Bilayer Reinforced Stressed Timber Deck Bridges

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Research efforts to improve the ability of timber to satisfy deflection criteria and to be economical for longer spans are leading to composite timber-steel deck bridges with transverse prestressing. The use of small size and length timbers is imperative because, quite simply, small-to-medium size trees are much more plentiful than large trees. A bilayer timber bridge model that employs both timber and steel is shown here to perform quite well. Results show that despite numerous butt joints and reliance on steel for horizontal shear, loads in excess of required magnitudes may be taken safely without undue deflections or stresses.

The use of timber for highway bridges is increasing. Some states have an abundance of wood, especially hardwood, which has not been used in bridge building. The notion that timber is unsuitable or less durable than other materials is false. In the Northeast, for example, deicing salts limit the average life of concrete bridge decks to about 15 years. Properly treated timber, on the other hand, is immune to the effects of such salts and will give useful service several times that long. An excellent paper concerning the long life and proper preservative treatment of timber bridges has been written by Muchmore (*1*).

Timber trusses of past years were vulnerable in the joints unless they were covered. The popular stressed laminated deck has no truss joints to deteriorate. Consequently, this type of timber structure, pioneered by Taylor and Csagoly (*2*) in Canada, provides a unique design for short spans wherein friction between longitudinal timber laminates is produced by tensioning of transverse prestressing rods.

Tests conducted at the University of Wisconsin (*3*) have established deflections as the governing design criterion in place of flexural or shear stresses for this type of timber deck. Resulting timber stresses are usually well within allowable values. In addition to bridges of this type built in Canada, several demonstration bridges have been built in the United States. The Trout Road Bridge, built near Houserville, Pennsylvania, in May of 1987, has been successfully monitored for 1 year (*4*). Dead and live load deflections, losses in bar forces, and moisture content of the creosoted timber deck were observed and analyzed.

Results indicate a well-behaved and esthetically pleasing bridge type for short spans. The span of 46 ft must obviously have timbers butted together at intervals. The usual procedure has been to limit butt joints no closer than every fourth member at any given bridge cross section. Large Douglas Fir timbers (4 x 16 in.) with a maximum length of 20 ft were used. Such large dimensions are scarcely procurable in most sections of the country. Ways have been sought to form composite stressed timber decks to fully use available timber and to increase practical span lengths.

The design of an innovative stressed timber deck with laminated veneer lumber (LVL) stringers was built as a demonstration bridge of 78 ft for the Regional Timber Bridge Conference at Charleston, West Virginia, in May 1988. Composite action between LVL stringers and longitudinal oak decking was confirmed. Details of this unique design are contained in a paper titled *Design of Stressed Timber T-Beams for Highway Bridges* (unpublished) by Barry Dickson and Hota V. S. Ganga Rao of West Virginia University.

Composite action between timber and steel is also being explored. In another presentation at the conference mentioned in the previous paragraph, the writer described in the *Effects of Steel Plates Inside of Prestressed Timber Deck Bridges* (unpublished) a method, based on model studies, whereby shorter timber lengths can be effectively used when steel plates are sandwiched between timber laminates before rod stressing. Bridge stiffness was shown to more than double when about 7 percent of steel was used. Moreover, timber lengths could be reduced from 20 to 12 ft—a more practical length.

Because of the high modulus of elasticity of steel compared with timber, longer spans, smaller timber depths, better control of creep, and better orthotropic behavior are all possible when steel plates are included. Full-scale testing of such composite behavior will begin in fall 1988 in the structural laboratory at The Pennsylvania State University.

The next logical step in the development of composite timber-steel bridge decks is to consider ways of using square or nearly square timber cross sections of modest sizes—these being most prevalent from medium-to-small trees. The bilayer reinforced stressed timber deck bridge has been recently investigated as a model by the writer and forms the main focus of this paper.

It should be noted specifically that this paper reports results of model tests that have not yet been verified by prototype studies. Economic studies, because of a lack of data, will therefore have to await the construction of prototype and demonstration bridges.

BRIDGE MODEL

A 1-to-12 scale model was built to simulate the behavior of an actual structure with a span of 46 ft. Figures 1*a* and 1*b* show dimensions and loads used for the model. To maintain load symmetry about mid-span, concrete blocks were added in pairs. After each pair was added, dial gauge readings at mid-span front and rear positions were taken and averaged

