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Field Evaluation of Stress-Laminated Hardwood Timber Bridges with Extended Service Life

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ABSTRACT: During the summer of 2019, seven stress-laminated deck timber bridges were evaluated to assess their field performance and serviceability. These bridges were originally constructed during a six year period (1988-1994) using hardwood lumber as part of a national program initiative in the USA. Its aim was to increase the use of wood in transportation structures primarily in the rural regions of the country. To assess bridge performance, comprehensive visual inspections were conducted and advanced inspection tools were utilized to accurately assess the bridge superstructure components using a more quantitative and reliable method. In addition, in-situ stressing bar forces were measured using hydraulic equipment and steel hardware.

This paper elaborates on the current condition of two of these bridges and makes a comparison to previously collected data gathered through previous monitoring studies. Generally, the field performance of the evaluated stress-laminated deck bridges can be categorized as varying levels of unsatisfactory performance. Ongoing maintenance is recommended including stressing bar retensioning (every 2 to 5 years) to ensure interlaminar stresses are sufficient to ensure wheel loads are distributed across the bridge deck in an efficient manner.

KEYWORDS: Stress-laminated, hardwood, bar force, maintenance, performance, non-destructive.

1 INTRODUCTION

In 2015, a study entitled A *Field Assessment of Timber Highway Bridge Durability in the United States* (Wacker et al, 2015) was completed to evaluate the performance of timber bridges nationwide. In the field study, five inspection teams inspected a total of 132 timber bridges, in a cluster-type fashion, that included girder and slab style superstructure types constructed of sawn lumber, glue-laminated (glulam) timber and/or steel girders. Two key prerequisites for study inclusion were that the bridges had to be in-service for more than 25 years and the maintenance/repair records were also available for review. However, stress-laminated bridge structures were not included in this initial field inspection study as their locations were widespread. The conclusions from the project were timber is a durable alternative for highway bridges and included exemplary bridges that performed satisfactorily for 75 years. Furthermore, use of advanced inspection tools, in addition to traditional visual inspection techniques, allowed the inspectors to fully and accurately assess the condition of the timber bridge members in a quantifiable way. These tools allow the inspector to not only locate areas of potential or early onset decay more accurately, but also quantify that decay such that future inspections can track the progression of any decay and efficiently implement a maintenance plan in a timely fashion.

To continue administration of this inspection program and the transfer of information, the gathering of current performance data on additional timber bridge superstructures is warranted. This includes a group of stress-laminated timber bridges built in Iowa and Pennsylvania. Information on the field performance of these structures will provide a high level view of the effectiveness of these designs in terms of service life, cost effectiveness, and general performance from the serviceability, maintenance/repair, and structural safety perspectives.

2 BACKGROUND

Three decades ago, the United States Congress passed legislation known as the Timber Bridge Initiative (TBI) with the objective to establish a national program to provide effective and efficient utilization of wood as a structural material for highway bridges. The U.S. Department of Agriculture (USDA) Forest Service took responsibility for the development, implementation and administration of this program and established three key program areas: demonstration, technology transfer, and research. One goal of the program aimed at encouraging innovation in bridge designs and applications through the

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use of new or previously underutilized wood products. Therefore, emphasis was placed on the new stress-laminated bridge superstructure design alternative which was recently adopted as an AASHTO (American Association of State Highway and Transportation Officials) Guide Specification, and permitted the utilization of secondary hardwood lumber species.

The Forest Products Lab (FPL), recognized as the national wood utilization research laboratory within the USDA Forest Service, was assigned the responsibility for the research portion of this program. Over the years, the FPL has taken the lead role in assisting local governments in evaluating the field performance of various timber bridge types and designs, many of which employ innovations in design or materials, or both, that have not been previously evaluated.

In the more than 30 years since its commencement, the TBI and FPL have developed, demonstrated, and evaluated timber bridge innovations and advancements all over the United States. Though numerous projects were completed as part of the TBI program, a subgroup of projects were conducted which involved the development, design, construction and field performance evaluation of several timber bridges in Iowa and Pennsylvania built with hardwood species.

