ASSESSMENT AND MAINTENANCE OF A 15 YEAR OLD STRESS-LAMINATED TIMBER BRIDGE

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Abstract
A timber bridge consisting of three 6.7 meter spans with a stress laminated deck was constructed in 1991 in the Spirit Creek State Forest near August, Georgia, USA. The stress laminated bridge uses a series of post-tensioning bars to hold the laminations together. The bridge remained in service until 2001 with no maintenance, at which time the bridge was inspected, load tested and the post-tensioning bars re-stressed. In 2005, the bridge was again inspected, load tested, and the bars were re-tensioned. This paper reports on results of the inspection and load tests. The overall condition of the bridge is reported, along with details on the moisture condition, overall deck deflection and timber strains under load. Details on the loss of post-tensioning forces in the bars, and an investigation of the causes of this loss, are presented.

1. Introduction

Stress-laminated bridge decks consist of multiple plies of dimension lumber held together with post-tensioning bars. These decks were originally developed in Canada in the late 1970's as a strategy for rehabilitating decks that had initially been mechanically joined using nails or other fasteners (1). In the United States, they were first considered for new construction in the late 1980's. A large number of these decks were built in the 1990's in the U.S. as a consequence of National Timber Bridge Initiative (2). Many of these decks were built by local governments, in single- and double-lane roads with low traffic volumes.

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Stress laminated decks are used primarily with the laminations parallel to the direction of traffic, on bridges with no longitudinal stringers. The decks bear directly on bridge pier caps or abutments. Given the commonly available depths of solid sawn lumber (250 to 400 mm), the span of these decks is generally limited to around 12 m (3). One of the distinct advantage of this bridge type is the ability to construct the bridge deck in self-supporting panels, which can be constructed of off-the-shelf lumber, and installed using relatively light construction equipment.

2. Bridge configuration and construction

The bridge discussed in this study was constructed in the Spirit Creek State Forest near Augusta, Georgia, USA, in 1991. The bridge was designed and constructed as a cooperative demonstration by the U.S. Forest Service and the Georgia Forestry Commission. Materials for the bridge were sourced locally and were donated by local forest products companies. The bridge decks were assembled on site and placed by a small crane by Georgia Forestry Commission staff.

The bridge consists of three, single-lane 6.7 meter simple spans using No. 2, 2 x 12 (38 mm x 286 mm) boards for the deck (Fig. 1). The three decks were founded on treated pine pilings, caps, and headwalls. All lumber was Southern Pine, pressured treated with CCA. Treatment level was 9.6 kg/m^3 for bridge superstructure elements and 12.8 kg/m^3 for the piles. The laminations in the deck were butt jointed and stressed with 18, 25 mm DWYDAG post-tensioning bars spaced at 1.2 m spacing. The design force in the bars was 240 kN which provided an interlaminar compressive stress of 0.69 MPa between the laminations. Design drawings for the bridge show that the bridge was designed for a deflection limit of L/360 and for a camber of 37 mm. Photographs from the original construction and discussions with people on-site at the time of construction seem to indicate that the camber called for was not achieved. The bridge was designed and installed just before the publication of the U.S. national design specification for stress laminated bridges, and the design deflection requirements and details differ somewhat from the national standard (4).

The bridge was installed with three different wearing surface conditions. The South span was installed with no wearing surface; the central span was installed with an asphalt overlay; and the North span was installed with an asphalt wearing surface and moisture barrier at the interface. Though the design drawings call for a 50 mm asphalt overlay, the actual depth of the overlay was measured as 150 mm at the crown in 2005 (tapering to 50 mm at the curb).


Figure 1: Spirit creek bridge after installation in 1991.