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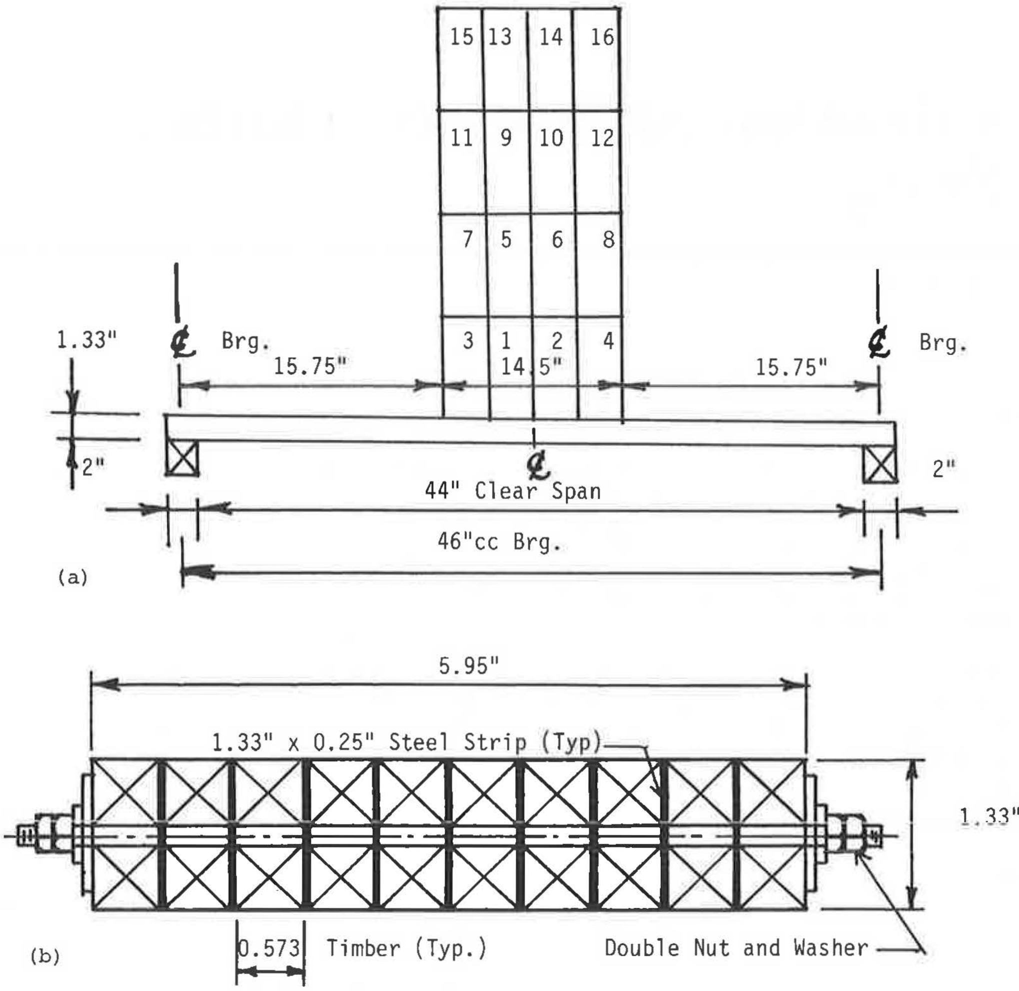

FIGURE 1 (a) bilayer model bridge with 16 concrete blocks; (b) bilayer model bridge cross section with 20 timbers and nine steel strips.

to give the displacement in inches. Each time the bridge model was reassembled (with nuts tightened), a shakedown load of 320 lb was applied and removed before any data were taken. Also, the full load remained on each assembly for 24 hr, after which unloading data were taken with concrete blocks removed in pairs. Thus the effects of creep became apparent.

Dimensions and loads for both model and prototype are listed in Table 1. The maximum load of 320 lb on the model is corrected upward by the square of the scale factor to 46,080 lb on the prototype. Moreover, if a more realistic bridge width of 26 ft is envisioned, then the equivalent total maximum load would be 26 × 46,080/5.8 = 206,566 lb, which exceeds two HS25-20 truck loads (2 × 90,000 = 180,000 lb). The maximum model load of 320 lb thus represents more than sufficient loading for today's highway bridges.

Figure 2 shows an end cross section of the bilayer model bridge resting on a 2-in.-wide timber sill. Each layer is 10 timbers wide with nine 24-gauge sheet metal strips sandwiched between timbers. Double nuts were used to discourage stripping of the threads of the soft steel rods. The grain direction for timber cross sections was chosen randomly and the average modulus of elasticity found to be 1,405 ksi. The red oak used is well-seasoned wood taken from an old church pew. The average moisture content was 9.7 percent, and the average modulus of rupture was 11,815 psi based on failure testing of eight specimens. Figure 3 shows the partially assembled model with stressing rods and steel strips visible. The numbering shown was necessary to ensure that timbers and steel would be in the same relative positions each time the structure was reassembled.

TABLE 1 DIMENSIONS AND LOADS

Variable	Model	Prototype
Scale	1:12	1
Span	46 in. cc brg.	46 ft cc brg.
Timber size	0.580 × 0.580 in.	6.96 × 6.96 in.
Plate size	0.25 × 4/3 in.	0.30 × 16 in.
Number of timbers	10 each layer	10 each layer
Deck width	5.80 in.	5 ft 9.6 in.
Rod size	3/16 in.	1 in.
Rod spacing	4 in. cc	4 ft cc
Loads	16 20-lb blocks	—
Maximum load	16 × 20 = 320 lb	$320 \times 12^2 = 46{,}080$ lb

NOTE: cc brg. = center-to-center bearings.

FIGURE 2 End view of bilayer model bridge resting on sill.

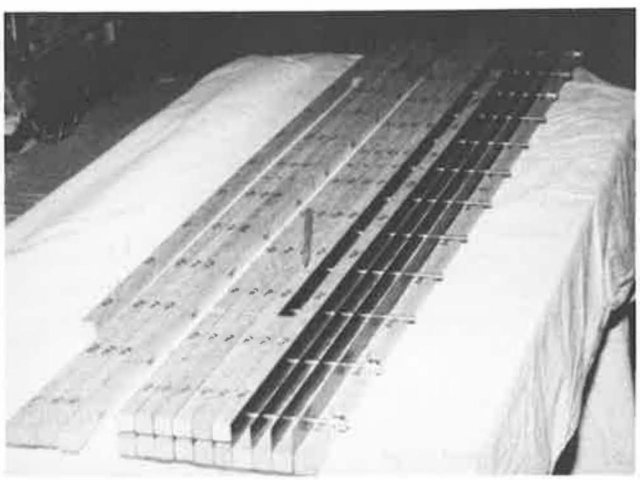

FIGURE 3 Partially assembled bilayer model bridge with steel strips and stressing rods.

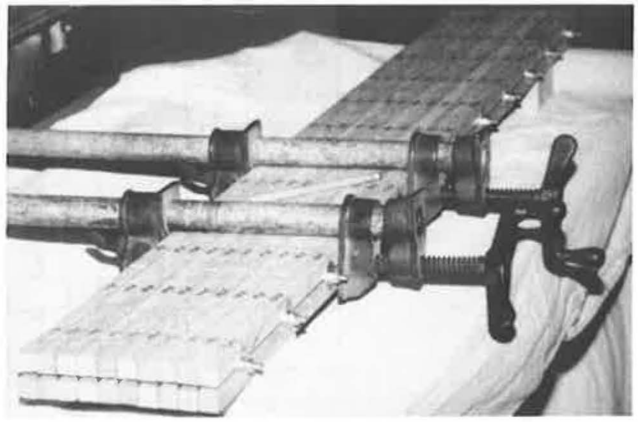

FIGURE 4 Clamping technique used before bolt tightening of the bilayer model bridge.

