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FINAL REPORT

GLULAM TIMBER DECK BRIDGES

by

Michael M. Sprinkel
Research Engineer

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

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SUMMARY

This report discusses the construction and initial condition of the Virginia Department of Highways and Transportation's first three bridges built with glulam panels on steel stringers. The data show that superstructures with glulam deck panels are more expensive than the conventional alternative of solid plank on steel stringers. It is felt that in some instances the higher cost may be justified because the data indicate that the glulam superstructures can be constructed about 45% faster than the conventional alternative and because it is anticipated that maintenance will be less.

In general, the bridge superstructures were assembled quickly and easily but many of the panels were wider at the ends than in the middle, which resulted in obvious gaps between the panels and decks which were longer than were specified in the plans. The panels were more than adequately treated with creosote and the excess creosote bleeding from them was undesirable. Panels on two of the three bridges exhibited an initial moisture content in excess of the 16% considered to be the upper limit for assuming a dry stress condition and a further evaluation of the in-service moisture condition is recommended. Cracks developed in the bituminous concrete wearing surface on one of the bridges within four weeks after it was installed.

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INTRODUCTION

For years solid timber planks have been used as decking material on many bridges on secondary roads in Virginia. The planks are easily installed and maintained by maintenance forces who generally are not equipped to handle concrete. The timber decking has an advantage over concrete in that it is not adversely affected by deicing salts. However, the planks tend to work loose under traffic, and the connections must be periodically tightened. Also, the bituminous wearing surface usually cracks at each plank joint and produces an unsightly, rough deck surface. In recent years it has become increasingly difficult to obtain timbers long enough to extend over the width of a typical bridge.

Interest in industrialized timber bridge structures was initiated by Brown in 1972.[1] At that time the timber industry had developed several types of laminated structural members that were being used in the building industry and which were thought possibly to be suitable for use in bridge construction. Under contract, the American Institute of Timber Construction cooperated with the Research Council in providing the Department a document entitled "Typical Timber Bridge Design and Details",[2] and conducted a seminar in November 1973 to explain the document and report recent innovations in the timber industry. Advantages of laminated structural members over solid, sawed members recognized at that time were:

1. Defects in the former are scattered so that higher allowable stresses can be achieved;

2. the members may be laminated according to design stresses so as to facilitate the use of economical, low strength timbers in areas to be subjected to low stresses; and

3. the laminated members can be fabricated to much larger dimensions than are available with solid sawed timbers.

Interest in glulam continued and in February 1975 the Suffolk District designed a three-span bridge consisting of glulam stringers and deck panels to be advertised as an alternate to a composite steel stringer-concrete deck bridge;[3] however, for various reasons the bridges were not advertised until the fall of 1978. In July 1976 the Bridge Division of the Central Office issued a standard for bridges with steel beams and glulam flooring;[4] then, in May 1977 the Culpeper District completed plans for a 72-ft. span bridge consisting of steel stringers and glulam panels.[5] A contract for the bridge in the Culpeper District was awarded in the fall of 1977 and construction was completed in May 1978. About the same time the Culpeper bridge was advertised, the Salem District advertised for bids for materials to be used by maintenance forces in the construction of two bridges — one a 56-ft. span and the other a 46-ft. span. The first of these two bridges was completed in July 1978 and the second in August.

The purpose of this report is to present information on the construction, cost, and in-service condition of the three bridges constructed to date with glulam deck panels.

DESCRIPTION OF BRIDGES

A description of the three bridges with glulam deck panels is provided in Table 1. Because the glulam deck superstructures are considered an alternative to the more conventional steel stringer-timber plank superstructures, a description of two of these conventional bridges is also provided in Table 1 for comparison. It was envisioned that the glulam deck superstructures would be constructed faster than the plank deck superstructures because there are fewer pieces to handle at the bridge site. The hypothesis is confirmed by the construction time data for the five bridges which are reported in Table 2. A description of the five bridges and details of their construction follow Tables 1 and 2.

Table 1

Description of Bridges

Type Deck	Location (County)	Span (ft.)	Roadway width (ft.)	Steel Stringers No.	Steel Stringers Size	Structural Steel (lb.)	Timber Bolsters (MFBM)	Timber Decking No. Pieces	Timber Decking MFBM
Glulam	Fairfax	72	30	5	W36 x 230	100,600	4.0	18	15.23
Glulam	Patrick	56	26	4	W36 x 182	55,410	1.8	14	10.21
Glulam	Henry	46	26	4	W36 x 135	37,750	1.5	12	8.38
Plank	Pittsylvania	42	23	14	W21 x 62	41,426	NA	52	5.62
Plank	Nelson	30	21	13	W16 x 45	22,660	NA	37	3.87

Table 2

Construction Time for Bridge Superstructures in Man-Hours
(Man-hours per ft^2)

Construction Activity \ Bridge	Glulam Deck Superstructures: Fairfax County	Glulam Deck Superstructures: Patrick County	Glulam Deck Superstructures: Henry County	Steel Stringer — Plank Deck Superstructures: Pittsylvania County	Steel Stringer — Plank Deck Superstructures: Nelson County
Hauling steel to site	Site Delivered	60 (0.04)	28 (0.02)	Site Delivered	Site Delivered
Placing and connecting structural steel and diaphragms	120 (0.06)	132 (0.09)	172 (0.14)	120 (0.12)	144 (0.23)
Hauling timber	Site Delivered	24 (0.02)	18 (0.02)	56 (0.06)	80 (0.13)
Connecting timber bolsters	40 (0.02)	84 (0.06)	68 (0.06)	N/A	N/A
Installing timber deck	200 (0.09)	128 (0.09)	120 (0.10)	296 (0.31)	112 (0.18)
Painting Steel	240 (0.11)	N/A	N/A	64 (0.07)	48 (0.08)
TOTAL TIME	600 (0.28)	428 (0.29)	406 (0.34)	536 (0.55)	384 (0.61)

Fairfax County Bridge

The first bridge with glulam deck panels to be opened to traffic carries Rte. 641 over Pohick Creek in Fairfax County 0.1 mile west of Rte. 638 near the Woodbridge Exit of I-95. Guy H. Lewis & Sons, Inc. of McLean, Virginia, constructed the 72-ft. span structure which is described in Table 1 and shown in Figure 1.

Construction on the substructure was begun in January 1978, about the same time the Southern pine glulam panels were being fabricated at a plant in Morrisville, North Carolina. The panels were delivered to the bridge site in February after having been treated with creosote at a plant in Salisbury, Maryland. The panels were stored at the bridge site as delivered, in 6 bundles each containing 3 panels bound together with metal straps and separated by thin wooden strips. The panels remained in storage until installed in the bridge during the last of March.

Construction time data for the bridge superstructure are reported in Table 2. The 5-man crew spent 3 days placing and connecting the structural steel, 1 day attaching the timber bolsters, and 5 days placing and connecting the glulam panels. Another 10 days were required for 3 of the 5 men to apply the three coats of paint on the steel. The painting was time-consuming because of the scaffolding required and because of the creosote dripping onto the steel.

A 45-ton truck crane was used to position the panels, primarily because the fill material required for the roadway had not been placed and a small crane or front-end loader could be gotten to the deck at the time. Also, the 45-ton crane was on hand because it had to be used to position the structural steel. A steel section extending the width of the bridge was positioned between rail posts to support the three 10-ton jacks used to jack the panels together (see Figure 2). Although the dowels appeared to fit loosely in the dowel holes, some of the panels tended to bind during the last stage of being jacked together. Because the panels were wider at the ends than in the middle there often was a crack as wide as 3/4 in. between adjacent panels. A section which varied in width from 8-1/2 in. at one end to 11-1/4 in. at the other had to be cut from the last panel. Since the concrete backwalls had been cast with the abutments, several of the last panels had to be jacked upward in order to get the last panel jacked onto the dowells while positioned over the backwall. Primarily because many of the panels had to be jacked apart and turned around, repositioned, or jacked up off the bolsters and because the contractor was waiting for advice from the fabricators of the panels and from highway officials, 5 days elapsed between positioning of the first and the last panels. Some of the panels had to be installed twice because numbers were not clearly stamped on all of them and the contractor was not aware that they had to be installed according to the numbers. The panels had to be jacked from the bolsters because the contractor failed to place the flashing under the panels at each joint.