3. Inspection, assessment and maintenance of the bridge

The bridge was first inspected in 2001 by a team of researchers from the Advanced Wood Products Laboratory at Georgia Tech and staff from the Georgia Forestry Commission. The goal of the first inspection visit was to assess the overall condition of the bridge, to re-stress the post-tensioning bars, and to load test the bridge. In 2005, the same team returned to the bridge and repeated the inspection regime. Each inspection visit included the following set of assessments/tests:

- moisture tests in wood members
- re-stressing of post-tensioning bars
- photo documentation of bridge site
- load test with deflection and longitudinal strain measurements
- In situ vibration testing (frequency and mode shapes), 2005 only

It is important to note that most guidelines for the maintenance of stress-laminated bridges recommend that the stress in the post-tensioning bars be checked annually for the first two years of the bridge's life, and then once every 2 to 4 years afterwards (1, 3). In this instance, the first re-tensioning came 10 years after the bridge's installation. This example provides a unique opportunity to assess the condition of a stress-laminated deck that was not maintained according to recommended procedures.

In the text that follows, the following information is presented and discussed. In Section 4, residual forces in the post-tensioning bars are provided. In Section 5, moisture contents in the various elements of the bridge are provided. In section 6, results of load testing (deflection, longitudinal strains) are presented. Vibration data, taken primarily to establish an initial "signature" for the bridge for future testing, and to help estimate the dead load on the bridge, are not discussed here.

4. Residual forces in post-tensioning bars

In 2001 and 2005, the residual post-tensioning forces in the bars were assessed using the "nut turn method" (3). While the assessment team does not have measurements of the level of forces in the bars as installed in 1991, it is known that the design-level force in the bars was 240 kN and that FPL (Forest Products Laboratory) recommendations for jacking of bars was followed when the bridge was installed.

These residual forces as measured in 2001 and 2005 are shown in Figure 2. Bar forces are reported to the nearest 5 kN, as this is considered a reasonable degree of confidence in the measurement given the method and equipment used for checking the bars. In 2001, 10 years after the bridge was constructed, the average force in the bars was 70 kN, or about 30% of the original design force. A number of the bars at the central span had forces as low as 10 kN, indicating that these bars were essentially loose. At the high end, some bars had residual forces of 110 kN, almost 50% of the original force. Ritter indicates that a drop of interlaminar stress to 0.15 MPa (approximately 25% of the original stress) is necessary to allow the laminations to slip relative to one another (1).

Early work by Batchelor predicted that losses in post-tensioning forces would be limited to around 50% and that the rate of stress-loss over time goes to zero; later work by Oliva et al. showed that losses could be much higher than 50%, and in one experiment, the rate of stress-loss over time did not go to zero (5). Oliva also indicates that moisture cycling is a primary cause of stress-loss. It is interesting to note that for the Spirit Creek Bridge, the central span is directly over the creek, and that the creek is generally dry under the end spans. Our assessment of moisture content in the bridge deck is not sufficient to indicate whether the laminations in the central span are at a higher moisture content than the end spans. Future work is planned to address this limitation.

It is also interesting to note that the central span of the bridge was installed with an asphalt wearing surface, and that the average dead load imposed on the span due to this load is around 150 kg/m^2. The North span of the bridge was also installed with an asphalt wearing surface, but this span does not show the level of post-tensioning losses as compared to the central span.

Even though the residual bar forces were quite low, there was no evidence of slippage between the wood laminations in the 2001 assessment. Any lamination misalignments noted in 2001 appeared to correspond to those shown in photographs taken at the time of construction.