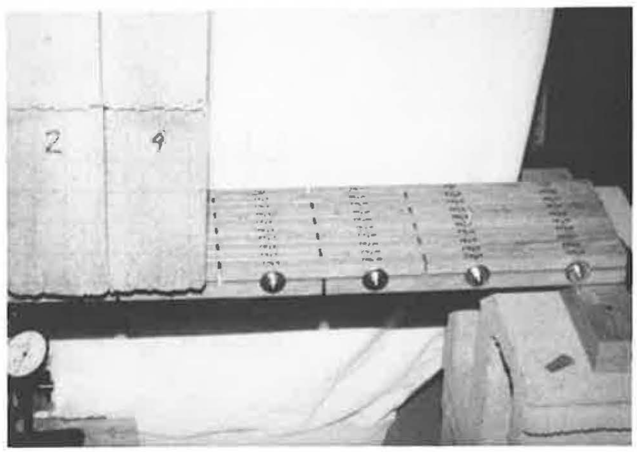

FIGURE 5 Right half of bilayer model bridge under load and showing butt joints.

Tightening of the rods was realized by first tightening clamps near the rods as shown in Figure 4. The fact that the timbers are held in place and transfer stresses to the steel strips by friction alone may seem a bit peculiar, but no slippage was observed at any time—even when an overload was on the structure. In an actual bridge deck, smaller rods could be employed in pairs to pass through top and bottom layer timbers or in a staggered pattern, although this is not necessary for structural purposes. Note also that timbers used for the model required no drilling and were simply placed. Several trials are necessary for rod tightening because the tightening of one rod tends to ease the pressure on adjacent rods. Excessive tightening however leads to shear failure of the threaded rod.

Figure 5 shows a closeup view of about one-half of the loaded beam resting on a timber sill at the right end and loaded with concrete blocks. Butt joints can be seen in both the top and side. These gaps are about 0.10 in. so that no contact between butted timber lengths will occur during the load tests. Butt joints were introduced first near beam ends then progressively toward the mid-span until a total of 80 butt joints and 100 timber lengths made up the model. At this point all timber lengths were either 8 or 12 in. Observe that every second member is butted for top and bottom layers, which means that one-half of the timber cross section has been cut at any section of butt joints. Later results will show surprising strength of this type of composite structure despite weakening by butt joints just mentioned.

Figure 6 shows a plan view of the butt joints for the top layer. The pattern for the lower layer is simply obtained as the mirror image. Butt joints were cut first near the ends and then progressively toward mid-span until a total of 80 butt joints (100 timber segments) were present.

After completion of load tests using nine steel strips and up to 80 butt joints, further tests were run with 80 butt joints and up to 18 steel strips. The strips were added in a symmetrical fashion as depicted in Figure 7. Friction between steel strips, as well as friction between wood and steel, was found to be adequate with no apparent slippage.

TEST RESULTS

Consider the structural behavior of the model bridge. The steel strips must resist all horizontal shear at mid-depth and

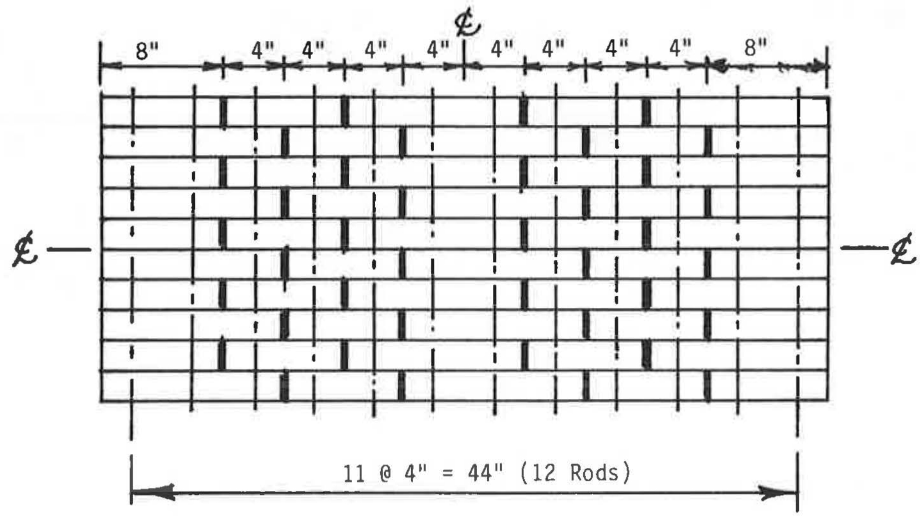

FIGURE 6 Plan view of butt joint pattern for top layer; bottom layer is mirror image.

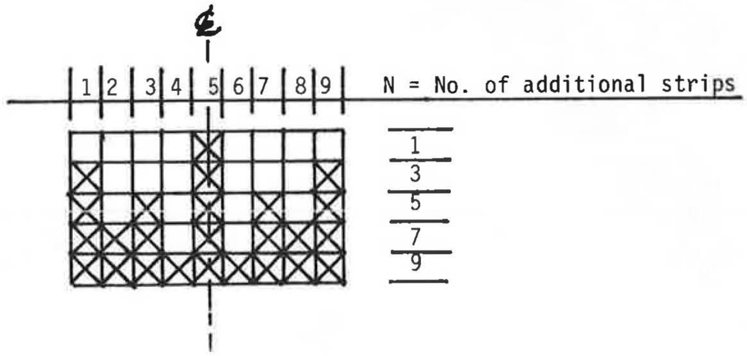

FIGURE 7 Sequence for addition of second set of nine steel strips.

transfer part of it to timbers placed away from the neutral axis. Both steel and timber resist flexural stresses and contribute to the flexural rigidity necessary to resist large displacements. The number and placement of butt joints together with the number of steel strips used become important for the structural action involved. In what follows, stresses in steel and timber are computed to show their variations. Equivalent truck loads are also shown for several conditions.