3 STRESS-LAM DECK DESIGN

Originally developed in Canada to rehabilitate delaminated nail-laminated bridge decks, the method of stress-laminating timber bridge superstructures was adapted to the construction of new longitudinal timber deck structures. These structures were typically made up of timber laminations 52 to 104 mm (2 to 4 in.) in thickness and with depths varying from 305 to 406 mm (12 to 16 in.), vertically oriented and squeezed together using high strength threaded steel bars extending transversely through all laminations at mid-depth. Nuts at each end of the threaded bar are tightened to a level at which a specified design force is achieved in the bar. Additional mechanical fasteners or adhesives are not required between laminations. Staggering the butt joints between adjacent laminations affords the bridge to be longer than any single timber member. In a couple of instances, 10 mm (0.38 in) thick steel plates and/or steel channels were combined with the timber laminations to enhance the overall strength and serviceability. Figure 1 illustrates the composition of a stress-laminated bridge superstructure in plan, profile and section views.

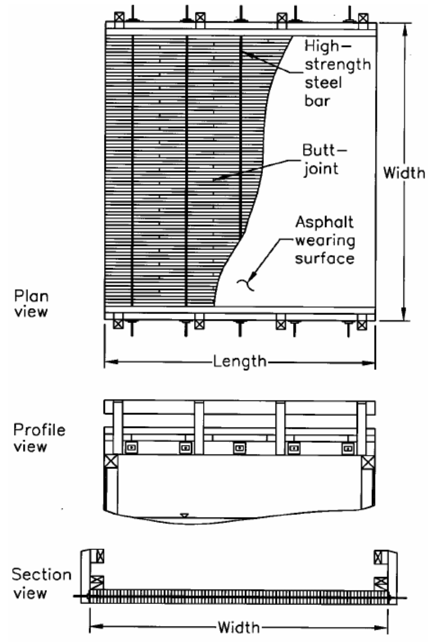

Figure 1 *Stress lam bridge configuration [3].*

4 EVALUATION METHODOLOGY

To assess the long-term performance of hardwood stress-laminated timber bridge decks, a multi-pronged approach to gathering information and inspection data was employed. The protocol included conducting visual observations, measuring the in-situ bar force of stressing bars, collecting moisture content data at select locations, and employing a resistance micro-drill to penetrate the timber laminations to detect internal defects and voids. Also, a review of the key maintenance and repair records was performed where available.

4.1 VISUAL OBSERVATIONS

Numerous visual observations were made and noted at each bridge that include but are not limited to many of the following: Crushing of outer deck, asphalt cracking, lamination slip, broken or damaged stressing bars, corrosion of stressing bar, corrosion of nut, sagging of the bridge deck, out-of-plane deck distortions of the bridge deck, plumbness of guardrail posts, areas of damage exposing untreated wood, failure of any members, plant growth, and decayed areas.

4.2 BAR FORCE

The existing force in the stressing bars at each bridge was measured individually when bars were safely accessible. The repetitive process required applying a tensile force to the bar until the compressive force was equalized or slightly exceeded and the compressive bar force effectively approached zero. The process required several tools including: a stressing bar extension, a coupler nut, fabricated steel chair, steel plate washer, hydraulic jack, hydraulic pump with pressure gage, reaction nut, and a pry bar. Future guidance on these techniques for measuring in-situ bar forces will be made available.

Figure 2 *Jacking assembly showing the steel hardware and hydraulic equipment installed at one bar location.*

4.3 RESISTANCE MICRO-DRILL

Select areas were identified at each bridge site to obtain measurements using an IML PD-400 micro-drilling resistance tool. This instrument records drilling resistance as a function of drilling depth, which is subsequently plotted allowing for an assessment of deterioration within the drilled cross-section. Deviations in the micro-drilling resistance, especially where the resistance is markedly decreased, indicates a potential area of internal deterioration or void. There are several instruments from different manufacturers that work under the same principles; they measure the electrical power consumption of a needle rotation motor which is proportional to the mechanical torque at the needle and is a primary function of the wood density. The needle tip measures approximately 1.5 to 3 mm (0.06 to 0.12 in) wide and up to 457 mm (18 in) long. As these instruments continue to advance in capabilities, electronic storage and wireless options have come available. Further discussion of these instruments is currently available (Brashaw et al, 2014).

Figure 3 *Resistance micro-drill in use at outer deck laminations.*

4.4 MOISTURE CONTENT

A conductance-type moisture meter, Delmhorst Instrument Company, Model RDM-3, was used to collect moisture content data in accordance with ASTM D4444. Insulated probe pins 76 mm (3 in) in length were driven into selected areas of the bridge superstructure, typically to 38 and 76 mm (1.5 and 3 in) depths with moisture content measurements taken at each respective depth. Corrections were made both for temperature and for wood species in accordance with the manufacturer's recommendations.