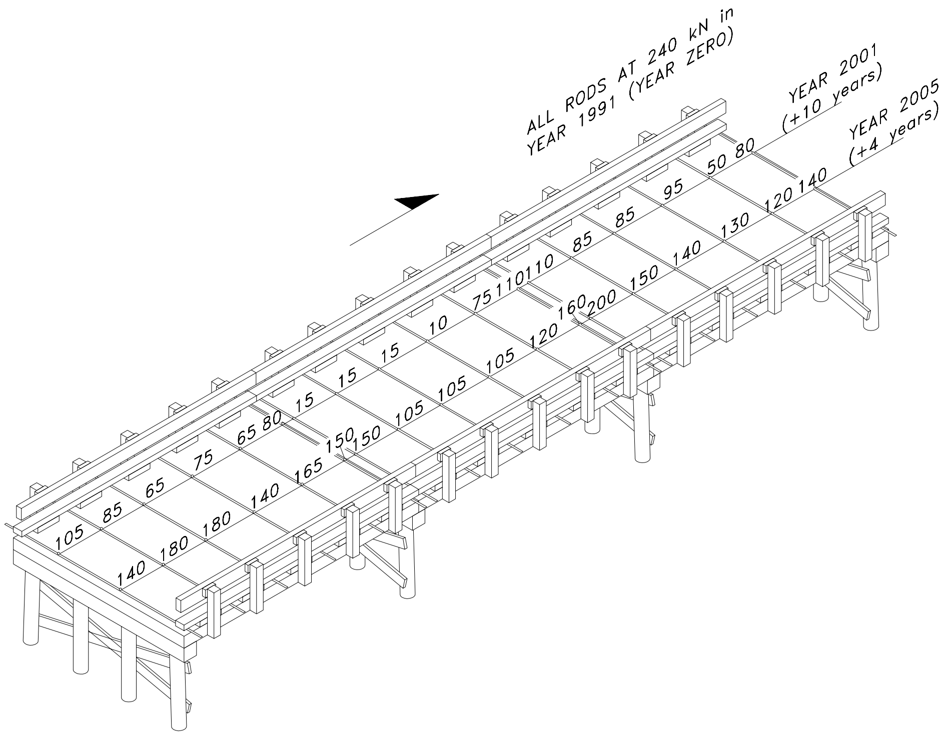
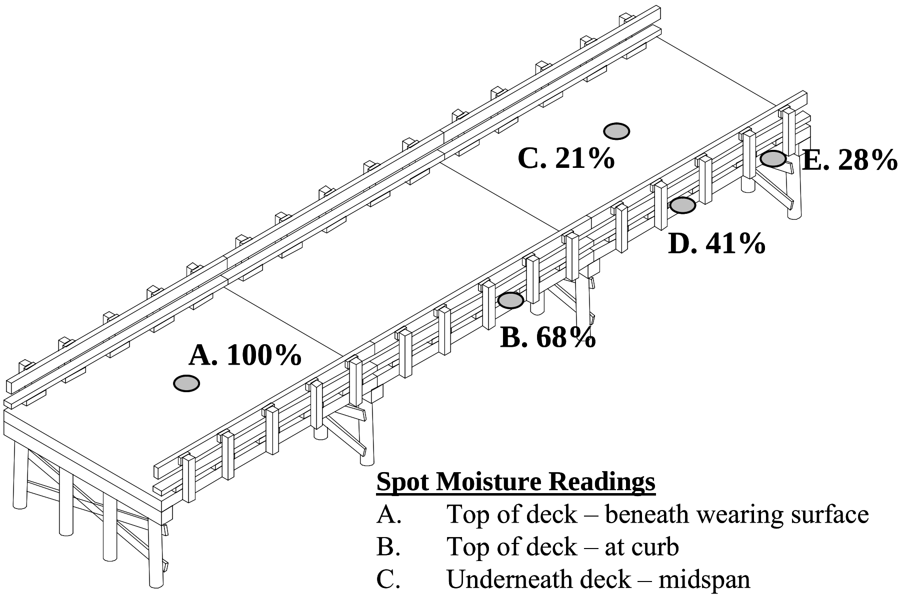


Figure 2: Residual post-tensioning forces in DWYDAG bars, 2001 and 2005 (in kN).

In the 2005 assessment, the force in the bars was measured again, using the same procedure. The average force in the bars was 145 kN (60% of the 2001 re-tensioning force), with the low being 105 kN (45% of the 2001 re-tensioning force). Once again, the lowest residual forces were found in three bars in the central span of the bridge.

5. Moisture content in bridge elements

In 2001 and 2005, moisture contents in readily accessible elements of the bridge were taken using a Lignomat G1000 moisture meter with an E12 electrode and 50 mm pins. Representative moisture contents observed in 2005 are shown in Fig. 3. At locations along the side of the deck, moistures contents were taken just adjacent to the post-tension bar bearing plates. At the top of the deck, the moisture contents were taken underneath the asphalt wearing surface at the centerline of the roadway and at the curb (where no asphalt was present).



At location "A", approximately 150 mm of asphalt was removed to reveal the top portion of the wood deck. There was no moisture barrier between the asphalt at the deck at this location. The wood at surface of the deck in this location was essentially saturated, and was found to be soft. The moisture reading from the meter was 100%.