Figure 8 shows a linear relationship between applied loads and average centerline displacements except when all butt joints are present under the maximum load of 320 lb. With only 20 butt joints present near end supports, the displacement curve lies close and nearly parallel to the curve for no butt joints, indicating that although half of two timber cross sections have been cut away, the structure's flexural rigidity has been affected very little. As more butt joints are employed, the curves are seen to be spaced farther apart, indicating the increasingly detrimental effects of added joints placed closer to the region of high flexure.

It is not surprising that the curve for 80 butt joints lies farthest to the right and diverges from a straight line if it is recalled that 100 pieces of timber and nine steel strips, held together by friction, are being used to resist the maximum loading. The vertical line representing a displacement-span ratio of 1/200 indicates that fewer than 20 butt joints would be permitted for full load. Intersection of this line with the curve for all 80 butt joints suggests that about 280 lb would be permitted if such a displacement-span stipulation were in effect. As seen later, this 320-lb load will be more than adequate.

Figure 9 illustrates an increased drop in timber modulus of elasticity (MOE) from 1,230 ksi to 865 ksi as the number of butt joints is increased toward 80. Not shown, however, is the drop from the actual modulus of solid wood of 1,405 ksi to 1,230 ksi caused by the use of the composite section in place of solid wood. The modular ratio, defined as the ratio of steel MOE/timber MOE, follows from Figure 9. These values are plotted in Figure 10. The large magnitudes between 24 and 34 shown for the modular ratio suggest the increasingly important contribution to strength made by the steel strips as the number of butt joints increases.

The modulus of elasticity of all steels lies between 29 and 30 million psi. Therefore, high-strength steels provide no better defense against large deflections than low-strength steels. They do, however, have higher allowable stresses.

Maximum timber and steel stresses under full (320 lb) loading are seen in Figures 11 and 12 to be well within usual allowable design stresses. First, for steel with a yield stress of 50 ksi, about two-thirds (33.3 ksi) would be a reasonable allowable design stress where plate buckling is precluded. For

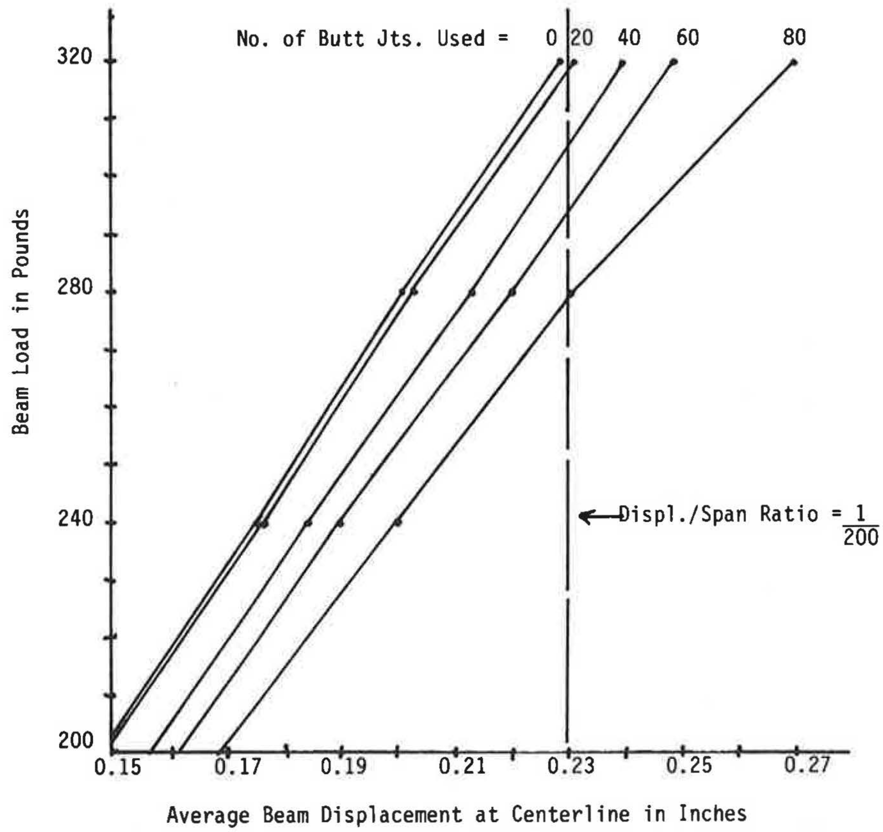

FIGURE 8 Beam load versus centerline deflection for various butt joints (nine steel strips used throughout).

