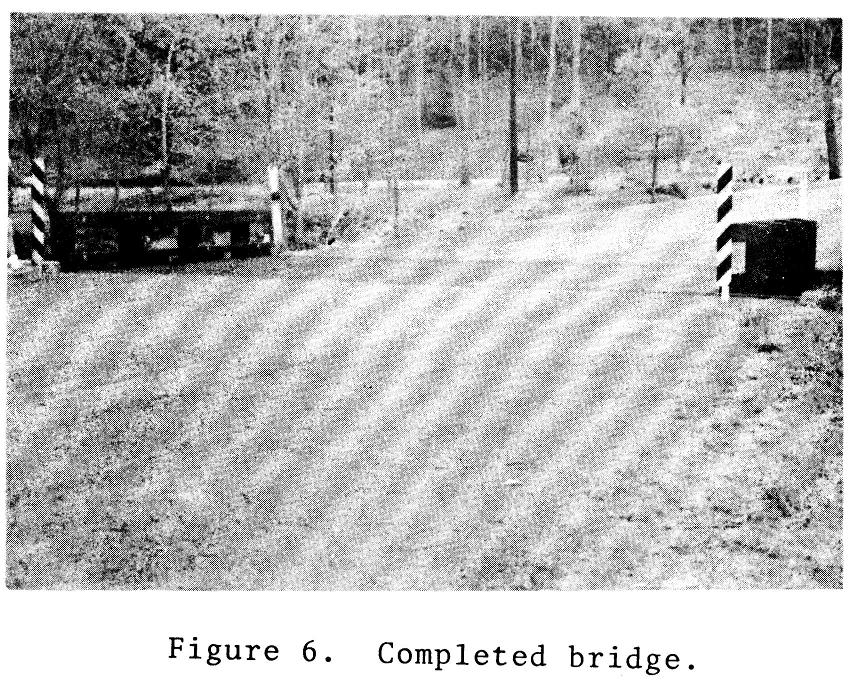


Figure 6. Completed bridge.

The press-lam bridge was assembled quickly with only minor delays and inconveniences being associated with the following items.

1. The deck panels increased in width (direction perpendicular to the plane of the glue lines) when treated with creosote.

2. The last panel was difficult to jack into place because the jack could not be positioned between the backwall and the panel. A crane was used to support the jacks as the last panel was positioned.

3. Creosote leaking from the press-lam members caused undesirable working conditions.

A report issued by the author the first part of this year has showed that a conventional steel stringer-timber deck (SS-TD) structure of comparable size can be constructed by a similar crew at the rate of 1.5 ft.^{2} (0.14 m^2) per man-hour and 3.0 ft.^{2} (0.28 m^2) per equipment hour.(4) Based on the information reported in Table 1, the press-lam bridge superstructure required 45% fewer man-hours and 40% fewer equipment hours for construction than did the comparable SS-TD structures.

The press-lam structure can be constructed faster than an SS-TD structure for the following reasons:

1. The deck panels are larger than the individual timbers used in SS-TD structures.

2. The deck panels are fastened to the stringers with spikes which are spaced at about 2-foot (0.61 m) intervals along each stringer, whereas conventional timbers are fastened to steel stringers with bolts and special clips placed at about 10-inch (0.25 m) intervals along each stringer.

3. The press-lam members are treated prior to being shipped, whereas steel stringers are usually painted at the bridge site.

Although four days were required to install the press-lam structure, the road was closed to traffic for only one work day. With experience, the bridge crew could probably construct a press-lam bridge somewhat faster than reported here.

A road must be closed less than one work day when an older superstructure is replaced with a new SS-TD structure, because the stringers and timbers can be positioned in several hours and the timbers can be anchored, the rails connected, and the structural steel painted while the bridge is open to traffic. Since steel dowels are used for load transfer between the press-lam panels, the panels must be connected as they are placed and the bridge cannot be conveniently opened to traffic until all the panels are placed and connected. Labor and equipment costs are substantially less, but the road closure time is slightly greater for the press-lam structure. Because material costs account for 70% of the total cost of a 20-ft.(6.1 m) span SS-TD superstructure, material costs will likely determine if a press-lam timber bridge is competitive. The estimated material cost for a 20-ft.(6.1 m) span SS-TD superstructure is $\$9.00/\text{ft.}^2$ ($\$96.88/\text{m}^2$).(4)

