Field Investigations of Stress-Laminated T-Beam and Box-Beam Timber Bridges

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Abstract

Field investigations of selected stress-laminated T-beam and box-beam bridges in West Virginia showed that several bridges had cracks in their asphalt wearing surfaces at the web-to-flange interface and lower than expected levels of stressing bar forces and interlaminar compressive stresses. This indicated possible slip between webs and flanges resulting in lack of full composite action and levels of structural safety lower than what was assumed by the designers. While the bridges continue to carry vehicle traffic, the inspection program found several areas of concern that will affect their long-term ability to safely carry vehicle loads in a cost-effective manner.

Introduction

Stress laminating is one of the newest techniques used in modern timber bridge construction. Stress-laminated deck superstructures consist of a series of lumber laminations that are placed edgewise and are transversely compressed with high-strength prestressing bars to create large structural assemblies. In contrast to longitudinal glued-laminated timber (glulam) deck assemblies and nail-laminated assemblies, which achieve load transfer among laminations by structural adhesives or mechanical fasteners, the load transfer between laminations in stress-laminated bridges is developed through compression and interlaminar friction. This interlaminar friction is created by the same type of high-strength steel stressing elements typically used in prestressed concrete.

Findings from early research efforts and funding of the USDA Forest Service's Timber Bridge Initiative opened the door for the construction of many stress-laminated deck bridges in the United States. The early stress-laminated deck bridge designs had simple rectangular cross sections composed of longitudinal sawn lumber laminations with transverse prestressing (i.e., slab type systems). These designs proved relatively successful in short span applications and they were relatively cost effective. However, to meet the need for longer spans, researchers (Dickson 1995) developed new superstructure designs that consisted of stress-laminated box-beams and T-beams (Fig. 1). The webs formed by deep beams significantly increase the stiffness of the bridge deck, thereby making longer spans possible. Although initial predictions for these stress-laminated T-beam and box-beam bridge designs were encouraging, questions about long-term performance of the bridge systems arose during further field testing. Several investigators have reported detailed performance results for these types of T-beam and box-beam bridges in the United States (Dickson 1995; Wacker et al. 1998; and Kainz 1998). Performance data considered typically consisted of stressing-bar forces, wood moisture contents, results from static load tests, and observations from visual inspections.

Objectives

Specific objectives of the inspection program were to identify and evaluate specific field performance characteristics and trends for stress-laminated T-beam and box-beam bridges constructed in the United States. Several bridges in West Virginia were visually inspected for signs of distress. Stressingbar forces, indications of interlaminar slip, performance of the wearing surface, and other bridge characteristics were evaluated.

Evaluation Methodology

The bridges were inspected by a group of personnel from Auburn University, the University of Alabama, and the USDA Forest Products Laboratory. The field evaluation techniques used in the investigation, which are described next, follow general procedures outlined in reports cited previously and in Ritter *et al.* (1991). However, no load tests were performed in this study

Condition Assessment

A total of 18 bridges were visited by the group during September 1997. Inspections were conducted on eight T-beam bridges and ten box-beam bridges. The names and locations of the bridges are listed in Table 1. Each bridge was visually inspected and sketches were made listing dimensions of the bridge. Data recorded on each bridge included bridge width, bridge span, web sizes and spacings, and flange sizes. In addition to dimension data for the bridges, occurrences of creep and possible interlaminar slip were noted where present. Condition of the wearing sur-

Table 1.—List of bridges inspected in West Virginia.