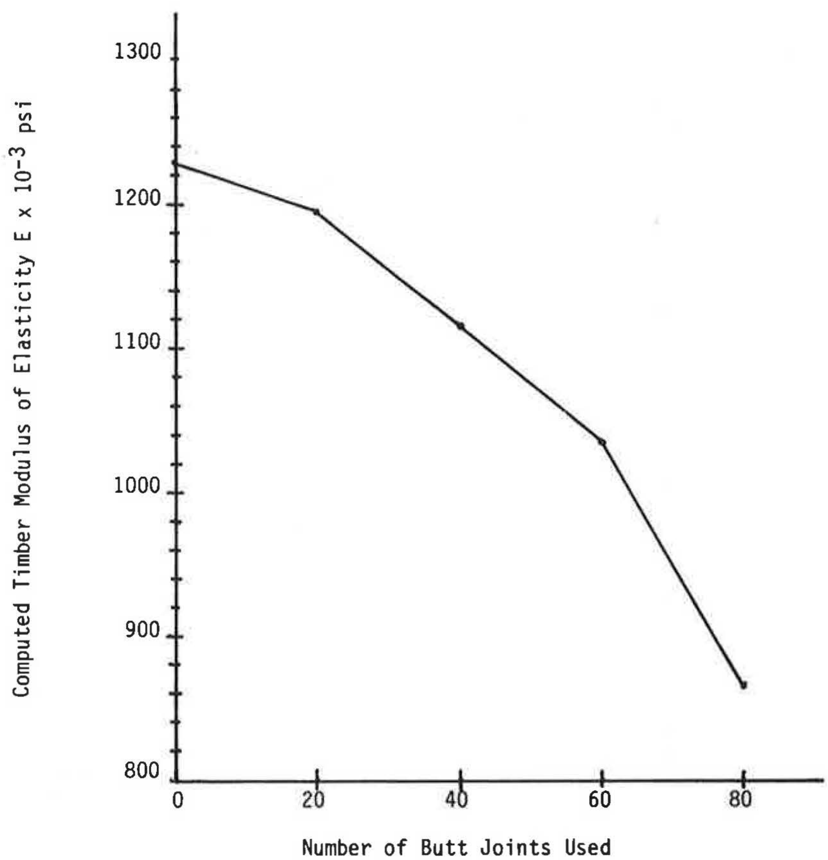

FIGURE 9 Timber modulus of elasticity versus number of butt joints used (nine steel strips used throughout).

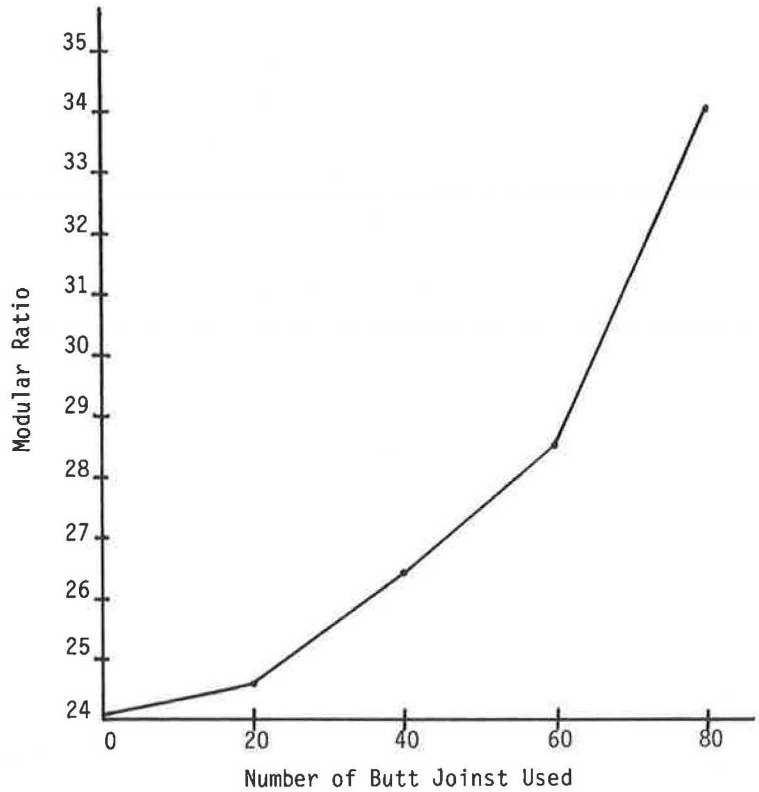

FIGURE 10 Modular ratio versus number of butt joints used (nine steel strips and 320 lbs used throughout).

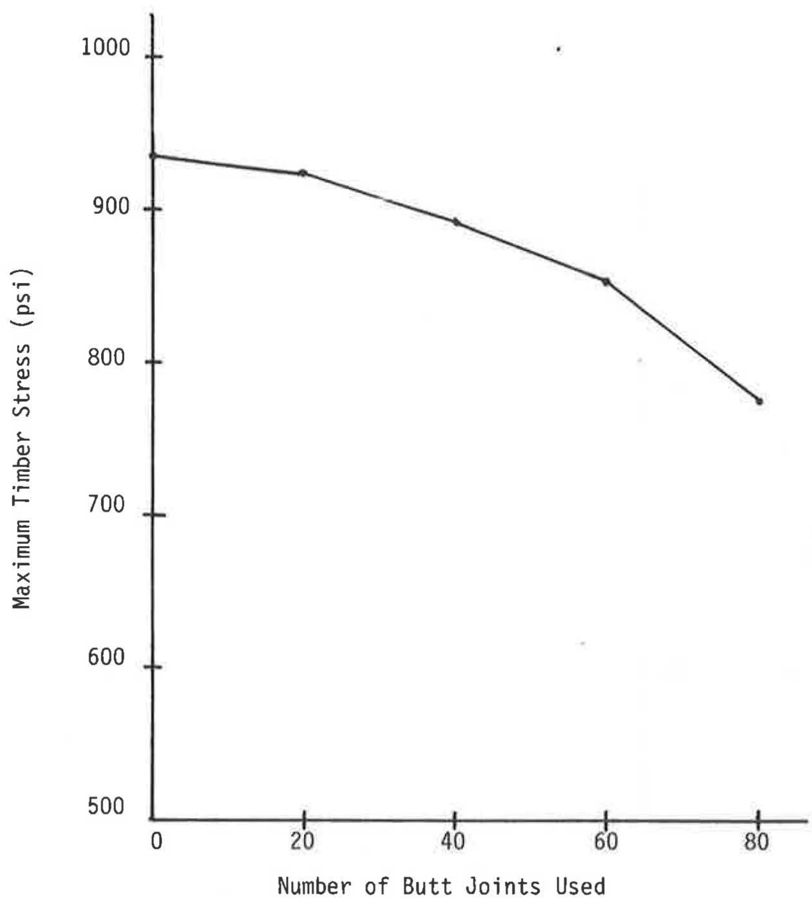

FIGURE 11 Maximum timber stress versus number of butt joints used (nine steel strips and 320 lbs used throughout).