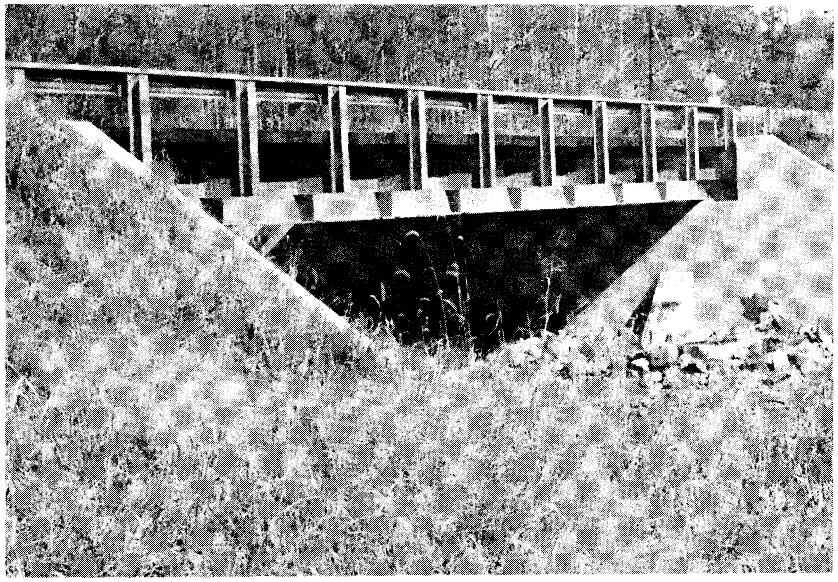

Figure 1. Fairfax County bridge.

Figure 2. Panels are jacked together on Fairfax County bridge.

Patrick County Bridge

The second bridge with glulam panels was constructed on Rte. 645 near the North Carolina line in Patrick County. Shown in Figure 3, the 56-ft. single-span structure was constructed by a bridge crew from the Department's Martinsville Residency to replace a deteriorated timber deck structure having three short spans. The superstructure is described in Table 1 and is identical to the design specified on the Standard Plans for Glulam Panels on Steel Stringers.(4)

The 14 Southern pine glulam panels for the bridge were fabricated at the same plant that fabricated the panels for the Fairfax County bridge, and were shipped from the plant to Salisbury, Maryland, to be treated with creosote. From Maryland, the panels were shipped to the Department's Peters Creek Area Headquarters on Rte. 103, which is several miles from the bridge site. The panels were fabricated in January 1978, treated in February, and delivered to the area headquarters in March.

Traffic was maintained on Rte. 645 as the new bridge was constructed by widening one existing abutment and altering one existing pier to provide the other new abutment. While the old bridge was in service, two of the new steel stringers were placed on the widened portion of the substructure and covered with a temporary plank deck. Traffic was directed onto the temporary deck while the old structure was dismantled and the other two new steel stringers were erected. Once all the structural steel was in place, the timber planks were removed and the glulam panels were positioned as shown in Figure 4. Although the first few panels were jacked into position after being placed on the stringers, it was decided that the best procedure was to place all the panels on the stringers and then jack them together. Once all the panels were on stringers there was a larger surface upon which to work than was provided by the one-lane, temporary deck.

The panels went together easily because many of the dowels fit fairly loosely in their holes and most of the panels fit together fairly well. Quite often the front-end loader used to position the panels on the stringers could be used to pull or push the panels together (as shown in Figure 5) and the jacks were not needed.

The construction time data for the bridge superstructure are shown in Table 2. Three and one-half days were required for the bridge crew, which ranged in size from 4 to 6 men, to place and connect the structural steel and another 2 days were required to install the timber bolsters. The 6-man crew plus 2 extra men worked 2 days to place and connect the glulam panels. Traffic was maintained during the installation and no vehicle had to be delayed more than 10 minutes.

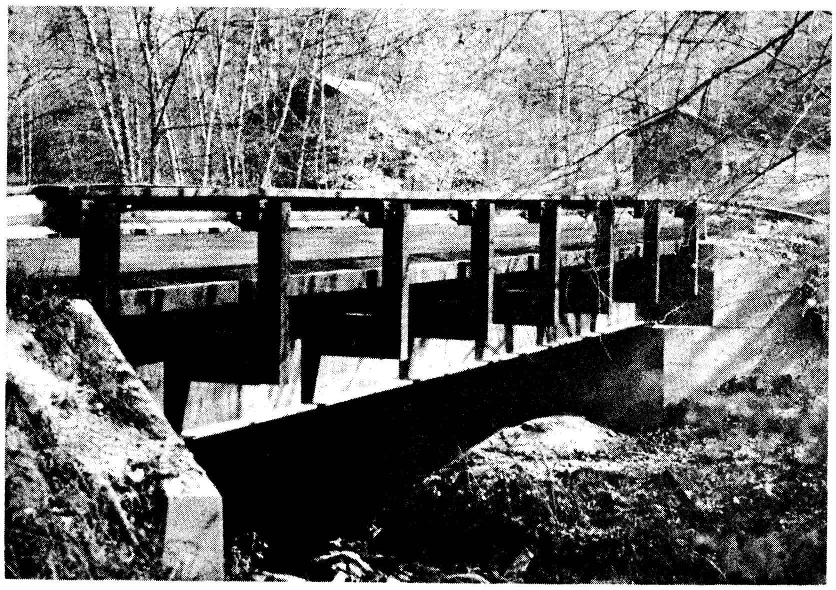

Figure 3. Patrick County bridge.

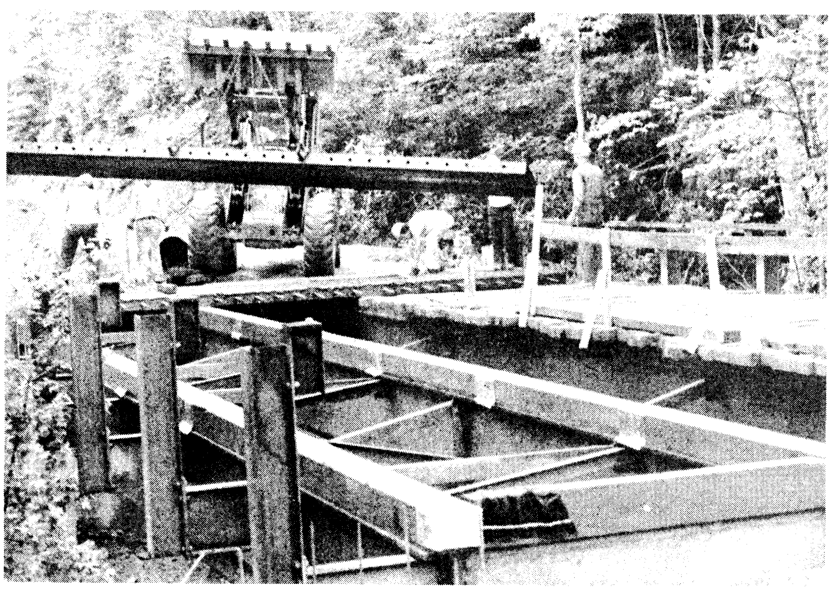

Figure 4. Traffic was maintained as temporary timber planks were replaced with glulam panels on Patrick County bridge.

Figure 5. Front-end loader used to pull panels together on Patrick County bridge.

Some of the panels were wider at the ends than in the middle so there were cracks between some of the panels. In general, the panels fit better than on the Fairfax County bridge and the total width of the fourteen 4-ft. wide panels was only 6.5 in. more than the 56-ft. specified on the plans. Since the concrete backwalls were cast after the panels were installed, the extra 6.5 in. did not have to be cut from one of the panels.

Henry County Bridge

The same bridge crew that constructed the Patrick County bridge constructed the third glulam bridge. This bridge, which is shown in Figure 6, is located on Rte. 687 in Henry County not far from Martinsville. The 12 Douglas fir panels used were fabricated in Albert Lee, Minnesota, in February 1978, shipped to Richmond, where they were treated with creosote in March, and were delivered to the Martinsville Residency in March. Each end panel was 3 ft. wide and the 10 interior panels were the standard 4 ft. in width. A description of the bridge superstructure, which is of the standard design, is given in Table 1.