Figure 4 *Measuring moisture content at deck underside.*

5 BRIDGES OF STUDY

A summary of the bridges studied for this project are provided in Table 1. The bridge location, the year of construction completion, and period of past performance monitoring are included.

Table 1 *Bridges included in overall study (asterisk denotes bridges included in this paper).*

Bridge name	Construction completed	Past monitoring period
State of Iowa		
Hibbsville*	Jan. 1994	2 yrs, 1994-1996
Decatur	June 1994	2 yrs, 1994-1996
State of Pennsylvania		
Little Salmon	Nov. 1988	4 yrs, 1988-1992
Hoffman Run*	Oct. 1990	3 yrs, 1990-1993
Dutch Hill	Dec. 1992	4 yrs, 1997-2001
Brookston	June 1992	4 yrs, 1997-2001
Laurel Run	Aug. 1992	4 yrs, 1997-2001

5.1 HIBBSVILLE

The Hibbsville Bridge, constructed in 1994, is located on a single-lane, gravel roadway, approximately 3.2 km (2 miles) southwest of Numa, Iowa (40° 39'34.45"N, 92° 59'20.55"W) that provides primary access to a rural cemetery. It was designed for AASHTO HS20-44 truck loading and is composed of a 7.3 m (24 ft) long by 5.2 m wide (17 ft) wide, stress-laminated superstructure made up of 355 mm (14 in) deep eastern cottonwood sawn lumber laminations. Butt joints were placed at every fourth lamination with a 1.2 m (4 ft) longitudinal spacing between butt joints in adjacent laminations. Timber components were treated with creosote in accordance with the American Wood Preservers' Association (AWPA). The bridge deck remained fully-exposed to weathering without a recommended waterproofing membrane in conjunction with an asphalt wearing surface. The stressing bar system is composed of six, 25 mm (1 in) diameter, high-strength steel bars spaced at 1.2 m (48 in) on center. The design force in the bars was such that the imposed uniform interlaminar compressive stress would be 690 kN/m^2 (100 psi). This represents a bar force of 299 kN (67,200 lb).

Figure 5 *Hibbsville stress-lam bridge.*

5.2 HOFFMAN RUN

The Hoffman Run Bridge, constructed in 1990, is located just outside Dahoga, Pennsylvania on Township Road T364 (41° 35'53.21"N, 78° 43'39.67"W) which provides access to local residences and recreation areas within the adjacent Allegheny National Forest. The decision to construct this stress-laminated timber bridge was made, in large part, due to the success of the Little Salmon Creek Bridge constructed nearby only a couple of years earlier. The bridge design was based on a draft version of Timber Bridges: Design, Construction, Inspection, and Maintenance (Ritter 1990) and the Standard Specifications of Highway Bridges (AASHTO 1983) from which the design loading HS 20-44 was used. The bridge is composed of a 7.9 m (26 ft) long by 4.9 m (16 ft) wide, stress-laminated superstructure made up of 305 mm (12 in) deep, visually graded No. 2 or better mixed hardwoods, mostly beech, sawn lumber laminations. Butt joints were placed at every fourth lamination with a 1.2 m (4 ft) longitudinal spacing between butt joints in adjacent laminations. Timber components were treated with creosote in accordance with the American Wood Preservers' Association (AWPA) Standard. The bridge has no wearing surface.

The stressing bar system is composed of seven, 25 mm (1 in) diameter, high-strength steel bars spaced at 1.1 m (45 in) on center. The design force in the bars was such that the imposed uniform interlaminar compressive stress would be 827 kN/m^2 (120 psi). This represents a bar force of 289 kN (65,000 lb).

Figure 6 *Hoffman Run stress-lam bridge.*

6 RESULTS AND DISCUSSION

6.1 CONDITION ASSESSMENT

6.1.1 Hibbsville Bridge

Inspection of the geometry of the Hibbsville Bridge indicates the stressing bar forces are too low preventing the timber laminations from acting as a unit connected by interlaminar friction. That is, the interlaminar stress is low enough to allow sliding between laminations on the vertical plane. As shown in Figure 7, laminations are frequently offset vertically at the butt joints up to 25 mm (1 in) and as much as 38 mm (1.5 in) in some locations. Longitudinal separation of the butt ends by 13mm (1.5 in) to 19 mm (0.75 in) was common and as much as 25 mm (1 in) in some locations (see Figure 8). The plane of the bottom side of the deck is uneven with laminations consistently varying from those immediately adjacent. The unevenness of the deck laminations was also apparent on the topside since there is no wearing surface, only some gravel debris, to obstruct viewing of the deck. Even with the low interlaminar stress, there were no noticeable separations between laminations.