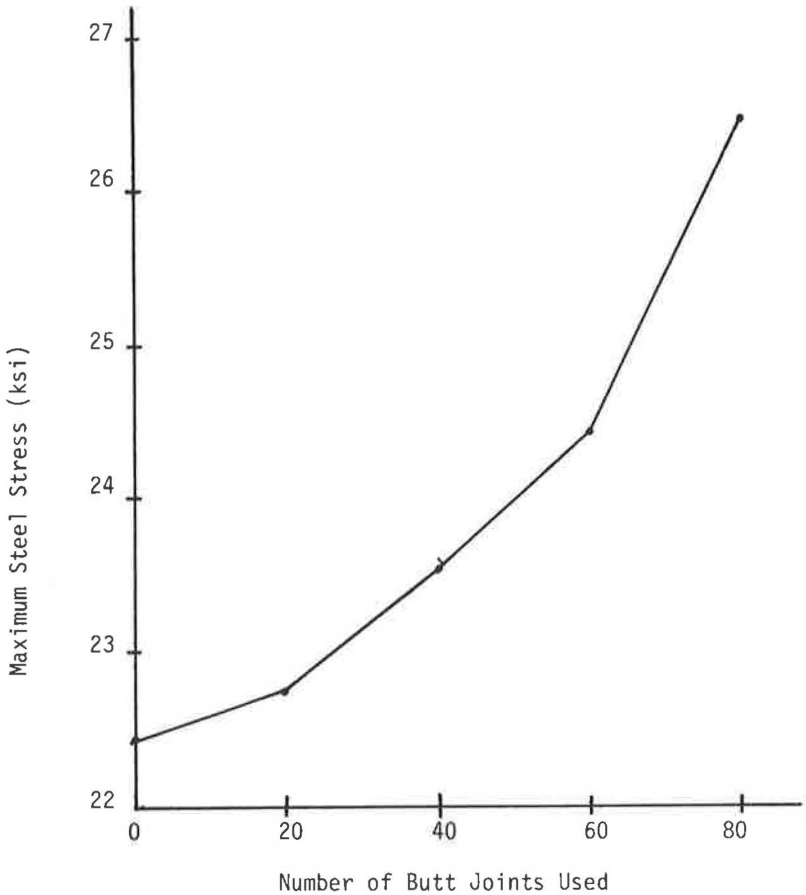

FIGURE 12 Maximum steel stress versus number of butt joints used (nine steel strips and 320 lbs used throughout).

northern red oak, used at a maximum moisture content of 19 percent, the Northeastern Lumber Manufacturing Association lists 1,600 psi as the allowable design stress for select structural beams and stringers. Note the increase in the steel stress curve to compensate for the decrease in the timber stress curve as the number of butt joints increases.

Creep effects were obtained by leaving the full load in place for 24 hr before removing it. Displacements that occurred during that 1-day period are the differences between the broken and solid curves of Figure 13. The two curves converge gradually left to right, indicating better control of creep as the amount of steel used is increased. The horizontal line representing the displacement/span ratio of 1/200 suggests that about 12 steel strips are required to meet such a deflection requirement. Plotted also for 18 steel strips are centerline displacements obtained for closed butt joints. This closing of all butt joints allowed bearing stresses to be transferred between timber ends resulting in a 5.2 percent reduction in deflections.

Flexural rigidity (*EI*) is a measure of the internal stiffness of a cross section. It is affected by the number and placement of butt joints, by layering of timbers as opposed to solid full-depth timbers, by the amount and arrangement of steel used, and, to some degree, by the lateral compressive stress caused by the stressing rods. As a base, a solid timber deck 5.8 in. wide and 4/3 in. deep, with no butt joints, is used. Together with $E = 1{,}450$ ksi for the timber, the resulting base *EI* is 1,609.28 k-in.^{2}. Other values of *EI* are taken relative to this base *EI*. Define a flexural rigidity ratio (FRR) as $FRR = EI/$ base EI.

The variation in this FRR is depicted in Figure 14 as both the number of butt joints and the number of steel strips are varied. The initial value of 1.687 suggests that the composite beam with no butt joints and nine steel strips has 68.7 percent greater flexural rigidity than the beam with a base cross section. As butt joints are added, the FRR is seen to drop to a value of 1.43 for 80 butt joints and nine steel strips. The vertical line shows large FRR increases caused by the addition of more steel strips such that when 80 butt joints and 18 steel strips (9 percent) are employed, the FRR = 2.245. So, despite the decrease as a result of added butt joints, added steel raises the flexural rigidity considerably to 2.245 times the base value for a solid timber cross section of the same depth. These values bear cogitation. This increase in flexural rigidity is shown to be linear in Figure 15.

To gain a better appreciation of load effects, the equivalent truck loading, with three axles spaced at 14 in. apart on the model, was computed as 170 + 170 + 42 = 382 lb versus 320 lb actually applied. Obviously, the concrete blocks were concentrated nearer the centerline but gave the same deflection as the envisioned total truck load of 382 lb. This effect occurs independently of the number of butt joints present.

