Oak A-Frame Timber Bridges Meeting the Modern Deflection Requirement

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One of the most difficult requirements for timber bridges has been meeting the live-load deflection limitation of $L/500$ (span length divided by 500) at competitive costs. An 18- × 18-ft all–Pennsylvania oak A-frame bridge has been designed in accordance with the 1991 AASHTO *Standard Specifications for Highway Bridges*, analyzed by a finite element method, and built and tested under an equivalent HS-20 truck loading at Bucknell University to satisfy this live-load deflection requirement at a low cost. An 18-ft-long timber A-frame bridge consisting of two timber A-frames, six timber stringers, two steel hanger rods suspended from the apex of the frames, one steel transverse beam at midspan supported by the hanger rods, and panelized timber decking met the $L/500$ deflection requirement. The total cost of the bridge was $1,500 for material and $2,500 for student labor (estimated). This same A-frame-type bridge can be built economically and also satisfy the $L/500$ live-load requirement for spans up to 50 ft in length.

The use of hardwood timber for modern highway bridges, regardless of span length, has been minimal, particularly with Pennsylvania hardwood. For stringer-type bridges, it is nearly impossible to build bridges with sawn lumber of hardwood species such as oak, because of the maximum live-load deflection requirement of $L/500$, coupled with the high costs caused by this requirement. All timber bridges must meet the $L/500$ live-load deflection requirement and still be economical to compete with other types of bridges.

In some regions, stressed timber bridges are prohibited because of the extensive maintenance required for prevention of adverse shrinkage-swelling and creep effects, which makes the application of timber bridges even more difficult. Another problem with hardwood timber bridges is that there is no available glulam technology for hardwood lumber.

The construction and load testing of an 18- × 18-ft Pennsylvania hardwood timber bridge constructed by students at a cost of $1,500 (material only) are described. The bridge was tested under an HS-20 truck loading and fulfilled the $L/500$ live-load deflection requirement.

DESIGN AND CONSTRUCTION OF BRIDGE

The 18- by 18-ft bridge was constructed with oak timber available from local sawmills at a cost of $0.24 per board foot (Figure 1). The bridge was first designed in accordance with the 1991 AASHTO *Standard Specifications for Highway Bridges* (*1*) to carry an HS20 truck loading.

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The bridge consisted of two 7- × 9-in. A-frames, six 7- × 9-in. stringers, and a series of 40-in.-wide 3- × 6-in. nail-laminated deck panels. For this test program, in order to simulate worst-case service conditions, the deck panels were not attached to each other or to the stringers below. The six stringers were supported at their midspan by a W10 × 26 steel beam, with ¾-in. cover plates welded to the flanges, which was supported by 1-in.-diameter A588 Grade 50 weathering-steel hanger rods from an A-frame apex (Figures 2–5).

The 7- × 9-in. A-frames and stringers were Grade 2 white oak members with moisture content values varying from 40 to 50 percent. The A-frame members were connected to the exterior stringers with 2½-in. split-ring timber connectors. Additional lateral support for the A-frames was provided by connecting each end of an A-frame to an exterior stringer with two steel angles (3½ × 3½ × ½ thick) and four ¾-in. bolts per angle.

The guide rails and wheel guards were made of 6- × 6-in. Grade 2 white oak with 7- × 9-in. posts spaced at 9 ft center to center.

TESTS

Before the present test program, 7- × 9-in. oak timber specimens were tested for flexure and the moduli of elasticity were determined experimentally as part of a research project funded by the Ben Franklin partnership program with the state of Pennsylvania (*2*) and the Burke, Parsons, and Bowlby Corporation (*3*). The values of the moisture content of these specimens varied from 40 to 50 percent and the values of Young's modulus averaged 1.45×10^6 psi. In accordance with the AASHTO design specifications for HS20 truck loading, a 6-ft rear-axle load of 30 kips at midspan with one wheel at a distance of 2 ft from a wheel guard was simulated with a bar joist loaded by a 600,000-lb universal testing machine in Bucknell University's Structural Testing Laboratory as shown in Figure 6. The load was increased in 1-sec intervals by a computer-controlled system from 0 to 32 kips. For each 1-sec load interval, the vertical deflections were measured at three points along the length of the transverse W10 × 26 steel beam (each end and midspan) with transducers at those points that would produce maximum deflections.