When compared with the precast concrete slabs recently used to widen and replace some short span bridges of similar size in the same area, the press-lam superstructure construction required several hours more road closure time, 40% more man-hours, and 45% more equipment hours at the bridge site. The precast slab structures were installed at the rate of 4.5 ft.^2 (0.42 m^2) per man-hour and 9.0 ft.^2 (0.84 m^2) per equipment hour, and at an average total cost of $\$9.49/\text{ft.}^2$ ($\$101/\text{m}^2$).(4)

LOAD TESTS

On May 4, 1977, the rear tandem axle of a trailer loaded with a D16 dozer was used to load test the press-lam bridge (see Figure 7). Prior to the tests a scales 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 8. The scales indicated that, within 3%, each of the pairs of wheels supported 25% of the total load on the tandem. Therefore, for practical purposes it was assumed that each pair of wheels produced a load of 10,200 lb.(4,590 kg).

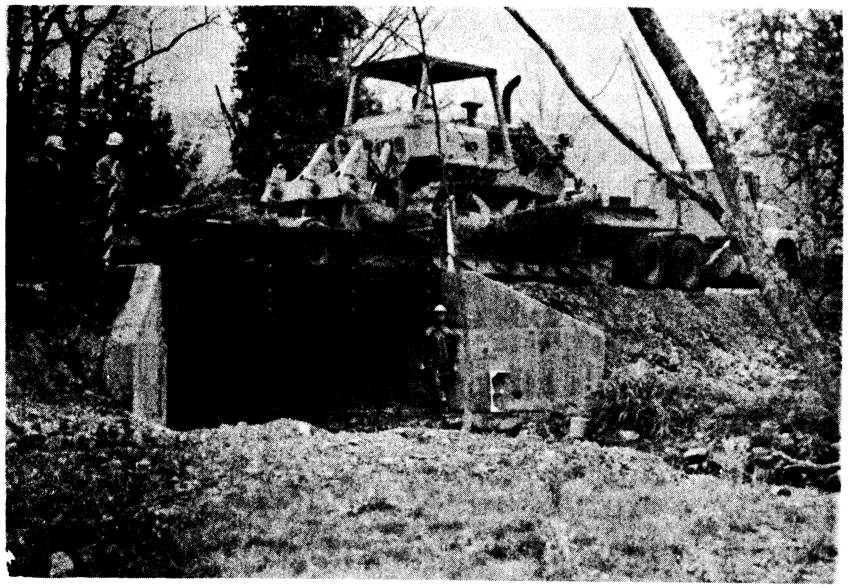


Figure 7. Loaded rear tandem axle of trailer used for load test.

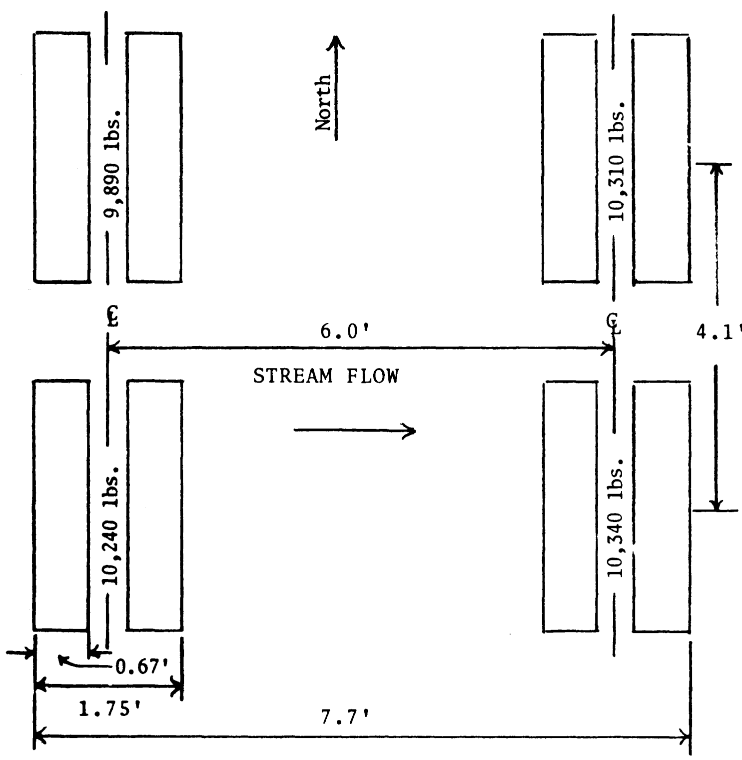


Figure 8. Rear axle dimensions and wheel loads. 1 lb. = 0.45 kg, 1 ft. = 30.5 cm.