Figure 7 *Hibbsville deck underside*

Figure 8 *Hibbsville gapping at lamination butt joints*

A sag in the bridge deck was visually apparent and was quantified along a single line mid-width of the bridge. The measured values are shown in Figure 9.

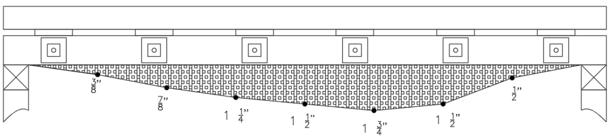

Figure 9 *Sag at mid-width of Hibbsville Bridge (1 in = 25.4 mm)*

Additionally, the deck ends did not fully bear on the pile caps; the deck plane deflected upward at the corners. The measured upward movement with respect to the pile cap is 32 mm (1.25 in), 13 mm (0.5 in), 29 mm (1.13 in), and 13 mm (0.5 in), at the NW, SW, SE, and NE corners, respectively. A good example of this occurrence is shown in Figure 10. It is not clear what causes these distortions but is likely linked to the moisture accumulation along the curbs.

Figure 10 *Hibbsville deck uplift at corner bearing*

The stressing bar bearing plate assembly is comprised of multiple discrete steel bearing plates. Aside from some minor corrosion, each appeared in good condition. The galvanized stressing bars and nuts were also void of notable corrosion.

The outermost laminations did not have a waviness along its length reflecting the stressing bar locations like many other bridges. However, as a general rule, many laminations appeared wavy with no clear patterning.

Plant growth, pictured in Figure 11, was present at the curb lines and on top of the pile cap extensions beyond the bridge deck edge. Gravel and soil from the roadway on which this bridge is located is being tracked onto the bridge and accumulating in these areas. The soils and their inherent ability to retain moisture provide an environment conducive to ongoing plant growth and premature deck decay.

Figure 11 *Hibbsville curb line vegetation*

6.1.2 Hoffman Run Bridge

Visual inspection of the Hoffman Run Bridge revealed minor distortion in the stress-laminated bridge deck. The deck laminations did not fully bear on the abutment wall on both the north and south ends. Rather, uplift was present at three of the four corners totalling 13 mm (0.5 in), 19mm (0.75 in), and 19 mm (0.75 in) at the NW, NE, and SW corners, respectively (see Figure 12). In addition, a stringline was extended between the bottoms of the midspan guardrail posts which showed the bottom side plane varied in the transverse direction. A differential elevation of 35 mm (1.38 in) between the highest and lowest points on the deck was measured.

Figure 12 *Hoffman Run deck uplift at corner bearing*

As shown in Figure 13, longitudinal undulations were observed in the outer laminations. Depressions inward were observed at the stressing bar locations. The galvanized stressing bars remain in good condition. Checking was present along the outer lamination on the downstream side. The remaining laminations appeared in good condition with no apparent areas of decay or distress despite evidence of water seepage through the deck seen by the water beads on the deck underside. See Figure 14.

Figure 13 *Hoffman Run outer lamination waviness*

Figure 14 *Hoffman Run underside water accumulation*

The topside, pictured in Figure 15, showed evidence of vehicle wear and abrasion on the wheel lines. Minor rutting or depressions at these locations was observed. There is a stop sign at the north end of the bridge and the braking action of vehicles is believed to contribute to the visible wear patterns. The butt joints between laminations were consistently gapped approximately 25.4 mm (1 in) to 32 mm (1.25 in) and, in almost all cases, the gaps were packed with soil.

Figure 15 *Hoffman Run topside of deck*

The topside was generally free of soil and debris with the exception being at the curb lines where soils have accumulated and hold moisture against the outermost laminations and curb. The west curb especially has experience significant decay as shown in Figure 16.

Figure 16 *Hoffman Run debris accumulation leading to significant decay of curb members.*

6.2 BAR FORCE

The results of the bar force measurement for the Hibbsville and Hoffman Run bridges are presented in Table 2. The original stressing bar design force is shown along with the percentage of design force that remains in the bar obtained through the measurement task. One can observe the remaining bar force is greatly reduced from the original design force.

In addition, Figure 17 and Figure 18 show the average bar force measured during the initial monitoring period post-construction for Hibbsville and Hoffman Run, respectively. These figures indicate a similar reduction in stressing bar force during a short period of time after initial or subsequent tightening of the bars.