In an attempt to predict maximum live-load deflections at midspan, a finite element program (IMAGES 3D) was run during the bridge construction and load testing to compare the predicted deflections with the actual deflections. The value of Young's modulus used was 1.45×10^6 psi determined previously for the 7- × 9-in. timber tested. IMAGES 3D

FIGURE 1 18- × 18-ft oak A-frame timber bridge.

predicted the deflections to be much less than those during the load tests. Therefore, trial and error approaches had to be used to arrive at the as-built configuration that met the live-load deflection requirement.

TEST RESULTS AND CONCLUSIONS

The load test results are shown in Figure 7 (initial-cycle loading), Figure 8 (second-cycle loading), and Figure 9 (third-cycle loading). Figure 8 shows that, at the design load of 32 kips on the bridge, the transverse steel beam deflection was 0.431 in. in the middle of the beam, 0.179 in. at one end, and 0.104 in. at the other end. However, Figure 7 shows slightly higher values, 0.454 in. in the middle, and 0.126 and 0.004 in. at the ends of the steel transverse beam. The corresponding

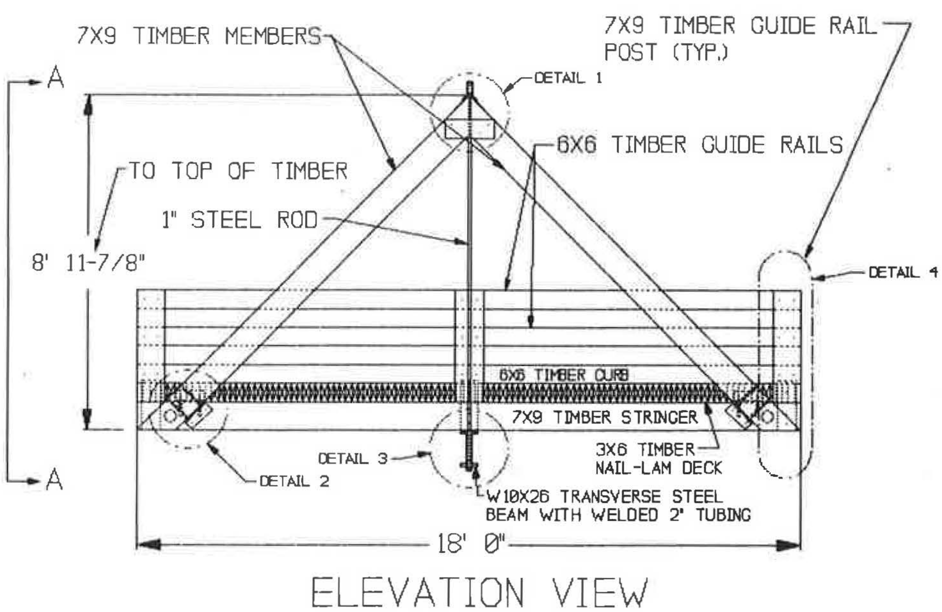

FIGURE 2 A-frame sideview detail.

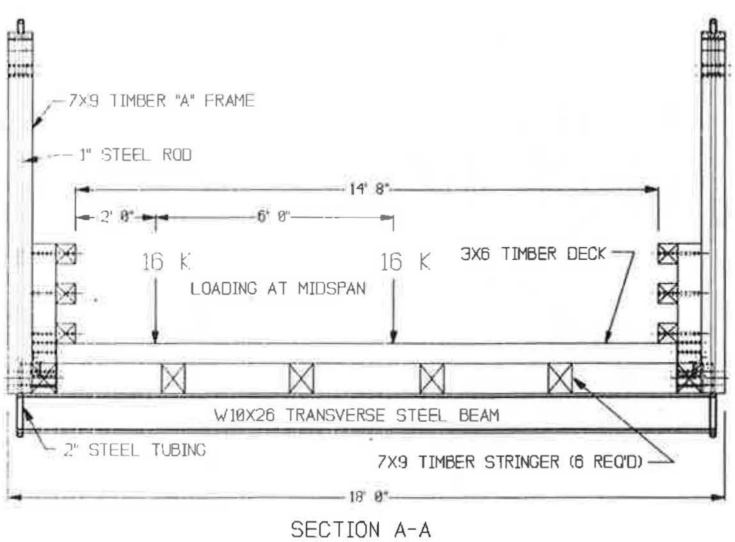

FIGURE 3 Bridge cross-section.