Tests of Interior Stringers

The theoretical flexural stress in the interior stringers produced at midspan by the test vehicle was not as much as 1% less than the theoretical flexural stress that would be produced by one 32,000 lb.(14,400 kg) AASHTO design axle. Because of the short span length of the press-lam bridge the AASHTO 32,000 lb. (14,400 kg) concentrated load controls the moment design of the stringers. Permits are issued in Virginia for tandem axle loads up to 44,000 lb.(19,800 kg), which is 7.8% greater than the tandem load of the test vehicle. The theoretical midspan interior stringer deflection, D, for an AASHTO 32,000 lb.(14,400 kg) axle placed at midspan is

$$D = PL^3/48\ EI,$$

where

P = (32,000) (1/2) (S/4) = 10,000 lb.(4,500 kg),

S = stringer spacing = 2.5 ft. (76.2 cm),

L = design span length = 17.5 ft. (533.4 cm),

E = 1.7×10^6 psi (11.7×10^6 Pa), and

I = 3,000 in.^{4} (124,869 cm^4).

Therefore,

$$D = (10,000)\ (17.5)^3 / [\ (48)\ (1.7 \times 10^6)\ (3,000)\ ]$$

$$D = 0.378 \text{ inch } (9.60 \text{ mm})$$

The theoretical midspan interior stringer deflection, D, produced by the load test vehicle is

$$D = P\,a\,(3L^2 - 4\,a^2)\ /\ 24\ EI,$$

where

P = (40,800) (1/4) (s/4) = 6,375 lb.(2,869 kg), and

a = (17.5 - 4.1) / 2 = 6.7 ft.(204 cm).

Therefore,

$$D = (6,375)\ (6.7)\ [\ (3)\ (17.5)^2 - (4)\ (6.7)^2\ ] /$$
$$[\ (24)\ (1.7 \times 10^6)\ (3,000)\ ]$$

$$D = 0.446 \text{ inch } (11.3 \text{ mm})$$

The stringer deflections for the 13 load test positions shown in Figure 9 are shown in Table 2. 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. For load position #11 the data for stringers 2 and 3 seem to be in error and the probable deflections are shown in parentheses The maximum midspan deflection for an interior stringer with the test vehicle positioned in one lane was 0.24 inch (6 mm). With the test vehicle centered in each lane (positions #3 and #11) simultaneoulsy as simulated by position #14, there was a fairly uniform distribution of the load over the interior stringers, with a maximum deflection of 0.276 inch (7 mm). If loading position #9 were applied to both lanes simultaneously, stringer #6 would deflect 0.31 inch (8 mm) as simulated by loading position #15. Using 0.31 inch (8 mm) as the greatest live load deflection for an interior stringer, a distribution factor of S/X, where X = (4) (11.3)/8 = 5.65, 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 5 to 6 stringers support the wheel loads produced by the test vehicle.