At location "B", which was on the deck beneath the guardrail, there was approximately 50 mm of dirt and debris. The moisture content here was 68%. At locations "C" and "D", taken adjacent to the post-tensioning bar bearing plates, the moisture contents were 41% and 28%.

In all locations, the moisture contents were considered to be high, and it may be that moisture contents are correlated with the loss in post-tensioning force observed. Yazdani et al. completed a parametric study of stress-laminated wood decks, and included considerations of moisture in this study (6). Both this work and earlier work by others (Oliva, Batchelor) indicate that moisture cycling leads to loss of post-tensioning force. What is not clear is whether high moisture contents lead to increased transverse compressive creep of the wood – which would then lead to subsequent post-tensioning losses. This phenomenon is not described in the stress-laminated deck literature. Pellicane and colleagues describe a model for the transverse compression of wood (7).

Unfortunately, it is not clear how this analytical work can be extended to include the effects of moisture cycling.

6. Load test data

In both 2001 and 2005, a load test was performed. Load test data from the 2005 assessment are presented and discussed below. The bridge was loaded with a Georgia Forestry Commission truck (3 axle) and trailer (2 axle), carrying a fire plow. The weight of the truck was 180 kN (on one steering and two non-steering axles) and the weight of the trailer was 125 kN. This load is slightly below the standard HS-20 loading (rear axle approximately 13% low), which was the design loading for the deck. During the load test the truck moved across the bridge in 1.2 m increments, from North to South. At each load "station", the truck was stopped and deflection and longitudinal strains in the wood deck was measured. A graph of deck deflection as function of truck position is given in Fig. 4. A graph of longitudinal strain as a function of truck position is given in Fig. 5.

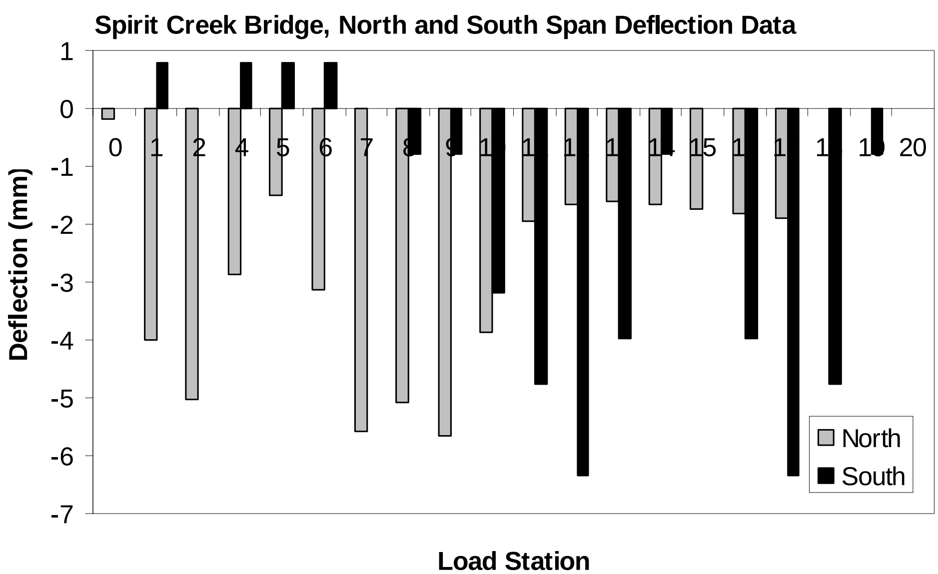


Figure 4: Deflection under Moving Truck Load.