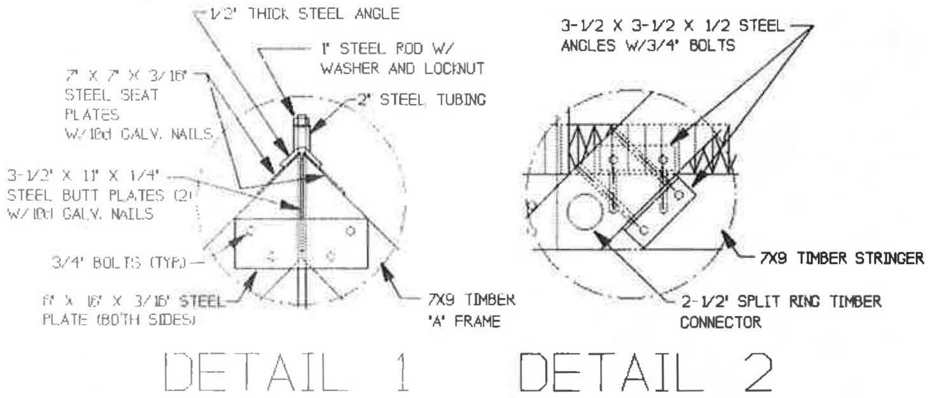

FIGURE 4 A-frame connection detail.

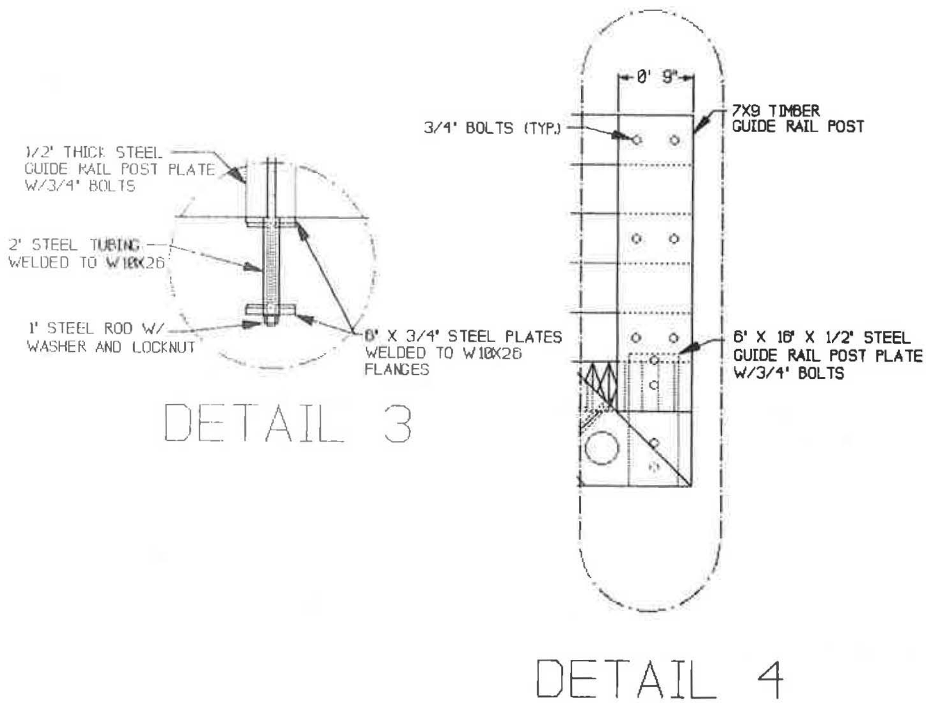

FIGURE 5 Other connection details.

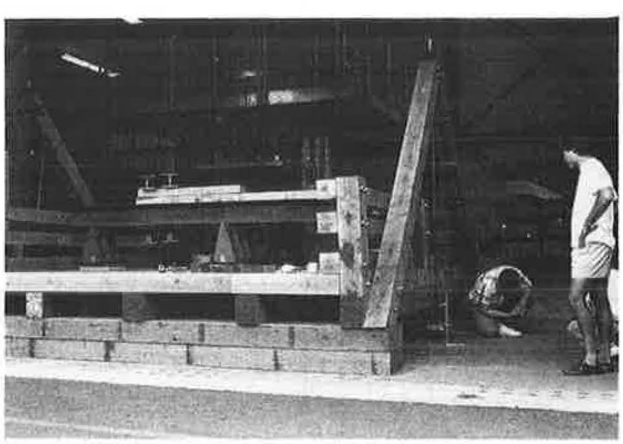

FIGURE 6 Load test set-up for an HS20 loading.