Bridge name and number	Bridge type	Year built	Span	Width	Flange depth	Location
			--------- (m) [ft.] -------		(mm) [in.]	(county)
1. Barlow Drive	T	1989	22.3[73.3]	5.1 [16.8]	229 [9]	Kanawha
2. Hope Station	T	1994	2@11.6 [2@38.0]	7.8 [25.5]	229 [9]	Lewis
3. Lightburn	T	1993	2@18.1 [2@59.5]	6.4 [21.1]	229 [9]	Lewis
4. Fieldcrest	Box	1990	12.3 [40.5]	8.8 [28.9]	178 [7]	Monogalia
5. PovertyRun	Box	1994	16.7 [54.7]	6.5 [21.4]	229 [9]	Doddridge
6. Bonds Creek	Box	1990	11.7 [38.4]	5.6 [18.3]	178 [7]	Ritchie
7. Claylick Run	Box	1990	12.8 [41.9]	6.7 [22.1]	178 [7]	Jackson
8. Upper Five Mile No. 2	Box	1991	12.6 [41.5]	5.5 [18.1]	178 [7]	Mason
9. Upper Five Mile No. 1	Box	1990	12.5 [41.0]	5.6 [18.2]	178 [7]	Mason
10. Camp Arrowhead	T	1992	19.3 [63.5]	7.3 [23.8]	178 [7]	Cabell
11. Trace Fork	T	1993	13.644.51	6.5 [21.3]	229 [9]	Putnam
12. Sixmile Creek	T	1993	7.6 [25.0]	7.8 [25.5]	229 [9]	Boone
13. Little Huff	Box	1990	13.9 [45.5]	6.3 [20.7]	229 [9]	Wyoming
14. Steeles	Box	1991	16.4 [53.7]	6.0 [19.8]	178 [7]	Wyoming
15. Little Stoney Creek	Box	1994	10.8 [35.5]	7.7 [25.2]	229 [9]	Summers
16. Mash Fork	Box	1991	19.3 [63.4]	7.3 [24.0]	229 [9]	Mercer
17. Left hand Run	T	1992	15.8 [52.0]	7.3 [23.8]	229 [9]	Roane
18. Nebo	T	1992	10.2 [33.5]	6.5 [21.4]	229 [9]	Clay

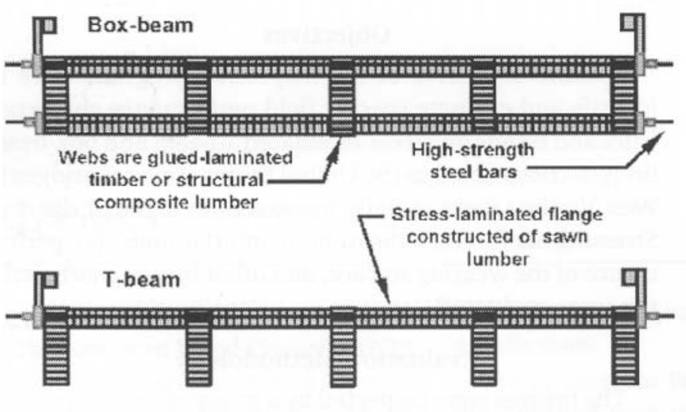

Figure 1.—Section views of stress-laminated box-beam and T-beam bridge systems.

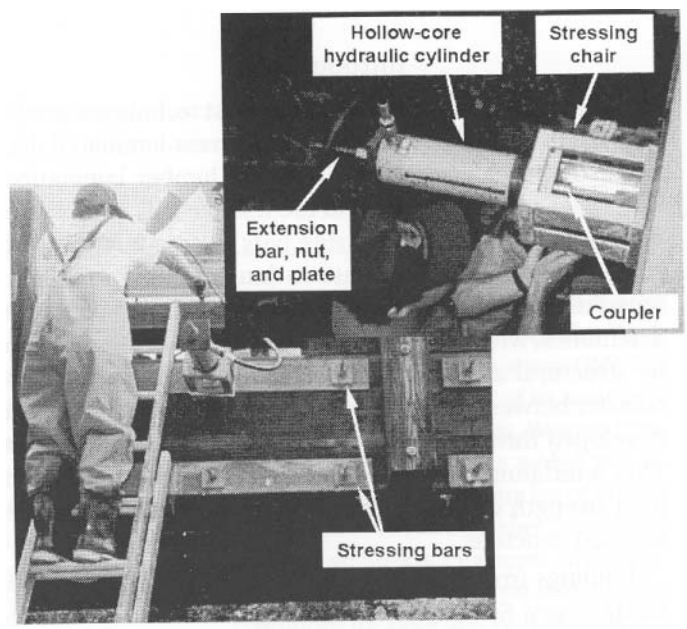

Figure 2.—Determining stressing bar force on a typical stress-laminated box-beam bridge.

faces and other performance characteristics were also evaluated. Photographs were taken of each bridge with particular emphasis placed on potential problems with the bridges.