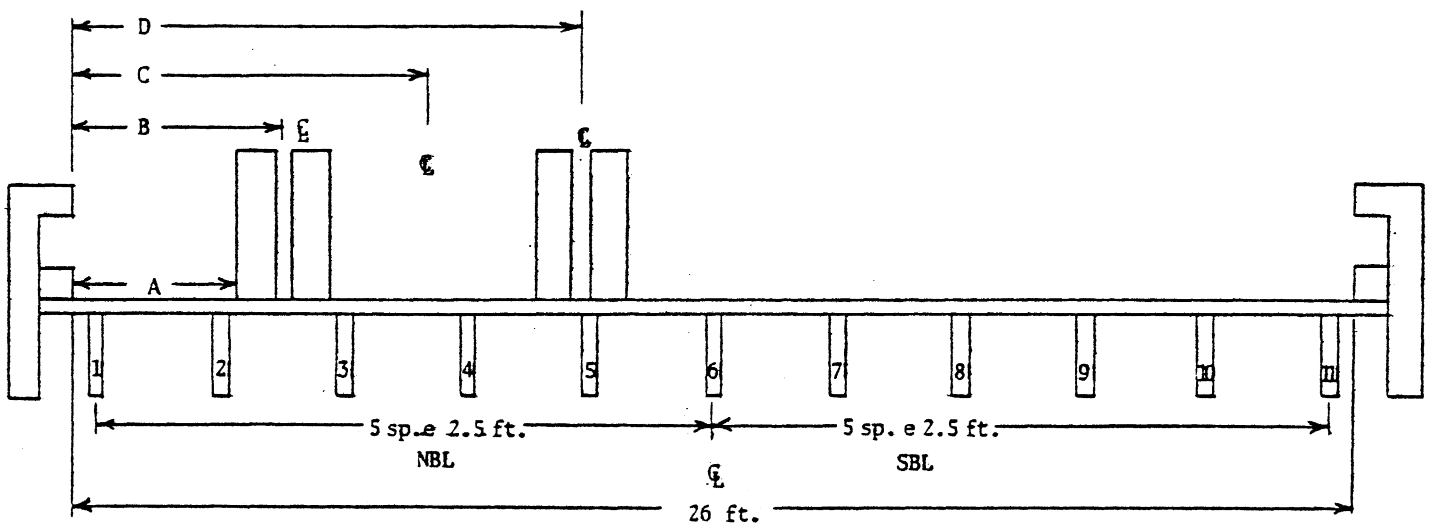
Table 2

Stringer Deflections at Midspan (mm)

Stringer Number / Loading Position	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	2.0	5.0	3.5	6.0	7.0
2	0	-0.5(a)	0	0	1.0	2.0	3.0	6.0	4.0	6.0	5.0
3	0	-0.5	-1.0	0	-0.5	3.0	5.0	6.0	6.0	6.0	3.0
4	0	-0.5	-1.0	1.0	2.0	5.5	5.0	6.0	4.0	2.5	1.0
5	-0.5	0	0	1.0	2.0	5.5	4.5	6.0	2.0	1.0	1.0
6	-0.5	0	1.0	3.0	5.0	4.5	6.0	4.0	2.0	0.5	0.5
7	-0.5	0.5	1.0	3.5	5.0	5.5	5.0	3.5	1.0	0	0
8	0	2.0	4.0	5.5	4.5	4.5	4.0	1.0	0	0	0
9	1.0	3.5	5.0	5.5	5.5	4.0	2.0	1.0	0	0	0
10	2.0	5.0	5.0	6.0	6.0	4.0	1.5	1.0	0	0	0
11	2.5	(6.0)(b) -0.5	(6.0) 4.5	6.0	5.0	4.0	1.0	0.5	0	0	0
12	4.0	6.0	4.5	6.0	4.5	2.0	1.0	0.5	0	0	0
13	7.0	5.5	4.0	5.0	2.5	0	0.5	0.5	0	0	0
14(c)	2.5	(5.5) -1.0	(5.0) 3.5	6.0	4.5	7.0	6.0	6.0	6.0	6.0	3.0
15(c)	1.0	3.5	5.0	6.5	7.5	8.0	7.5	6.5	5.0	3.5	1.0

(a) negative sign implies upward deflection.
(b) values in parentheses are probable deflections.
(c) data for positions 14 and 15 were simulated from data for positions 3, 9, and 11.

1 mm = 3.9×10^{-2} inches



Loading Position	Wheel Positions (feet) A	B	C	D	Condition of Load
1	17.71	18.59	21.59	24.59	Maximum load stringer 11
2	16.13	17.00	20.00	23.00	Maximum load stringer 10
3	15.38	16.25	19.25	22.25	Center of southbound lane
4	12.38	13.25	16.25	19.25	Midspan stringers 8 and 9; stringer 6
5	10.88	11.75	14.75	17.75	Midspan stringers 5 and 6; stringer 8
6.	9.13	10.0	13.0	16.0	Center of bridge
7	8.38	9.25	12.25	15.25	Midspan stringers 4 and 5
8	5.88	6.75	9.75	12.75	Midspan stringers 3 and 4; stringer 6
9	4.63	5.5	8.5	11.50	Stringer 3
10	3.38	4.25	7.25	10.25	Midspan stringers 2 and 3; stringer 5
11	2.88	3.75	6.75	9.75	Center of northbound lane
12	2.13	3.0	6.0	9.0	Stringer 2
13	0.60	1.47	4.47	7.47	Maximum load stringer 1

Figure 9. Loading positions used to measure stringer deflections. (1 ft. = 30.48 cm)

Tests of Exterior Stringers

AASHTO indicates that when designing exterior stringers it will be assumed that the flooring acts as a simple span between the stringers. The test vehicle was positioned as close to the curb as possible for load positions #1 and #13. For these positions the outer edge of the outside pair of tires was 0.6 ft.(18.3 cm) from the curb and the inner edge of the inner tire of the outside pair was 2.35 ft.(338 cm) from the curb. Assuming that the flooring acts as a simple span, the theoretical wheel load, P1, on the exterior stringer is

$$P1 = P(2c + b)/2L,$$

where

P = 10,200 lb.(4,590 kg),

C = 2.31 - 1.75 - 0.01 - 0.55 ft.(16.8 cm),

b = 1.75 ft.(53.3 cm), and

L = 2.31 ft.(70.4 cm).