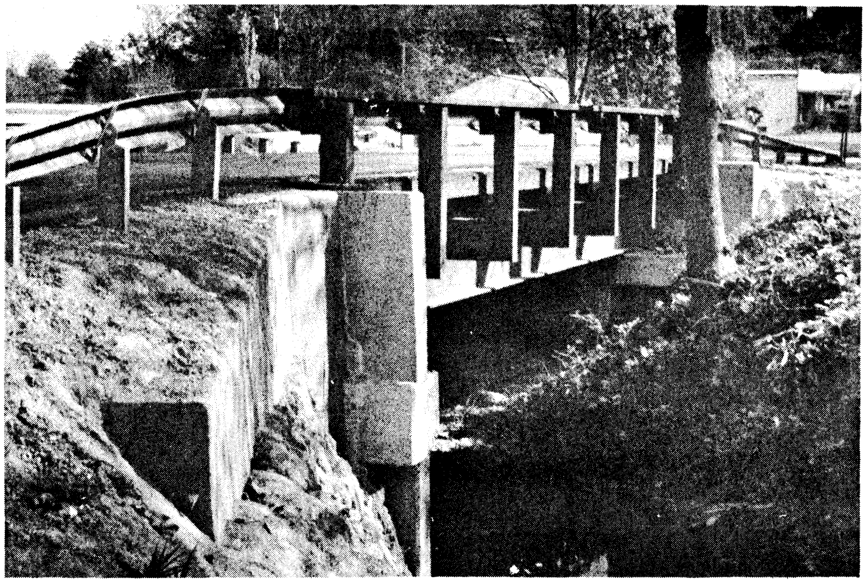

Figure 6. Henry County bridge.

Although the substructure supporting the old bridge was widened while the bridge was in service, the road was closed for about 2 weeks for the construction of the new superstructure. As with the Patrick County bridge, the grade had to be raised approximately 2 ft. to accommodate the additional depth of the glulam superstructure above that required for the steel stringer-timber plank deck superstructure being replaced. The road was closed because it was easier to construct the new bridge and place the fill without accommodating traffic and because the detour was short enough not to present an appreciable hardship to the motorist.

Construction time data for the Henry County bridge are shown in Table 2. Eight to 10 men were involved in the construction of the superstructure. Two and one half days were required to place and connect the structural steel, 1 day to install the timber bolsters, and 1-1/2 days to complete the installation of the glulam panels. As with the Patrick County bridge, two front-end loaders were used to set the steel stringers and one was used to position the panels. The panels were fabricated to close tolerances and the dowels fit snuggly. In fact, they fit so snuggly that the excess creosote in the dowel holes prevented the panels from coming any closer together than about 1/2 in. The problem was diagnosed after the first 4 panels were positioned and the excess creosote was removed from the holes on subsequent panels prior to pushing them together (see Figure 7). In most instances the front-end loader was able to push the panels together and the jacks were not needed. One backwall was constructed after the panels were installed and the extra 2 in. of deck length did not have to be removed from the last panel.

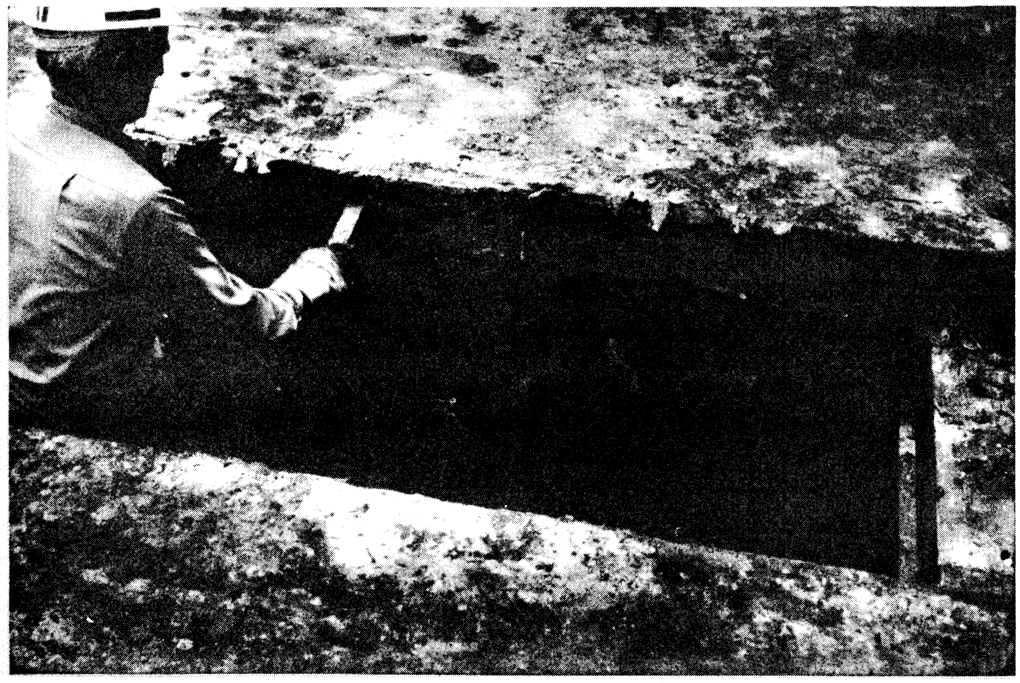

Figure 7. Excess creosote is removed from dowel holes on Henry County bridge.

Conventional Steel Stringer-Timber Plank Superstructures

Because the glulam deck superstructures are considered an alternative to the more conventional steel stringer-timber plank superstructures, case studies of two of these conventional bridges were made to get an idea of the relative advantages and disadvantages of the two bridge types. Information on a bridge constructed in Pittsylvania County and another in Nelson County was obtained for a report prepared in 1976.(7) A description of the superstructures of the two bridges is provided in Table 1. In the case of the Pittsylvania County bridge, which is shown in Figure 8, a 7 to 8 man crew spent 2 days placing and connecting structural steel and 5 days placing and connecting the timber planks. Four men spent 2 days painting the steel. In the case of the Nelson County bridge, a 4-man crew spent 2-1/2 days placing and connecting the structural steel and 3-1/3 days placing and connecting the timbers. Three men spent 2 days painting the steel. Construction time data for the bridges are shown in Table 2 and additional details can be found in reference 7.

From Table 2 it can be observed that approximately 45% less time was required to place and connect the structural steel on a glulam structure than on a steel stringer-timber plank SS6 structure. Likewise, approximately 45% less time was required to place and connect the timber bolsters and glulam panels than was required to place and connect the solid sawn plank. It appears that a 20% reduction in on-site construction time can be achieved by using weathering steel with either type structure. Although the time savings would vary depending upon the distance between the bridge site and storage area, it appears that a 20% reduction in construction time can be achieved by having the structural steel and timber shipped from the fabricating plant to the bridge site rather than to a storage area; however, this would not be practical in many situations.

In general, approximately 45% less time was required for the construction of a panel superstructure than was required for an SS6 superstructure. Since similar equipment is required for each type bridge, one can expect a 45% reduction in equipment costs for the construction of a panel superstructure.

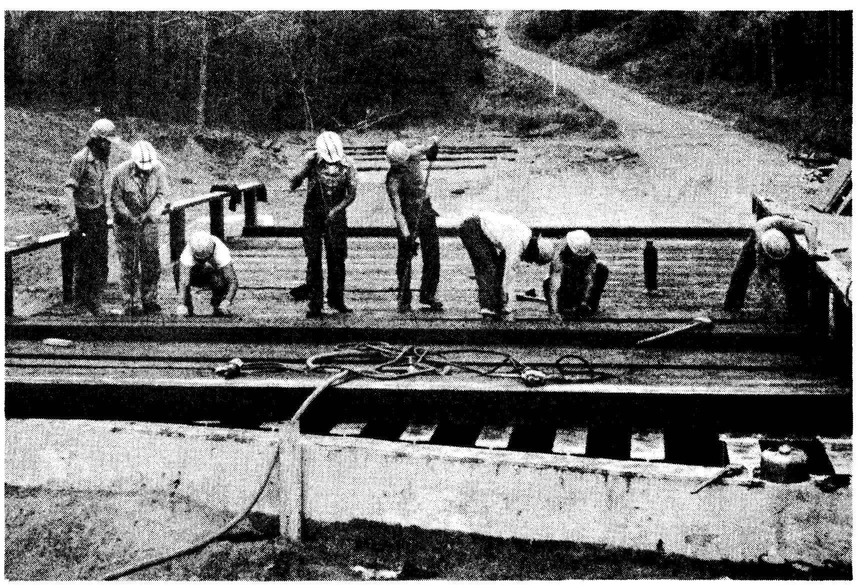

Figure 8. Solid sawed timber planks being installed on the Pittsylvania County bridge.