Bar Force and Interlaminar Compressive Stress

Stressing bar forces were checked on most of the bridges. However, the stressing bars on some bridges were inaccessible, and one bridge had the ends of the stressing bars cut off, thereby making it impossible to check its bar forces. On bridges where bars were accessible, bar forces were checked using a stressing chair, hydraulic cylinder, coupler, extension bar, and nut as shown in Figure 2. Pressure was gradually applied to the cylinder until the anchorage plate loosened. At this point, the pressure reading from the calibrated hydraulic pump was recorded. This pressure value was then multiplied by a calibration factor to determine the force in the stressing bar. Typically, a selected group of bars near each end and near the midspan of the bridge was selected. Once the stressing bar force was determined, the interlaminar compressive stress at each bar was determined by dividing bar force by bar spacing and flange depth.

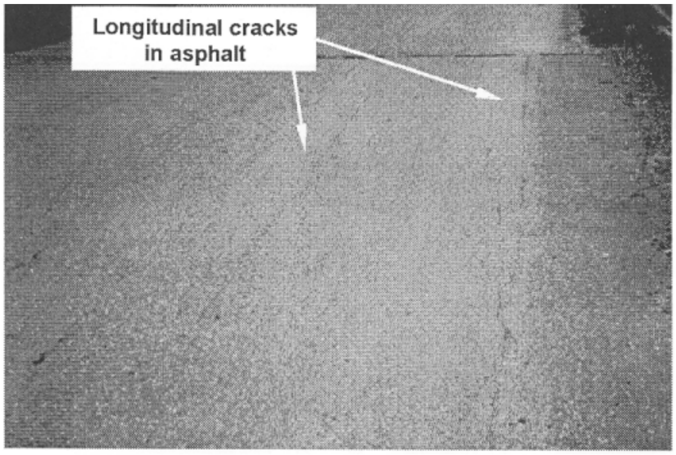

Figure 3.—Example of longitudinal cracks in the asphalt wearing surface of the Six Mile Creek bridge. Location of cracks corresponded to location of webs.

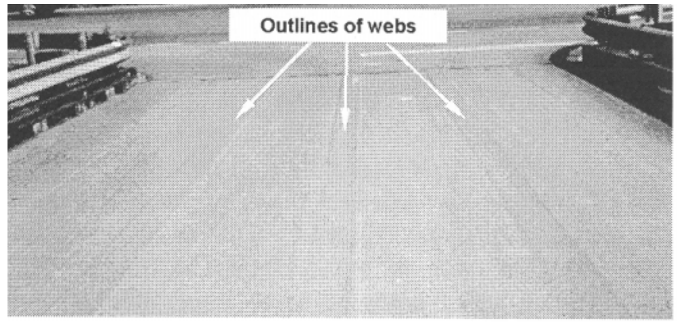

Figure 4.—Examples of web outlines that were visible in the asphalt wearing surface of the Hope Station bridge.

Moisture Content

Moisture content levels of bridge components were checked on most bridges using an electrical resistance moisture meter and by removing core samples from the underside of selected webs and flanges. In the case of box-beam bridges, flange measurements were taken only from the lower flange. Sampling areas were located approximately 1.5 to 3 m (5 to 10 ft.) from one of the bridges' abutments and were concentrated in a circular area with a radius of approximately 1.5 m (5 ft.).

Resistance meter readings were taken with a Delmhorst Model RDM-1 moisture meter with two prong electrodes approximately 32 mm (1.25 in.) in length. Electrodes were driven into the wood to their full length. Meter readings were adjusted with appropriate temperature and wood species correction factors. An increment borer was then used to remove core samples from webs and flanges in the same locations were the resistance measurements were taken. Holes in webs and flanges were plugged with wood dowels that were preservative treated with copper naphthenate. Core samples measured approximately 5 mm (0.2 in.) diameter by 152 mm (6 in.) long. Upon removal, cores were placed in plastic bags, which were then sealed until they could be tested in a laboratory setting. Cores were tested in accordance with ASTM D143 (ASTM 2001) to determine their moisture content.