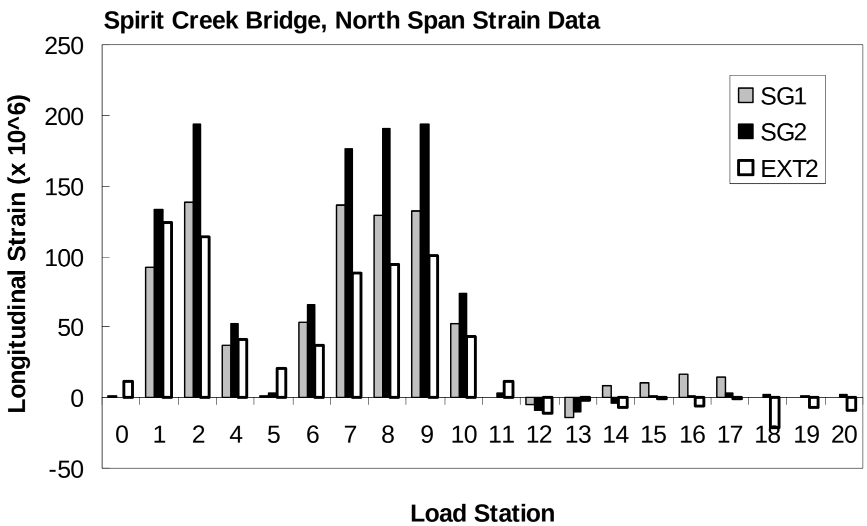


Figure 5: Observed strains in deck under moving truck load (2005).

Deflections were measured at mid-span of the North and South spans with a string potentiometer (North span) and laser distometer (South span) with an accuracy of ± 0.1 mm. The maximum absolute deflection observed was 6.4 mm. This equates to a deflection ratio of L/960 of the clear span, which is only one-third of the design allowable deflection of L/360. This might be considered surprising given that the 2 x 12 deck is at the upper bound suggested by AASHTO given the span (4). In many instances however, it is the limit on transverse flexural stresses, and not deflection or longitudinal stresses that controls the design of stress-laminated bridge decks. In laboratory testing, Olivia showed that deflections in decks with butt joints show substantially more deflection than decks with full-length laminations (5). The load test on the Spirit Creek Bridge, which has butt joints in all three spans, demonstrates that this additional deflection is probably not significant in a full-scale structure.

Longitudinal strains were taken only on the North span (Fig. 5). Three gages were used to record strain. Gage SG1 was located at the centerline of the bridge, at midspan. Gage SG2 was located at the edge of the bridge, directly under the loaded tire, at midspan. Gage EXT2 was adjacent to gage SG2. The SG gages were bonded resistance strain gages with a 100 mm gage length. Gage EXT2 was a screw-on extensometer that was joined to the wood lamination using wood screws. Initial readings of EXT2 appear to be quite close to SG2, but subsequent readings lead to conclusion that EXT2 may have slipped relative to SG2 and thus provides a low-biased reading.

The maximum strain read in any of the gages was 194 $\mu\varepsilon$, recorded in gage SG2 at load station 9 when the rear axle of the trailer was directly at the mid-span of the North span. At this point, the centerline gage read 132 $\mu\varepsilon$. Though there is an obvious strain lag across the bridge deck, it can be concluded that the centerline gages are participating substantially in the flexural capacity of the bridge.

Assuming a modulus of elasticity of around 10 GPa for No. 2 Southern Pine, and a peak strain of 200 $\mu\varepsilon$, this equates to a flexural stress in the wood of approximately 2 MPa in the wood under the load of trailer.

7. Conclusions

The overall condition of the Sprit Creek Bridge is good, especially given the lack of maintenance that it has seen over its 15 year life. Though there is some localized evidence of wood degradation, the majority of the wood is sound. Load testing indicates that the bridge is in excellent overall shape, and that the observed longitudinal stresses and deflections under load are well below the as-designed allowables.

Loss of post-tensioning forces in bars in the period from 1991 to 2001 was excessive. Though no transverse slippage of the laminations was noted in the 2001 inspection, this may have been due to lack of heavy vehicle loading. Post-tensioning losses over the four-year period from 2001-2005 are in the acceptable range (assuming that 50% loss is expected as per the 1983 Ontario Bridge Code), indicating that a four-year period between re-tensioning is acceptable.

High moisture contents observed in the laminations at the top of the deck and at the side laminations may be contributing to premature loss of post-tensioning forces in the bars. Data acquired to date is insufficient to confirm this suspicion. Future assessment protocols will be modified to probe the correlation between moisture content and level of residual bar force.

8. Acknowledgements

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9. References

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