COST INFORMATION FOR GLULAM BRIDGE SUPERSTRUCTURES

Cost information for bridge superstructures consisting of glulam panels on weathering steel stringers and solid sawn timber plank on weathering steel stringers is shown in Figure 9. The materials cost information was obtained by multiplying the average purchase price for glulam panels, weathering steel stringers, and solid sawn bolsters based on the purchase orders for the Patrick and Henry County bridges and the average purchase price for solid plank based on purchases during the same period by the quantity of materials required for a given span as reported in "A Standard for Bridges — Steel Beams with Glulam Flooring"(4) and "Standard Steel Beam Bridges".(6) The labor and equipment cost information was obtained by multiplying the average rate at which maintenance forces constructed the Patrick and Henry County bridges and the average rate at which similar forces constructed the Pittsylvania and Nelson County bridges (plank deck bridges) by the quantity of material required for a given span length for each bridge based on standard plans and by an average wage rate, including overhead, of $6 per hour and average equipment rate of $1.5 per man-hour, which is equivalent to about $3 per equipment hour. It should be realized that the labor and equipment cost information is theoretical since the rate at which materials are placed is not constant and would tend to be slightly less for short spans than for long spans. Cost information for each of the three bridges with glulam decks is plotted in Figure 9. It can be seen that the Patrick and Henry County bridges fit the cost curves fairly closely and that the Fairfax County bridge, which was constructed under contract, was much more expensive.

Significant facts reflected by Figure 9 are noted below.

1. Structural steel costs are much less for GL than for SS6 bridges (approximately 25% less for 60-ft. span).

2. The cost for glulam decking material is significantly higher than for solid sawn plank decking material (approximately 250% more).

3. Labor and equipment costs are less for GL than for SS6 (approximately 45% less).

4. The total cost is more for GL than for SS6 (approximately 10% more for a 60-ft. span, but the difference in cost increases as the span length decreases).

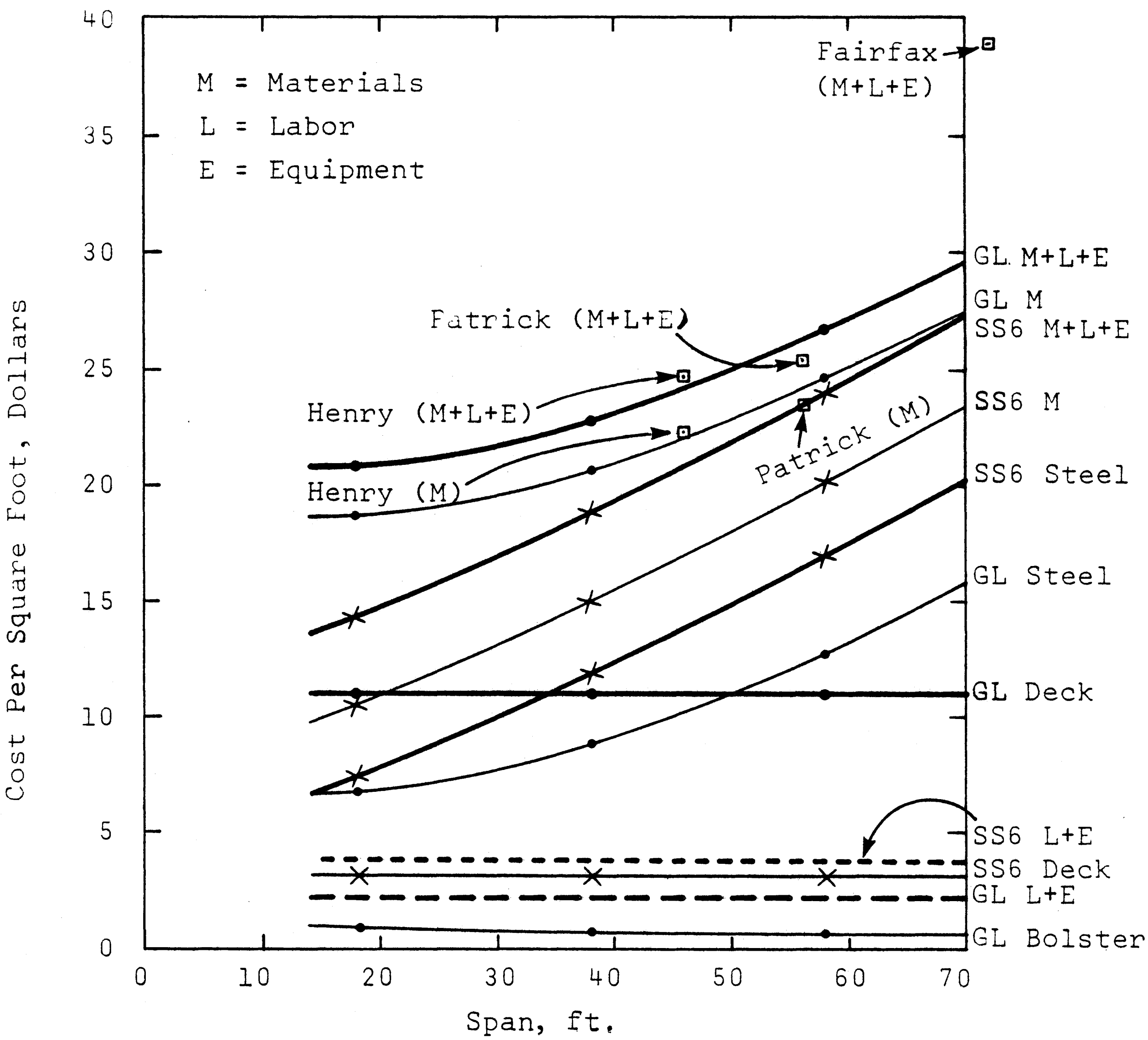

Figure 9. Cost vs. span for glulam and SS6.

5. Material costs for GL exceed total costs for SS6 (almost equal for 60-ft. span).

6. If the cost of the glulam deck panels could be reduced the GL system would be more competitive with the SS6 system (a reduction of 25% would make a 60-ft. span competitive but a reduction of 55% would be required to make a 20-ft. span competitive).

When either type structure is constructed by maintenance forces, labor and equipment costs amount to a small portion of the total cost of the bridge. Therefore, relatively little cost benefit is to be gained from being able to construct the glulam system faster than the SS6 system. Both deck systems can be installed while the bridge is opened to traffic, so the motorist benefits little from the reduced on-site construction time that can be achieved with the glulam deck system. With respect to maintenance costs, the glulam system should provide significant advantages over the SS6 system because (1) the bituminous wearing surface should adhere to the deck panels better than to the solid sawn plank, since deflection cracking should be less; and (2) there is less structural steel to paint with the glulam system than with the plank system, an advantage that will not be realized if weathering steel is used.

A disadvantage of glulam as compared to SS6 construction that is more significant in bridge replacement than in new construction is that the steel stringers have to be deeper for the glulam system since fewer stringers are required. As the depth of the stringers increases, more fill material is required and, if a suprestructure is replaced, the grade will likely have to be raised.

Possible changes in the glulam deck system that could result in a reduction in cost include the elimination of the timber bolster and the dowels. It is believed that at least $1 and maybe more per square foot could be saved by eliminating the timber bolster. If the bolsters are eliminated, it is likely that it would be necessary to work beneath the deck to secure it to the stringers and therefore, from an installation standpoint, it may be desirable to continue to use the bolsters. A glulam deck system has been developed that does not require dowels but rather relies on special clips which secure the panels to the stringers and reportedly[8] stiffens the panels enough to prevent breakup of a bituminous overlay due to differential deflection. With this system, the roughly $1.50 per square foot dowel cost is eliminated and the labor required to drill the dowel holes is eliminated. The author was not able to obtain a good cost estimate for the clip system at the time this report was prepared, but it is felt that it has sufficient merit to warrant further investigation.

There are certain to be many who will prefer the glulam deck system over the SS6 system despite the higher cost because it is anticipated that the glulam deck will maintain its pleasing appearance for a longer period of time and require less maintenance than the SS6 structure.

INITIAL CONDITION OF GLULAM DECK PANELS

To provide an indication of the condition of the glulam deck panels on each of the three bridges prior to being subjected to traffic, data were collected on the total width of the panels, the degree of creosote treatment, and the moisture content. These data should be useful in making an evaluation of the long-term performance of the panels.