Results and Discussion

Detailed notes for each bridge inspected are given in Taylor *et al.* (2000). These notes included sketches of each bridge superstructure, dimensions, general notes about the bridge, bar force data, and moisture content data. In all observations, condition of skewed bridges appeared to be no different from those without skew.

In general, all T-beam and box-beam bridges exhibited similar design and construction details and followed design recommendations developed by the West Virginia Division of Highways. On some of the T-beam bridges, extra sets of stressing bars were installed below the flanges to correct distorted modules and plumb the webs during construction.

Asphalt Cracking and Possible Lamination Slip

Perhaps the most immediate feature noted on many of the bridges was longitudinal cracks in the asphalt wearing surfaces as shown in Figure 3. Cracks were observed at locations that were immediately above the webs, in both T-beam and box-beam bridges. This feature was observed in seven T-beam bridges and five box-beam bridges. Most cracks were longitudinal; however, on two bridges, smaller transverse cracks in the asphalt were also observed. While these cracks could have resulted from reflective cracking due to fluctuations in wood moisture content, much of the cracking is believed to be due to slip between webs and flanges. The presence of longitudinal cracks in wearing surfaces has not been noted in previous performance reports on T-beam or box-beam bridges (Wacker *et al.* 1998, Kainz 1998). On four of the bridges that did not have cracks in their asphalt (Upper Five Mile No. 1 and No. 2, Steeles, and Little Huff), new asphalt had been applied to the bridges, so it is very possible that the original wearing surface may have had similar cracks before repaving.

Outlines of webs were visible in the asphalt wearing surface on several bridges, which may indicate that the flanges had slipped since the wearing surface was paved. Figure 4 is a photograph showing outlines of webs on the Hope Station bridge. In the case of one box-beam bridge (Six Mile Creek), visible slip between the web and top flange of the bridge was observed when a heavy truck passed over the bridge. Several bridges had butt joints in the flanges that were misaligned in the horizontal plane. These misaligned joints could have been the result of slip between laminations. On one T-beam bridge (Nebo), the bottom of the flange was in contact with the top of one of the diaphragms, while a 50 to 100 mm (2 to 4 in.) gap between the flange and diaphragm occurred at other diaphragms. This was further indication of possible slip in the flange. Since asphalt cracking and lamination slip has not been addressed in the literature, it may be a feature that has appeared in bridges that were not in the USDA Forest Service monitoring program, and it may

Table 2.–Moisture content data[a] from core samples taken from 18 stress-laminated T-beam and box-beam bridges inspected in West Virginia.

	Number of samples	Mean	Median	Coefficient of variation	Maximum	Minimum
		(%)				
Flange[b]						
Box-beam bridge	18	22.3	21.5	30.9	34.5	12.9
T-beam bridge	9	22.4	23.6	23.5	28.1	12.4
All bridges	27	22.3	22.1	28.1	34.5	12.4
Web[c]						
Box-beam bridge	18	19.0	19.6	27.2	26.5	6.7
T-beam bridge	10	18.4	18.5	10.0	21.2	15.2
All bridges	28	18.8	18.8	22.6	26.5	6.7

[a] Moisture content is expressed on a dry-weight basis.
[b] Flanges were constructed using red oak lumber.
[c] Webs were constructed using southern pine glued-laminated timbers.

also be a feature that does not manifest itself until later in bridge life. At any rate, the possibility of slip between webs and flange laminations is an indication that complete composite behavior is not occurring in the bridges. This would indicate that bridges do not have the levels of stiffness or strength for which they were originally designed.

Interlaminar Compressive Stress

Stressing bar forces and the resulting levels of interlaminar compressive stress were generally within design parameters. However, there were cases where bar forces fell below those levels deemed acceptable for adequate bridge performance. All bridges were designed for an initial level of interlaminar stress of 689 kPa (100 psi). Design guidelines allow for a 60 percent loss in bar force (ie., stress relaxation) before a restressing operation is recommended. This results in a lower limit of "allowable" interlaminar stress of 276 kPa (40 psi).