The study can be carried further by again considering the actual bridge with a 26-ft roadway width and the previously discussed displacement/span ratio of 1/200. Here, axle spac-

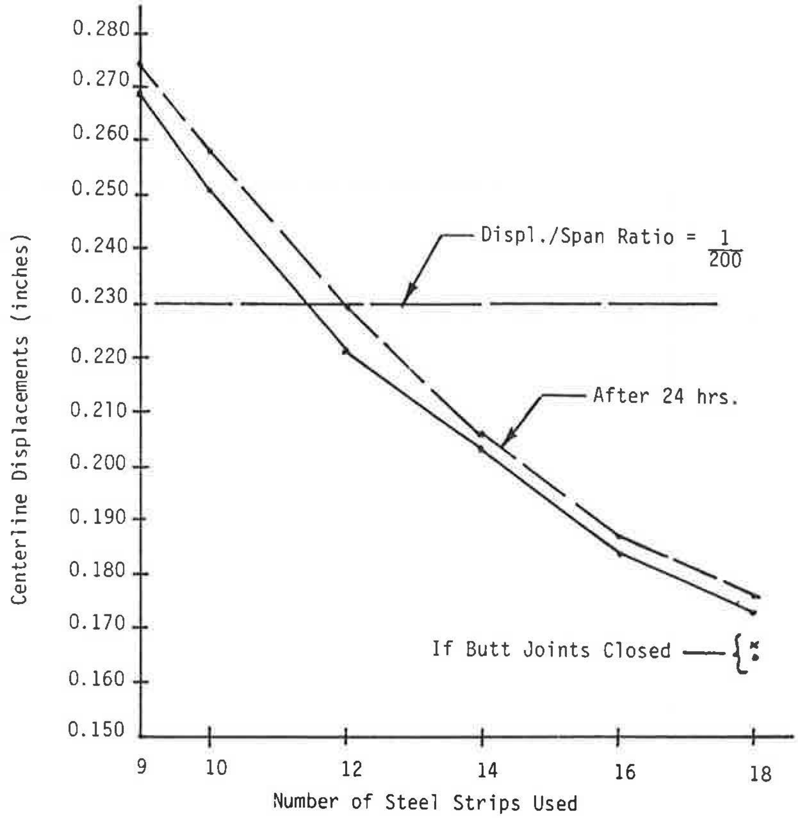

FIGURE 13 Centerline displacements for 320 lb versus number of steel strips used (80 butt joints used throughout).

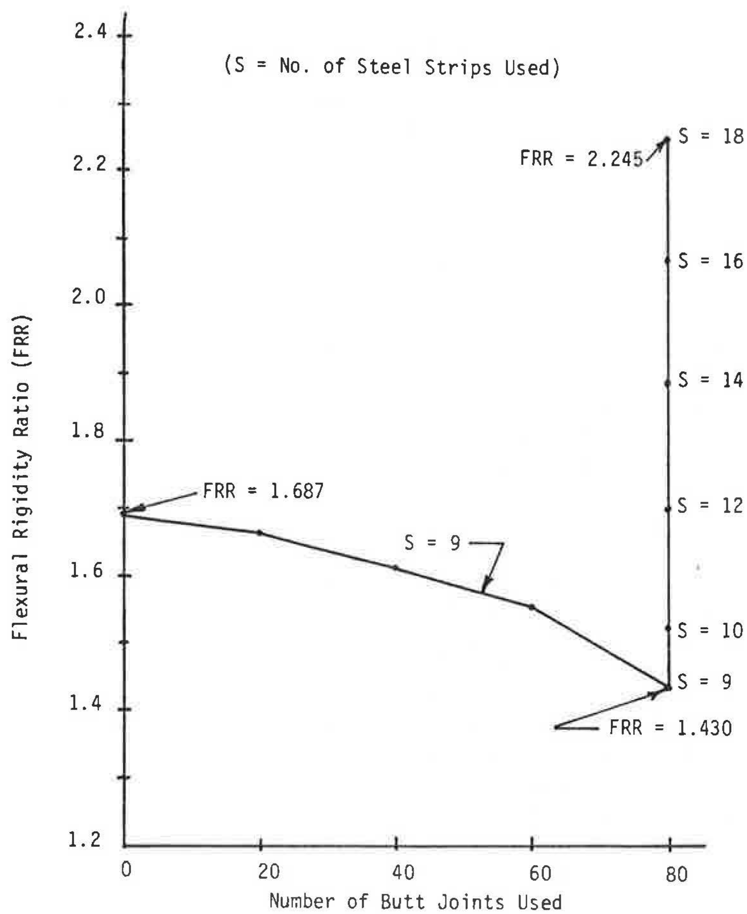

FIGURE 14 Flexural rigidity ratio versus number of butt joints and steel strips used.

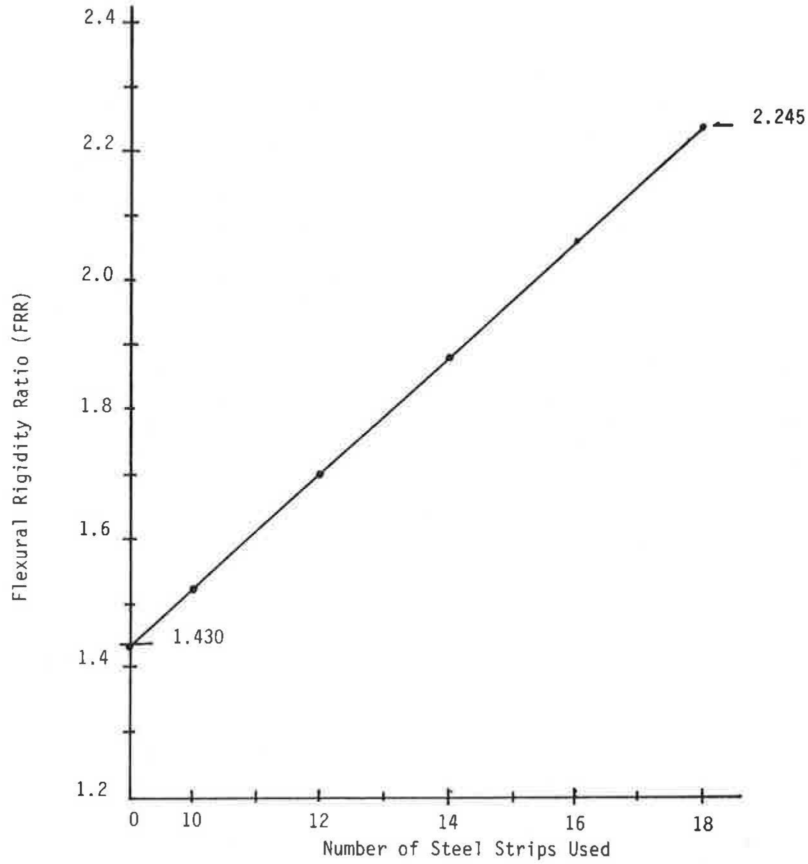

FIGURE 15 Flexural rigidity ratio versus number of steel strips used (80 butt joints used throughout).