The fact that the total width of the panels on each of the three bridges was greater than specified on the plans has been mentioned earlier. Table 3 gives the dimensional data, an explanation for the excess width, and the corrective measure used to accommodate it. From Table 3 it can be seen that the principal reason for the excess width of the panels on the Fairfax and Patrick County bridges was that many of the panels were wider at the ends than in the center (see Figure 10). On the Henry County bridge several of the panels would not come together because of creosote in the dowel holes. Since the backwalls were cast before the panels were placed on the Fairfax County bridge it was necessary to cut the extra 8-1/2 in. to 11-1/4 in. from the last panel. One or both backwalls were cast after the panels were placed on the Patrick and Henry County bridges and the extra width was accommodated by setting one backwall back several inches. It will be interesting to see if the cracks between the panels will promote deflection cracking in the bituminous overlay.

The American Institute of Timber Construction Voluntary Product Standard PS 56-73 for structural glued laminated timber indicates that the tolerance on width shall be plus 1/2 in. or minus 1/4 in. As can be seen from Table 3 many of the panels on the Fairfax bridge and some of those on the Patrick bridge did not meet this specification, but on the average the panels for the Patrick and Henry County bridges did.

Data on the creosote treatment for the three bridges are shown in Table 4. The data on the retention of creosote for the Henry County bridge, which was treated in Richmond, were obtained from a Department test report; those for the Fairfax and Patrick County bridges were obtained in a phone conversation with a representative of the treatment plant in Salisbury. The penetration data were obtained by drilling 1/4-in. diameter holes in each panel on the Patrick and Henry bridges and noting the depth at which the borings changed color. The Fairfax County bridge was not checked for creosote penetration since the panels were fabricated and treated about the same time and at the same plant as those for the Patrick County bridge and since it was noted that when the last panel was cut on the Fairfax County bridge, the creosote had almost completely penetrated it.

Table 3

Widths of Panels After Installation

Bridge	No. Panels	Total Width		Excess Width		Reason for Excess Width	Corrective Measures
		Design	Actual	Percent	Panel, in.		
Fairfax	18	72'-0"	72'-10"	1.2	0.58	Most panels were 0.5" to 0.75" too wide at ends.	Cut 8.5" to 11.25" from last panel.
Patrick	14	56'-0"	56'-6.5"	1.0	0.50	Some panels were as much as 0.75" too wide at ends.	Adjust position of backwall.
Henry	12	46'-0"	46'-2.5"	0.5	0.25	Excess creosote was not removed from dowel holes on first four panels.	Adjust position of backwall.

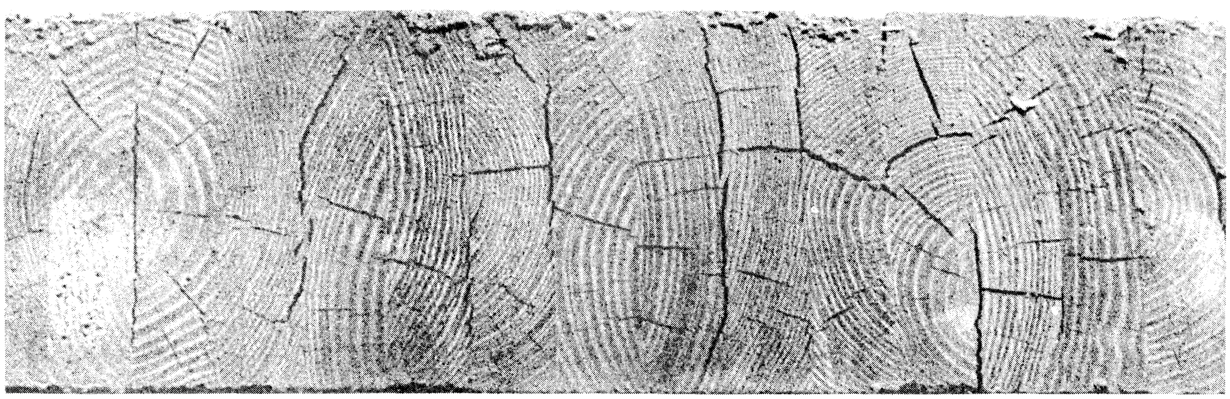

Figure 10. Cracks in the ends of the Southern pine panels contribute to the increase in the width.

Table 4

Creosote Treatment

Timber	Retention, ft.^{2}					Penetration, in.				
	Minimum		Measured			Minimum		Measured		
	AWPA	VDHT	Fairfax	Patrick	Henry	AWPA	VDHT	Fairfax	Patrick	Henry
Douglas fir	6.0	8.0	-	-	10.3	0.5	0.5 (90%)[a]	-	-	1.7 (92%)
Southern pine	6.0	8.0	10.9	10.2	-	3.0	2.5 (85%)	-	3.5 (100%)	-

[a]Percent of measurements satisfying requirement.

Based on these data it was concluded that satisfactory penetration was achieved and that the net retention was 2% to 3% more than required by Department specifications. As the panels were installed the sun tended to draw the excess creosote to the surface and create undesirable working conditions. One of the fabricators of the panels indicated that most treatment plants are equipped to remove excess creosote and that if the Department wished to have the panels as clean as possible it should so specify.

Creosote is the oldest type of wood preservative and long-term experience has shown that timbers which are properly treated have nearly permanent resistance to wood-destroying organisms. It is, therefore, envisioned that the panels will be immune to fungi and insect attack. A chromated copper arsenate (CCA) treatment has gained wide acceptance during the past 40 years and although it does not have a performance record equal to that of creosote, accelerated laboratory tests have shown that it can provide a life expectancy in excess of 50 years.* The CCA is harmless to people, plants, and animals and will not leach from the wood.[9] It could prove to be more expensive than creosote since the timber must be air seasoned or kiln dried after treatment to prevent shrinkage.[9]

The moisture content of the panels can have a significant influence on their performance. As the moisture content increases, strength decreases and susceptibility to decay increases. Changes in moisture content produce dimensional changes in the panels which can cause fasteners to work loose. A loss of moisture can cause shrinkage resulting in cracks between panels and voids around spikes.

*Personal communication with representative of AITC.

Moisture content data collected to date for the panels on the three bridges are shown in Table 5. These data were obtained with a Delmhorst portable, battery-powered, probe type meter. In taking the readings, the probes were driven to a depth of 1-1/2 in. in the 6-3/4 in. thick panels.

According to the AASHTO the panels may be designed assuming dry use stresses if their moisture content in service does not exceed 16%. Based on the data in Table 5, the moisture content of the panels at the time they were installed exceeded 16% for the Fairfax and Patrick County bridges but was less for the Henry County bridge. The decks as designed are structurally adequate for the assumption of a wet stress condition. However, it's interesting to note that the American Institute of Timber Construction suggests that "a dry use stress condition can be used for the design of glulam bridge deck panels since panels treated with creosote are unlikely to exceed 16% moisture content in service."(10) Based on the data in Table 5 the dry use stress condition cannot be used and the Institute has been notified of this apparent difference in experience.

Table 5

Moisture Content Data for Glulam Bridge Decks, in Percent

Bridge Location County	Date Measured	Middle	Edge	Avg.	σ
Fairfax	5/18	20	24	22	5
	11/1	24	24	24	5
Patrick	6/27	19	16	17	4
	8/16	22	25	24	7
	11/2	—	21	21	3
Henry	8/01	14	14	14	2
	8/16	14	16	15	2
	11/2	—	14	14	1

NOTE: Dry use condition stresses apply when moisture content is less than 16%.

Since the initial moisture readings for the Patrick and Henry bridges were taken as the panels were installed, one can easily conclude that the panels for the Patrick County bridge exhibited a high moisture content while in storage. Since it is logical to assume that the individual 2 in. x 8 in. timbers were kiln dried to a moisture content of less than 16% prior to fabrication into panels, the panels must have absorbed moisture between the time they were fabricated and the time they were treated with creosote. This would be possible for the Patrick and Fairfax County bridges since the panels were fabricated in North Carolina and shipped to Maryland for treatment. Another less likely explanation for the high moisture content in these panels is that they absorbed moisture after being treated with creosote and while in storage. The popular belief is that timber won't take up moisture after being treated with creosote, a belief which is supported by the data in Table 5 which show that, in general, the moisture content of the panels has not changed significantly during the 2-1/2 to 5-1/2 months of service life.

Since the panels for both the Patrick and Fairfax County bridges were wider at the ends than in the middle when installed, one could easily assume that they absorbed moisture and swelled after being fabricated to a uniform width. Since moisture can enter the ends of the panels more readily than the center portion, one would expect that the moisture content and swelling would be greatest on the ends (see Figure 10). As can be seen from Table 5, there was a tendency for the panels to have a higher moisture content at the edges.