The overall mean interlaminar stress for the box-beam bridges was 531.2 kPa (77.0 psi) with a coefficient of variation of 28.7 percent. For a total of 60 bars checked, the median interlaminar stress for box-beam bridges was 531.7 kPa (77.1 psi) while the maximum and minimum interlaminar stresses were 890.6 kPa (129.2 psi) and 114.9 kPa (16.7 psi), respectively Ten percent of the bars checked had interlaminar stresses below the minimum "allowable" level.

The overall mean interlaminar stress for the T-beam bridges was 422.2 kPa (61.2 psi) with a coefficient of variation of 33.4 percent. For a total of 27 bars checked, the median interlaminar stress for the T-beam bridges was 415.0 kPa (60.2 psi), while the maximum and minimum interlaminar stresses were 759.7 kPa (110.2 psi) and 229.8 kPa (33.3 psi), respectively. Eleven percent of the bars checked had interlaminar stresses below the minimum "allowable" level.

As previously discussed, many bridges had problems with cracked asphalt wearing surfaces. There were a total of 12 out of 18 bridges that exhibited cracks in their wearing surfaces. In addition, there were nine bridges that had what should be considered low bar forces; i.e., at least one bar out of those checked was below the allowable level of bar force as described previously Of these nine bridges with "loose" bars, seven bridges had problems with asphalt cracking, one had problems with asphalt rutting, and one had been recently repaved. Therefore, there appears to be a strong relationship between low bar forces and cracked asphalt wearing surfaces.

In several cases, the bar force was low enough that nuts on the stressing bars could be turned with a wrench or by lightly tapping them with a hammer. It appeared that newer bridges had lower bar forces than older bridges. This may have been due to restressing activities that have been carried out on older bridges and not on newer bridges. On one bridge observed, the ends of the bars had been cut off after construction, thereby preventing the possibility of attaching equipment to restress the bridge.

Moisture Condition of Wood Components

Moisture content (based on ovendry weight) data for the bridges are listed in Table 2. This table contains data for both types of bridges summarized by samples taken from flanges or webs. There was little correlation between resistance moisture meter readings and moisture content determined by oven tests, therefore only oven test results are provided in Table 2. Some of the difference in results may be due to the fact that the core samples represented a 152 mm (6 in.) deep cross section of wood, while the resistance meter probes only contacted wood at a depth of 32 mm (1.25 in.). The creosote treatment also may have affected moisture meter readings.

Overall mean moisture content, from oven test data, for flanges was 22.3 percent with a coefficient of variation of 28.1 percent. Overall mean moisture content, from oven test data, for webs was 18.8 percent with a coefficient of

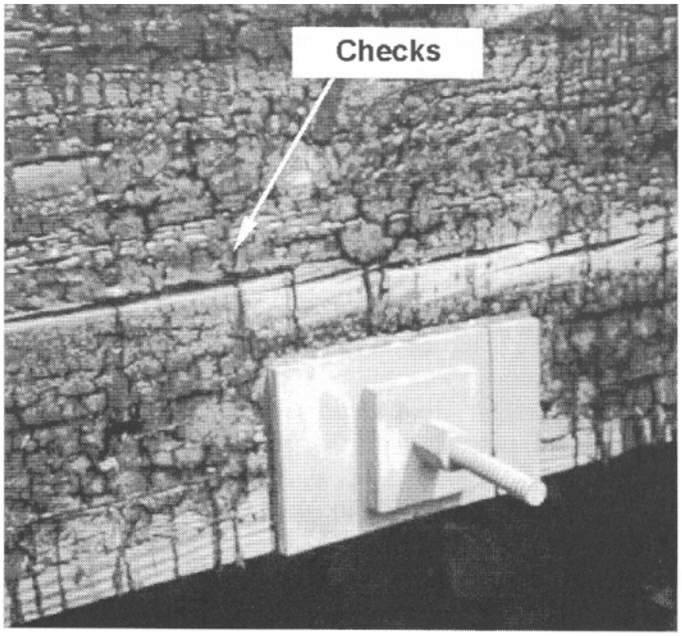

Figure 5.—Example of large horizontal checks in an exterior web of the Bonds Creek box-beam bridge.