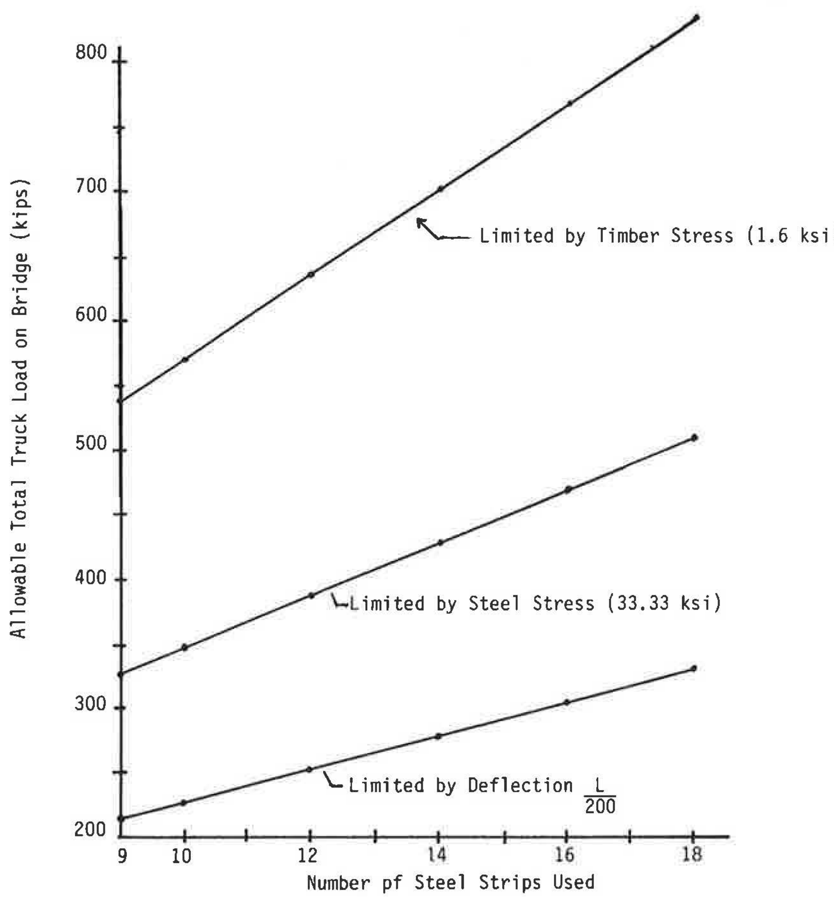

FIGURE 16 Allowable total truck load versus number of steel strips used.

ings of 14 ft and a 4-to-1 axle load ratio are assumed, similar to standard truck loadings of AASHTO specifications. Values of the allowable total truck loads, corresponding to the number of steel strips used for the model bridge, are shown as the lower straight line of Figure 16. The minimum allowable total truck (or trucks) load for the two-lane bridge is 211 kips to limit deflections and with only nine steel strips. Recall that for two HS25-20 trucks, the total load would be $2 \times 90 = 180$ kips.

What about the solid cross section? What allowable truck would it permit? Division of 211 kips by the previously given FRR = 1.43 indicates that $147 < 180$ kips would be allowed. Needless to say, the composite timber-steel deck is quite effective despite the many butt joints. Shown in Figure 16 also are lines representing the allowable truck loads with respect to steel and timber stresses. All curves vary linearly with the amount of steel used. The upper curve represents very high allowable loads based on the allowable timber stress. In other words, excessive timber stress is not the weak link in the design of such composite bilayer bridge decks.

SUMMARY

The composite model bridge deck, with two layers of timbers and with steel strips sandwiched between them, successfully carried larger loads than would normally be expected. Up to 80 butt joints (100 individual timber lengths) caused considerable reduction in the effective timber modulus of elasticity resulting in computed modular ratios between 24 and 34. These high values point to the importance of the nine steel strips used to effectively limit displacements. With 18 steel strips (9 percent of steel) and 80 butt joints, the model was found to have 2.25 times the flexural rigidity of a structure with the same cross-section dimensions but with solid timbers of the same species and no steel strips.

Results of the one-twelfth scale model were projected to an actual structure. Allowable total truck loads were computed with respect to deflections, steel stresses, and timber stresses. The smallest allowable truck loads were caused by the imposed deflection limitation. The minimum allowable truck load of 211 kips exceeds the 180 kips that would be caused by two standard HS25-20 trucks. Allowable truck loads, based on allowable timber stress for the red oak used, were much greater than needed. An important part of the economical design of such a structure lies in the selection of the proper depth and the correct amount of steel. It is hoped that full-scale testing of the bilayer reinforced stressed timber deck bridge proposed in this paper will commence in the not-too-distant future.

REFERENCES

1. F. W. Muchmore. Designing Timber Bridges for Long Life. In *Transportation Research Record 1053*, TRB, National Research Council, Washington, D.C., 1986, pp. 12–17.
2. R. J. Taylor and P. F. Csagoly. *Transverse Post-Tensioning of Longitudinally Laminated Timber Bridge Decks*. RR220. Ministry of Transportation and Communications, Downsview, Canada, 1979.
3. M. G. Oliva and A. G. Dimakis. *Behavior of Stressed-Wood Deck Bridges*. Report 86-4. College of Engineering, University of Wisconsin, Madison, Oct. 1986.
4. R. R. Mozingo and G. DiCarlantonio. *Trout Road Bridge Project*. Final Report. Department of Civil Engineering, The Pennsylvania State University, University Park, June 1988.

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