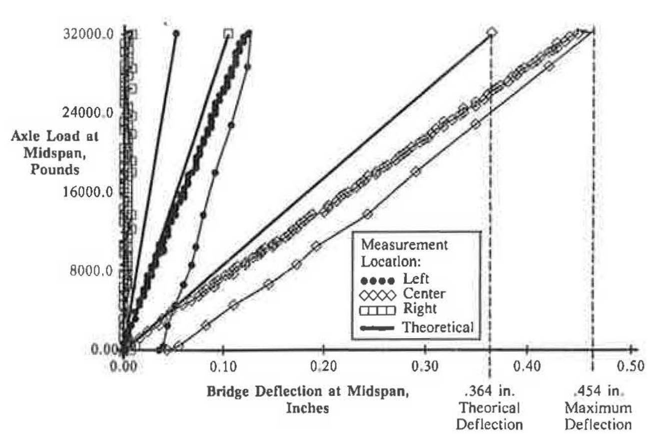

FIGURE 7 Load versus deflection for initial-cycle loading.

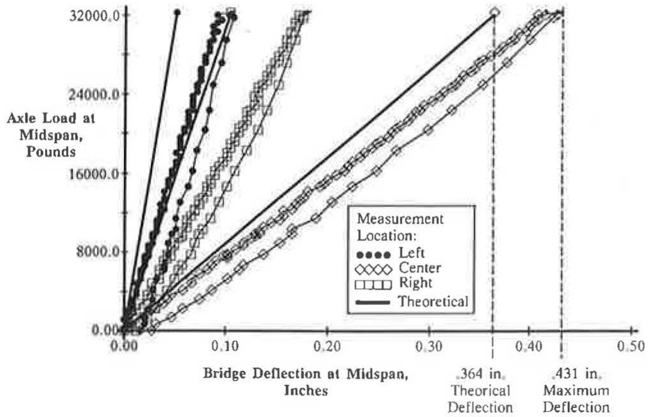

FIGURE 8 Load versus deflection during second-cycle loading.

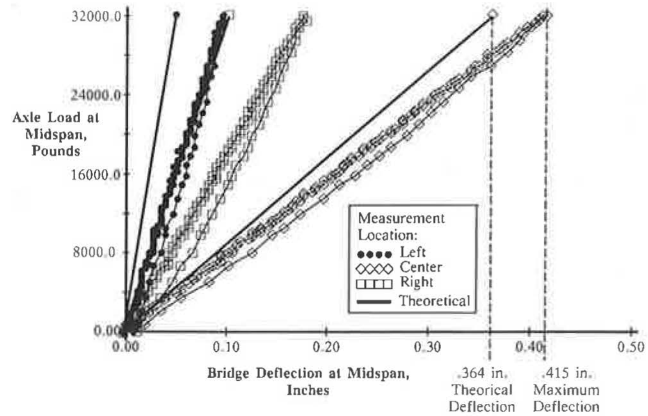

FIGURE 9 Load versus deflection during third-cycle loading.

values predicted by IMAGES 3D were 0.364, 0.104, and 0.051 in., respectively, as seen in the figure. Deflections during initial loadings were slightly greater than those during loadings after tightening the loosened connections. The bridge was then unloaded and reloaded with no significant changes in these deflections as demonstrated in the second cycle loading (Figure 8) and in the third cycle loading (Figure 9). The *L*/500 live-load deflection limitation for this test bridge is calculated to be 0.432 in. The load-deflection curves were linear (Figures 7–9), indicating a linear elastic behavior.

With stringer-type bridges, such as this A-frame bridge, the stresses in the timber members rarely control the design. For example, compressive stresses calculated by IMAGES 3D were less than 200 psi for the 7- × 9-in. A-frame members. The controlling factor was the live-load deflection restriction. Appropriately controlling live-load deflections with this type of A-frame timber bridge can be done easily and economically by varying the size of the transverse steel beam or varying the sizes of the steel hanger rods supporting this steel beam, or both.

The same A-frame bridge concept can be economically applied to longer spans in the range of 40 to 50 ft by lapping stringers at the transverse steel beam with a double A-frame system.

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