variation of 22.6 percent. Maximum moisture contents of 34.5 percent and 26.5 percent were observed for flanges and webs, respectively. There were few differences in moisture data from T-beam or box-beam bridge components. Median moisture contents for flanges and webs were 22.1 percent and 18.8 percent, respectively. Therefore, nearly half of the samples had moisture contents that were higher than 20 percent. Above this moisture level, decay problems may begin to occur in untreated wood, given favorable temperature and oxygen conditions. Since these core samples were taken from regions inside the components where the preservative may not have fully penetrated into the wood, these components may eventually experience problems with decay. Also, higher moisture contents will result in reductions in stiffness and strength of the wood components. However, the item to be most concerned with is the fact that there will likely be a corresponding loss in bar force and interlaminar stress as the moisture content of these wood components changes.

As expected, glulam webs had lower moisture contents than flanges, since glulam components would have been fabricated at moisture contents near 12 percent. However, since the median moisture content of glulam components was 18.8 percent with a maximum of 26.5 percent, it appears that the glulam components are gaining moisture in service. This increase in moisture may be aided by the presence of checks in the webs and by periodic flooding and subsequent inundation of the bridges. There was little difference between moisture contents of interior and exterior samples indicating that the asphalt wearing surfaces are not providing a dry environment for the webs.

In several cases, water was observed dripping from lower flanges on box-beam bridges. These drips were seen at weep holes in the flanges, but also at locations of butt joints. In one case, water was observed dripping from the core sample when it was removed. On one bridge, water was observed dripping from the horizontal stressing bar hole when the bar force was checked. These observations help explain why some of the moisture content data were relatively high.

During visual inspection of some of the bridges, dried mud and other debris were noticed on bridge components, which would indicate that bridges had been inundated by flood waters at some point. The possibility that bridges had been submerged would also help explain high wood moisture contents. Unfortunately, no data were available to indicate when flooding might have occurred; therefore, it is difficult to draw conclusions on high wood moisture conditions as a result of flooding.

Checking of Wood Components

Checking was observed on the outside face of exterior webs of many box-beam bridges and on some of the T-beam bridges near abutments. Figure 5 shows examples of these checks in a box-beam bridge. These checks were typically below upper bearing plates and above lower bearing plates. In the most extreme case, the check was 13 mm (0.5 in.) wide and 25 mm (1 in.) deep. Dickson (1995) reported that these checks may be related to problems with unsquare box-beam modules. However, field performance results for bridges in other regions of the United States have not mentioned this problem (Wacker *et al.* 1998, Kainz 1998). This situation, which developed during the stressing process, resulted in uneven compression of the box-beam modules. Uneven compression led to transverse bending and tension perpendicular-to-grain stresses in the webs. If webs experienced tension perpendicular to grain stress (with stressing bars applying lateral loads near the top and bottom of the web), this stress may have been a contributing cause of large checks in the outside faces of webs.

In the case of one of the T-beam bridges with extra stressing bars, checks were observed on interior faces of the webs also. In this case, checks appeared to propagate from horizontal holes drilled through the webs for the extra stressing bars that were installed below the flange. Assuming that the bottom of the web was restrained at the abutments, the top of the web was restrained by the flange, and the extra stressing bar was applying an additional lateral load below the flange, then additional transverse bending stresses would be created in the webs. This would result in tension-perpendicular-to-grain stresses on the interior face of the webs. Therefore, it is conceivable that the extra stressing bars contributed to the development of interior checks in T-beam bridges.

In several bridges, gaps were observed between webs of adjacent T-beam or box-beam modules. Also, on many of the T-beam bridges, webs were out of plumb. Typically, the top of the web was leaning toward the bridge centerline.

This was likely due to transverse compression of the top flange and the inability of the web to move at its attachment to the abutment. This condition could have occurred during or after construction and worsened during subsequent restressing of the bars. Again, resulting tension perpendicular-to-grain stresses may have led to checks in the sides of the webs.