The panels for the Henry County bridge were true in shape and satisfied AITC Voluntary Product Std. PS 56-73 at the time of erection, as would be expected based on the low moisture contents shown in Table 5.

During a field inspection on November 1, a significant number of cracks and splits as shown in Figure 11 were observed in the surface of the panels in the Fairfax County bridge. Consequently, the panels were rated with respect to the number of cracks and splits and the ratings were plotted against the moisture content data, which was collected on the same date. The panels were rated from 1 to 3 with a rating of 3 being assigned to a panel exhibiting a large number of cracks and splits. It's obvious from the data, which are shown in Figure 12, there is a correlation between moisture content and the amount of cracking and splitting in the panels. Panels with few cracks exhibited a low moisture content and panels with many cracks exhibited a high moisture content. It's reasonable to expect that the panels with a high moisture content developed cracks in proportion to the hydrostatic pressure which occurred during the creosote treating process.

Figure 11. Cracks and splits in panels on Fairfax County bridge.

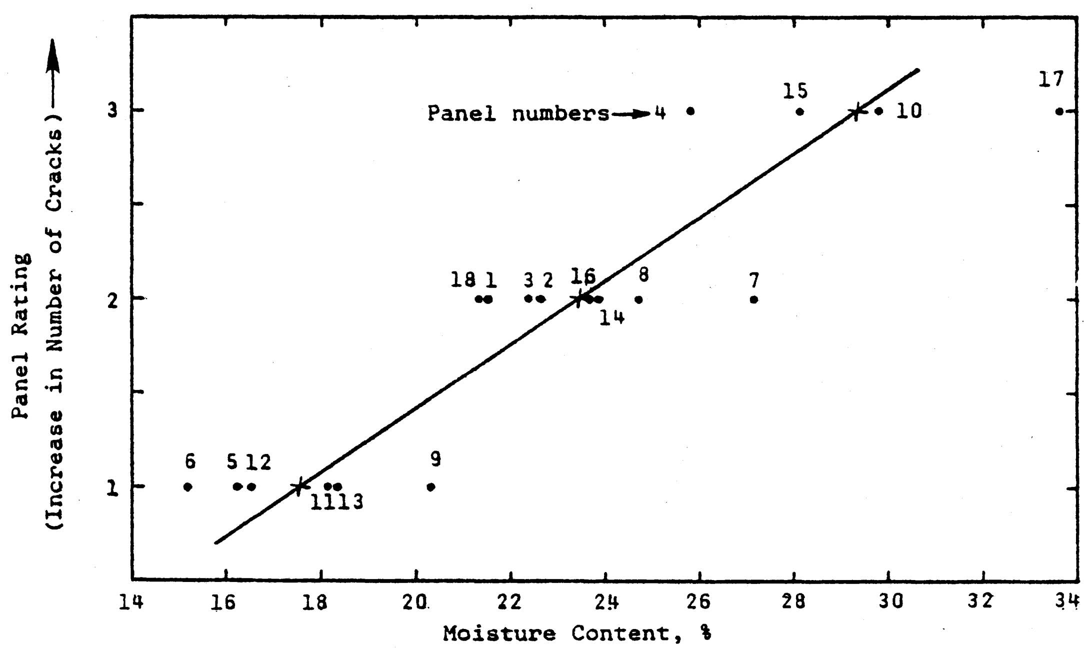

Figure 12. Moisture content versus panel rating for Fairfax County bridge.

The moisture content data collected to date are interesting and controversial to the point that it is believed these panels should be monitored closely over the next few years. The cracks in the panels should retain water and therefore create a high moisture content environment, which should prove detrimental to the performance of the panels.

FURTHER CONSIDERATIONS

Wearing Surface

Because of the excess creosote bleeding from the deck panels on each of the three glulam bridges, the wearing surface was not applied to the decks until they were exposed to summer temperatures for two or more months. The Fairfax County bridge was subjected to traffic for approximately 6 months before the overlay was placed; the Patrick County bridge for approximately 4 months; and the Henry County bridge for approximately 2 months. During the bleeding period sand was applied to the surface of the decks as needed to absorb the excess creosote and improve the skid resistance. Traffic gradually removed most of the sand and the panels were reasonably free of creosote at the time the wearing surfaces were applied.

A contractor paved the approach roadway and the deck of the Henry County bridge during the first week in October, and maintenance forces placed the wearing surface on the Patrick County bridge during the last week in October. Prior to placing each S-5 wearing surface with paving equipment, compressed air was used to remove dirt, sand, and debris from the panels and a CRS-2 tack was applied as mist. The wearing surfaces were applied at a variable thickness of approximately 4 in. along the center line tapering to approximately 2 in. along the rails. A steel-wheel roller followed the paver and compacted the bituminous concrete to a variable thickness which measured approximately 3 in. along the center line and tapered to approximately 1 in. along the rails.

During a field inspection on November 2, just 4 weeks after the wearing surface was installed, cracks were observed in the wearing surface on the Henry County bridge. Most of the cracks were located directly above the joints between the glulam panels but several cracks were also noted in other areas. The cracks differed in width but all ran parallel to the panel joints. Most of the cracks were hairline but a large crack, which is shown in Figure 13, had occurred at midspan. A quarter is shown next to the crack in Figure 13 to provide an indication of the width of the crack.

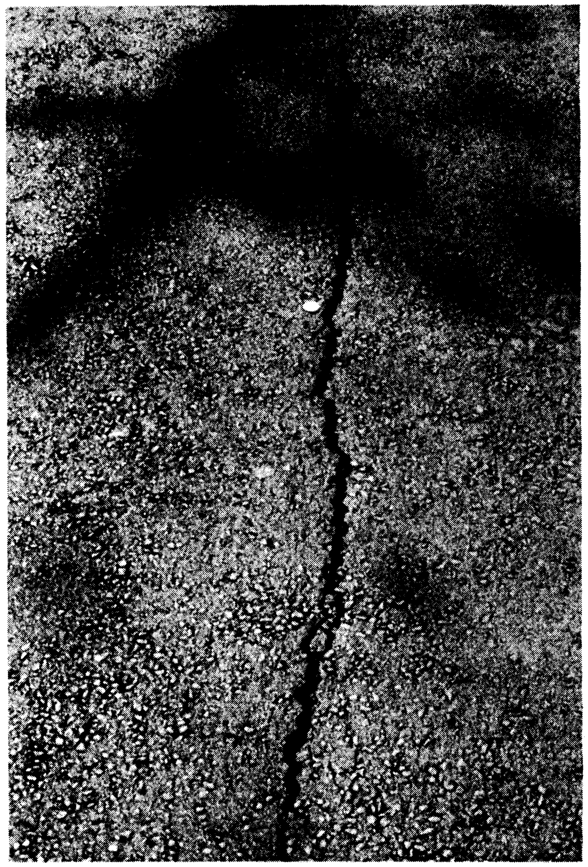

Figure 13. Crack in overlay on Henry County bridge.

It is not known when the cracks began to form but based on an estimated 1,500 vpd traffic count the overlay had carried 42,000 vehicles at the time the cracks were observed. The exact cause of the cracks is also uncertain; however, because the dowels between the panels fit tightly at the time of construction, it is believed that differential deflection between the panels is negligible and therefore not responsible for the cracks.

A more reasonable explanation for the cracks is that the tensile strain in the bituminous concrete was excessive. Maupin has shown that the fatigue life of a typical plant mix used in Virginia is a function of the bending strain to which the mix is subjected and that a bending strain of 0.001 can cause failure after 12,500 cycles of loading.(11)

The geometry of the standard glulam bridges is such that it is theoretically possible to produce a strain in the overlay of 0.001 with an HS20-44 loading. Although the stringers are significantly understressed when subjected to the design loading the top fiber of a 46-ft. stringer can experience a strain of 0.0005. Assuming a linear strain distribution in bending, the 0.0005 strain in the top flange of the stringer produces a strain of 0.001 in the overlay. Since the timber bolsters are connected to the stringers with bolts and the glulam panels are connected to the bolsters with spikes, it is likely that the timber in the vicinity of the spikes and bolts is compressed under loading and the bolsters and panels slip in the horizontal direction and relieve some of the strain in the panels and bolsters. Also, since there is no mechanism for transferring the tensile stress between adjacent panels, the joints between the panels open when the load is removed. If it is assumed that there is some bond between the panels and the overlay, it is likely that the strain in the overlay is concentrated at the panel joints. A calculation reveals that a uniform strain of 0.001 in the overlay can be relieved by a horizontal slip of 0.05 inch at 4 ft. intervals. The relief is provided by the formation of a crack in the overlay. It is conceivable that hairline cracks were initiated at the time the overlay was compacted, since a typical 10-ton steel-wheel roller can produce strains equal to about two-thirds the strain produced by an HS20-44 truck.