Therefore,

$$P1 = 10{,}200\,[\,(2)\ (0.55) + 1.75\,]\,/\,[\,(2)\ (2.31)\,]$$

$$P1 = 6{,}292 \text{ lb.}(2{,}831 \text{ kg})$$

The theoretical deflection for two wheel loads of 6,292 lb. (2,831 kg) symmetrically positioned with respect to midspan and 4.1 ft.(125 cm) apart is (0.446) (6,292 / 6,375) = 0.440 inch [11.2 mm]). The maximum deflection produced in the exterior stringers was 0.276 inch (7 mm) for loading positions 1 and 13. Therefore, the AASHTO design which assesses a simple span distribution of the wheel load 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 AASHTO 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 the values reported in

Table 3. The values ranged from 9.8×10^{-4} inches(0.025 mm) upward to 7.9×10^{-3} inches(0.20 mm) downward. The accuracy of the data appears to be about equal to the magnitude of the relative deflection; so no attempt was made to interpret the data. Since one pair of tires on the test vehicle distributed the load over a width of 1.75 ft.(53.3 cm) and the clear span between two stringers was 2.13 ft.(64.9 cm) 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.(30 cm) south of positions 11 and 12. In moving the test vehicle southward 1 ft.(30 cm) the rear wheels of the tandem moved from the center panel to the adjacent panel. The relative panel deflections were so small for each of the positions that the accuracy of the data is questionable and no attempt was made to draw conclusions. It appears that the steel dowels provided satisfactory load transfer.

Table 3

Deck Panel Deflections at Midspan
Relative to Adjacent Stringers (mm)

Loading Position	Adjacent Stringers	Relative Panel Deflections
2	9,10	0.15
2	10,11	0.15
3	7,8	0.10
3	9,10	0.18
5	5,6	0.05
7	5,6	0.08
8	5,6	0.10
11	3,4	0.13
11	4,5	0.13
11	5,6	0.18
12	3,4	0.20
12	4,5	-0.03(a)
12	5,6	0.20

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

1mm = 3.9×10^{-2} inches

LOADING HISTORY

Estimates of the type and number of vehicles using the bridge are being made with traffic counting equipment at selected times and from observations of the number and type of vehicles using the bridge during each site inspection. To date, data have been collected for several hours during the first day the bridge was under construction and during the two days the bridge was load tested and inspected, and for a 24-hour period on June 1-2, 1977. The data are reported in Table 4.

To gain an indication of the number of large loads and overloads, scratch gages were installed at midspan on the bottom side of stringers 2,5,7,10; however, no data are available at this time. The circular disks will be removed from the scratch gages periodically and mailed to the FPL for a microscopic examination.

Table 4

Loading History Data
(1 lb.= 0.45 kg)

Date	Time	No. Hours	No. Vehicles				
			>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

DIMENSIONAL DATA

On May 4, 1977, following the load test of the bridge, selected members were measured at selected locations, so that dimensional changes can be detected over the five-year period of evaluation. Calipers were used to measure the thickness 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. Because it is anticipated that growth or shrinkage will occur perpendicular to the plane of the glued surfaces, most of the dimensional data are for this direction. The deck panels increased in width about 3% and the stringers increased in width about 1.5% when subjected to the creosote treatment. The dimensional data for the members and the distances between selected reference tack points are on file at the Research Council.

MOISTURE PROBE DATA

A moisture meter supplied by the FPL was used in an unsuccessful attempt to determine the initial moisture contents of the press-lam members at selected points. Data taken on 6/06/1977 are on file at the Research Council but are not reported because they are not believed to be accurate. Other equipment will be supplied by the FPL and additional data will be taken as outlined in the working plan.(3)

CONCLUSIONS

1. The press-lam timber bridge was quickly assembled, and road closure time was limited to eight hours.

2. Results of load tests conducted two weeks after construction suggest that the AASHTO load distribution is conservative.

3. The creosote treatment caused the press-lam members to expand about 3% in the direction perpendicular to the glue planes. Creosote leaching from the treated members during construction and load tests caused undesirable working conditions. Alternative treatment methods should be considered for future installations.

4. Several heavy logging trucks and approximately 100 smaller vehicles cross the secondary bridge each day.

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