Crushing of Wood Under Bearing Plates

On several bridges, there was a substantial amount of deformation perpendicular to grain beneath bearing plates. On these bridges, crushing of the web under the plates measured up to 6 mm (0.25 in.) deep. Most plates were 152 by 254 mm (6 by 10 in.) by 25 mm (1 in.) thick. Bridge webs with continuous plates or channels showed only slight signs of crushing. This crushing is likely to have had a significant influence on loss in bar force. In addition, it may indicate that revisions are needed in procedures for sizing bearing plates.

Miscellaneous Items

In many instances, excess creosote was observed on outside web members. As discussed by Dickson (1995), this has been a recurring problem on many bridges treated with creosote and is more common on components exposed to the greatest direct sunlight. In several other cases, what may have been inadequate creosote levels was observed on many curbs and rail posts. While there were no visible signs of decay, severe checking of rail components was typical.

Some stressing bars showed slight corrosion. Corrosion was in the early stages with no pitting or flaking. On many of the bridges, bar ends had been painted in an attempt to inhibit corrosion.

Several T-beam bridges had diaphragms installed between webs. In some cases, diaphragms were not tightly fitted between webs. This was most likely done intentionally during construction to prevent diaphragms from restraining the stressing operations as discussed by Dickson (1995). However, if diaphragms do not fit tightly, they may not contribute to transverse load distribution in the bridge deck. It was because of this problem that the recommendation for diaphragms was dropped in subsequent bridge designs.

Summary

Overall, the bridges inspected were carrying loads for which they were designed. However, there were several issues with their performance that warrant concern. Based on field observations, the following conclusions and observations are presented:

- The relatively frequent instances of cracked asphalt wearing surfaces indicated that slip may have been occurring between webs and flanges. Occurrence of slip would result in bridge stiffnesses and strengths at levels lower than that intended at the design stage.
- The presence of low interlaminar stresses in 10 percent of the bars checked indicated that bridges probably require more periodic maintenance than was being provided. Low interlaminar stresses would result in bridge stiffnesses and strengths at levels lower than that intended at the design stage. Cracks in asphalt wearing surfaces appeared to be related to low levels of interlaminar stress.
- Elevated levels of moisture indicated that wearing surfaces were not providing dry conditions for the interior webs. Also, elevated moisture levels indicated that, eventually, there may be premature decay problems with wood components. More importantly, as moisture levels change, there will be corresponding fluctuations in interlaminar stress. These fluctuations in interlaminar stress will necessitate further restressing operations.
- The presence of checks in exterior webs of many bridges was cause for concern and may have been indicative of excessive levels of transverse bending stresses in the glulam beams. Since many of these checks have appeared during the years since the bridges were erected, cracks may continue to grow. This issue warrants further monitoring to determine if checks are changing in size with length of bridge service.
- The frequency of crushed wood under bearing plates indicates that the design procedure used to size bearing plates needs to be re-evaluated.

In summary, while the bridges are currently carrying the loads for which they were designed, there are several issues that may affect their longevity and their long-term ability to. safely carry vehicle loads.

Acknowledgments

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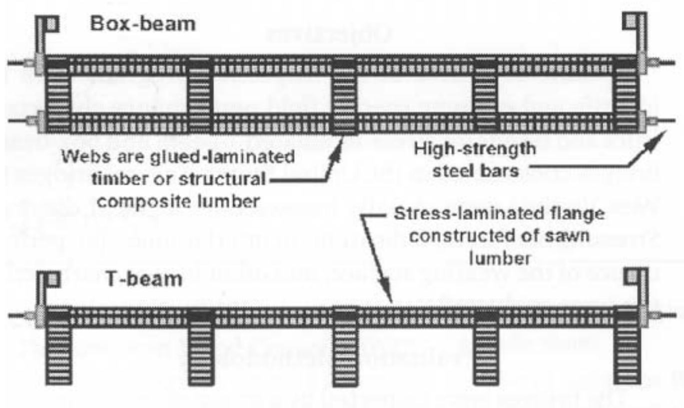
Box-beam
Webs are glued-laminated timber or structural composite lumber
High-strength steel bars
Stress-laminated flange constructed of sawn lumber
T-beam