It is anticipated that water will enter the cracks and migrate between the panels and the overlay, and the freezing of the water combined with the traffic loading will cause the overlay to spall in a short time. Filling the cracks with anything other than a very flexible material will result in the formation of new cracks, which will lead to spalling. It is believed that the bituminous concrete overlays as installed will not perform satisfactorily.

On future bridges it might be possible to reduce or eliminate the incidence of cracking in the overlay by installing a fabric between the overlay and the deck panels. The fabric would serve to distribute the strain in the overlay caused by the horizontal movement of the panels at the panel joints. Other measures that should reduce the formation of cracks in the overlay include eliminating the timber bolster and increasing the depth or stiffness of the stringers.

A surface treatment should perform better than a plant mix as it has a tendency to bleed during warm weather and therefore would tend to seal any cracks that form.

Maintenance forces applied a surface treatment to the deck of the Fairfax County bridge the first week in December. A surface treatment was used rather than bituminous concrete because of the experience with cracking of the bituminous concrete overlay on the Henry County bridge.

Connection Details

Tests by the Forest Products Laboratory have shown that steel dowels similar to those used in the three glulam bridge decks in Virginia provide the best mechanism for the transfer of moment and shear stresses between panels.[10] Simplified connection details such as a tongue and groove joint or a nail laminated joint do not provide adequate load transfer. However, considerable care must be exercised during the fabrication of the panels to ensure that the dowels will align properly during placement. It is believed that the extra care that must be exercised is partly responsible for the high cost of the panels. Dowels which don't fit properly can delay the placement of the panels. Also, if a panel is damaged in service, it may be necessary to remove many panels to get to the damaged one. Fabrication, erection, and removal of the panels would be simplified if the dowels could be eliminated.

A proprietary clip angle device for connecting the panels to either glulam or steel stringers has been developed by Weyerhauser.[8] Tests of a prototype bridge in which the clip angles were used to connect glulam panels and glulam stringers have indicated that because of the clips, differential deflection between panels was not great enough to cause detrimental cracks in the bituminous overlay.[8] It is believed that this system should be considered if it can be shown that the clip angles significantly reduce the cost of the panels.

Panel Thickness

Figure 14 shows the relationship between panel thickness and stringer spacing based on AASHTO formulas for moment and shear and the maximum stresses allowed by AASHTO for Douglas fir and Southern pine glulam panels for both the dry stress and wet stress conditions. The figure shows that for Douglas fir the shear stress controls the thickness of the panels for almost all stringer spacings, while for the Southern pine the shear stress controls the thickness for stringer spacings less than about five feet. Efforts to optimize the spacing of stringers for a given deck thickness are fruitless since the shear stress is not proportional to the stringer spacing. On the other hand, if the moment stress controlled the panel thickness, the stringer spacing could be optimized since moment stresses are a function of the stringer spacing.

Recent research conducted on glulam panels has resulted in a proposed revision to the AASHTO Standard Specification for Highway Bridges which would allow the designer to assume that the horizontal shear produced by a wheel load acts over the full width of a panel.[12] The proposed revision is in agreement with tests on Press-lam timber conducted by the Forest Products Laboratory, which showed that a deck thickness of 3-1/2 in. was adequate for shear with a stringer spacing of 28 in., whereas the current AASHTO formula for glulam requires a panel thickness of 6.1 in.[13] If the proposed revision is accepted by AASHTO, a glulam panel which is 48 in. wide and 3 in. thick will be adequate with respect to horizontal shear and panel thickness will be governed by moment stress and, therefore, can be optimized with respect to stringer spacing.

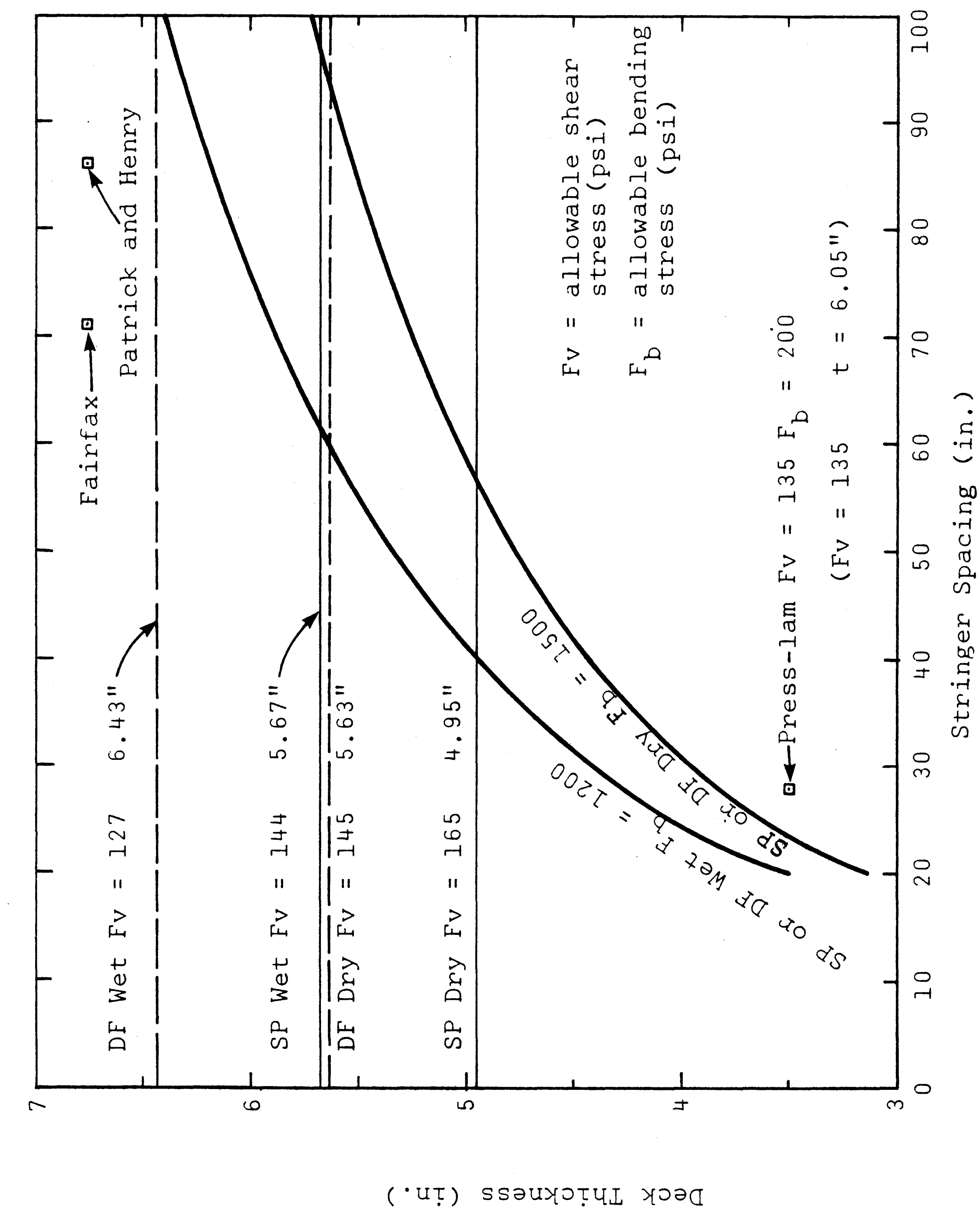

Figure 14. Deck thickness vs. stringer spacing for HS 20-44 loading.

The panels are available in only 3 standard thicknesses — 3 in., 5-1/8 in., and 6-3/4 in. A 5-1/8 in. thick panel should be about 20% cheaper than a 6-3/4 in. thick one. As shown in Figure 14, a 5-1/8 in. thick Southern pine panel would be adequate for a stringer spacing of 5 ft. or less and for a dry stress condition. However, because of the high moisture content of the Southern pine panels used in Virginia it is obvious that quality control standards must be enforced if the designed is to be in a position to assume a dry stress condition. Further research with respect to moisture content or shear stress which would lead to a reduction in panel thickness would make the panels more economical.

Glulam Stringers

Based on the first three glulam decks to be constructed in Virginia, glulam panels have proven to be costly when compared to solid, sawed plank flooring. On a board foot basis the panels cost three times as much as the plank. Based on private communications with two panel fabricators it is believed the high cost of the panels is due to the high cost of the labor and overhead necessary to maintain the quality control necessary in fabricating them. This cost must be added to the cost of the raw lumber, which is about the same as for the solid sawed plank. In addition there is some waste with the panels since the 2 in. x 8 in. timbers from which they are made must be planed to a thickness of about 6-3/4 in. to achieve a flat surface.

The significant benefits of the laminating process are not fully realized with the panels since they are loaded parallel to the plane of the glue lines. Bridge stringers, on the other hand, would be designed primarily for a loading perpendicular to the plane of the glue lines. The AASHTO allowable bending and shear stresses are approximately 20% greater for a loading perpendicular to the plane of the glue lines than for a parallel loading as can be seen in Table 6. In addition, if it is possible to have a dry stress condition in a bridge superstructure it is more likely to be obtainable in the stringer than in the deck panels. When one considers the possibility that a dry stress condition can be combined with a loading perpendicular to the plane of the glue lines in a bridge stringer, whereas in a deck panel a wet stress condition is combined with a loading parallel to the glue lines, it can be seen that glulam material can be used more efficiently as a stringer than as a deck. As can be seen from Table 6, allowable stresses of as much as 50% greater can be typically achieved. In addition, a hybrid stringer having an allowable bending stress of 2,600 psi can be obtained, which represents a 117% increase in the 1,200 psi bending stress which must be assumed for a deck panel having a moisture content greater than 16%. Consequently, it is felt that consideration should be given to constructing a bridge with glulam stringers.

Table 6

Allowable Unit Stresses for Stringers and Deck Panels (psi)

Type Wood	Panels Bending $\parallel$		Stringers Bending $\perp$		Ratio $\perp/\parallel$	Panels Shear $\parallel$		Stringers Shear $\perp$		Ratio $\perp/\parallel$	Ratio $\perp$ Dry / $\parallel$ Wet[b]	
	Dry	Wet	Dry[a]	Wet		Dry	Wet	Dry	Wet		Bending	Shear
Douglas Fir	1500	1200	1800	1440	1.20	145	127	165	144	1.13	1.50	1.30
Southern Pine	1500	1200	1800	1440	1.20	165	144	200	175	1.22	1.50	1.39

[a] Stringers with allowable bending stresses up to 2,600 psi can be obtained (2600/1200 = 2.17).

[b] Moisture content data suggest use of wet stress condition for deck panels, whereas it may be appropriate to use the dry stress condition for stringers and achieve a 30% to 117% increase in allowable stress for stringers as compared to deck panels. With glulam, economy may be in stringers rather than deck panels.

Stress Design for Stringers

Partly because the glulam deck is not composite with the supporting stringers, the size of the stringers is determined by the AASHTO requirements with regard to deflection. The stringers are understressed by about 40%, and when weathering steel is used for the stringers so as to eliminate the need for painting, the difference between the allowable stress and actual stress is about 60%.[14] Considerable savings in structural steel could be achieved if the stringers could be designed for allowable stress rather than for allowable deflection. However, with an increase in allowable deflection, there will be an increase in vibration from traffic and an increase in the likelihood that the connections will work loose and cracks will form in the overlay. But, the opportunity to reduce the cost of the structural steel is sufficient justification for further research in this area.

CONCLUSIONS

1. A system of glulam panels and steel stringers can be installed about 45% faster than can a more conventional system consisting of steel stringers and solid, sawed timber planks. The glulam system can be assembled faster because there are fewer components to assemble.

2. Traffic can be maintained as the deck panels are installed.

3. Glulam deck panels on steel stringers cost more than solid, sawed timber plank on steel stringers. The additional cost amounts to 40% for a 20-ft. span but decreases to 10% for spans of 60 ft. Glulam deck panels cost 250% more than solid sawed plank, but costs of structural steel and construction labor are less for the glulam system. For longer spans, a reduction in maintenance costs may justify the additional cost of the glulam system.

4. The use of A588 structural steel reduced labor costs about 20% over that required for steel which must be painted.

5. Bridges should be designed to accomodate the variation in panel width of - 1/4 in. to + 1/2 in. that will normally be encountered. Variations in panel width can be easily accommodated by adjusting the position of one backwall and by casting the backwall after the panels are installed.

6. Panels should be designed for a wet stress condition unless it is specified that the panels have a moisture content of 16% or less at the time they are treated with creosote.

7. Excess creosote bleeding from the panels is undesirable during installation and detracts from the aesthetics of the structure. Painting of the structural steel is difficult. If creosote is used, clean panels should be requested from the fabricator. CCA treatments should be considered.

8. A bituminous concrete overlay will develop a crack above the joint between glulam panels.

RECOMMENDATIONS

1. Specify glulam deck panels rather than solid, sawed plank when circumstances suggest that the reduction in on-site construction time and anticipated reductions in deck maintenance will justify the higher cost of the panels.

2. To facilitate the erection of the panels, request that the panels be as clean as practical when delivered or consider specifying a CCA type treatment.

3. Although the timber bolsters provide for the rapid installation of the glulam panels, they should be eliminated in the future as they are expensive and detract from the structural integrity of the bridge.

4. Accommodate variations in the width of the panels by constructing one backwall after the panels are installed and by positioning the backwall to fit the panels.

5. When practical, specify A588 rather than A36 steel for stringers.

6. Assuming a favorable cost estimate can be obtained, construct a prototype structure in which the dowels are eliminated and the panels are connected to the stringers with special clips.

7. Give consideration to using glulam as a stringer material.

8. Over the next several years, closely monitor the moisture content of the panels used on the first three bridges so that the in-service moisture condition can be firmly established for use in design.

9. To reduce structural steel costs, consider increasing the allowable deflection or stiffening the stringers.

10. To minimize the required thickness of the deck panels, consider revising the AASHTO formula used to compute the shear stress in the panels.

11. To minimize the formation of cracks in the overlay consider using a fabric between the overlay and the panels or increasing the stiffness of the stringers. Because of its inherent self-sealing properties, consider specifying a surface treatment rather than bituminous concrete for the overlay.

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REFERENCES

1. Brown, H. E., "Working Plan — Industrialized Timber Structures", VHTRC 72-WP7, September 1972.

2. "Typical Timber Bridge Design and Details", prepared for American Institute of Timber Construction by Hurlbut, Kersich, and McCullough Consulting Engineers, Billings, Montana, July 1973.

3. Rte. 603 over Parting Creek, Plans — Commonwealth of Virginia, Department of Highways and Transportation, February 1975.

4. "A Standard for Bridges — Steel Beams with Glulam Flooring", Virginia Department of Highways & Transportation, Richmond, Virginia, July 1976.

5. Rte. 641 over Pohick Creek, Plans — Commonwealth of Virginia, Department of Highways and Transportation, April 1977.

6. "Standard Steel Beam Bridges", Virginia Department of Highways and Transportation, Richmond, Virginia, September 1975.

7. Sprinkel, M. M., "In-house Fabrication of Precast Concrete Bridge Slabs", VHTRC 77-R33, December 1976.

8. Hale, Charles Y., "Field Test of a 40-Ft. Span Two-Lane Weyerhaeuser Panelized Wood Bridge", Report No. RDR-045-1092, May 1975.

9. "Koppers Pressure Treatments Help Wood Serve You Longer", FP-902, Koppers Company, Inc., Pittsburgh, Pennsylvania, 1977.

10. "Glulam Bridge Systems Plans and Details", American Institute of Timber Construction, Englewood, Colorado, 1974.

11. Maupin, G. W., Jr., and J. R. Freeman, Jr., "Simple Procedure for Fatigue Characterization of Bituminous Concrete", FHWA-RD-76-102, Virginia Highway and Transportation Research Council, 1976, p. 81.

12. Stone, Marlyn F., "New Concepts for Short Span Panelized Bridge Design of Glulam Timber". Paper presented to Forest Products Research Society, Atlanta, Georgia, June 26-30, 1978.

13. Youngquist, John A., and David S. Gromala, "Press-Lam Timbers for Exposed Structures", ASCE Spring Convention, March 1978.

14. Woodward, C. L., Computer printout for Standard Glulam Deck Steel Stringer Bridges.