__Table 2__ Results of bar force measurement at both bridges.

	Measured Bar Force (kN/kips) and Percentage of Design Force			
	Hibbsville (299/67.2 design)		Hoffman Run (289/65.0 design)	
Location				
1	150/33.8	50%	117/26.3	40%
2	167/37.6	56%	133/30.0	46%
3	145/32.6	49%	142/32.0	49%
4	156/35.0	52%	139/31.3	48%
5	133/30.0	45%	142/32.0	49%
6	145/32.6	49%	117/26.3	40%
7	-	-	106/23.8	37%

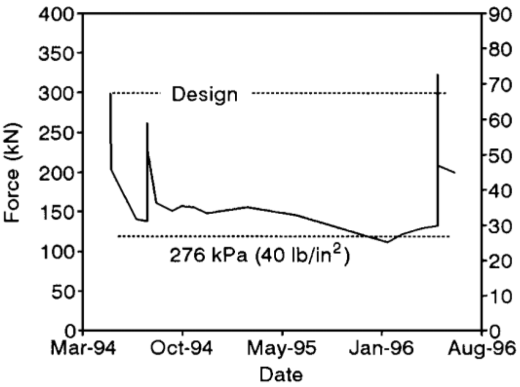

***Figure 17** Average bar force for Hibbsville 1994 – 1996.*

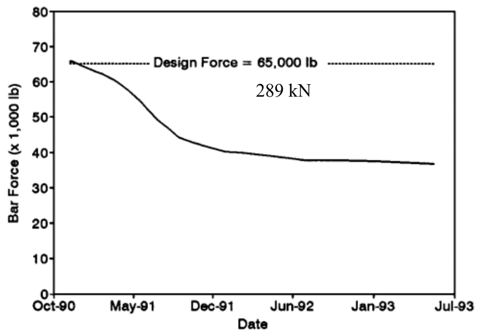

***Figure 18** Average Bar Force for Hoffman Run 1990 – 1993*

6.3 MOISTURE MEASUREMENT AND MICRO-DRILL RESISTANCE RESULTS

The moisture measurement and micro-drill resistance data collection locations at each respective bridge are shown in Figure 19 through Figure 21.

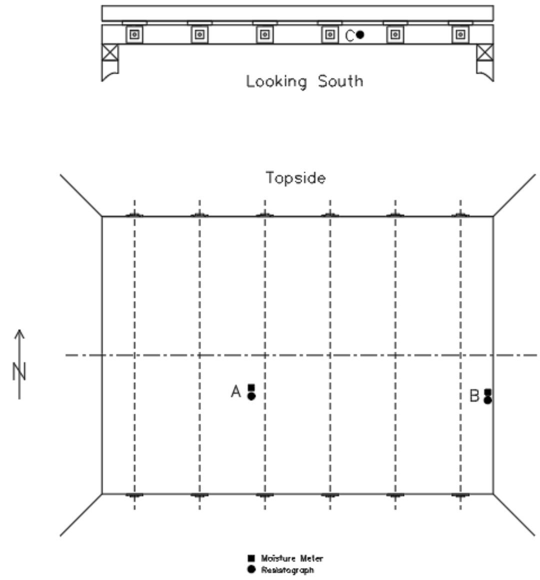

***Figure 19** Hibbsville moisture and resistance micro-drill locations*

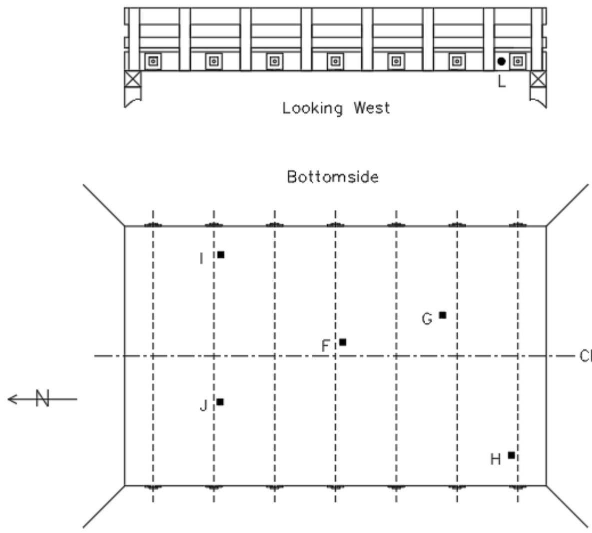

***Figure 20** Hoffman Run moisture and micro-drill resistance locations*

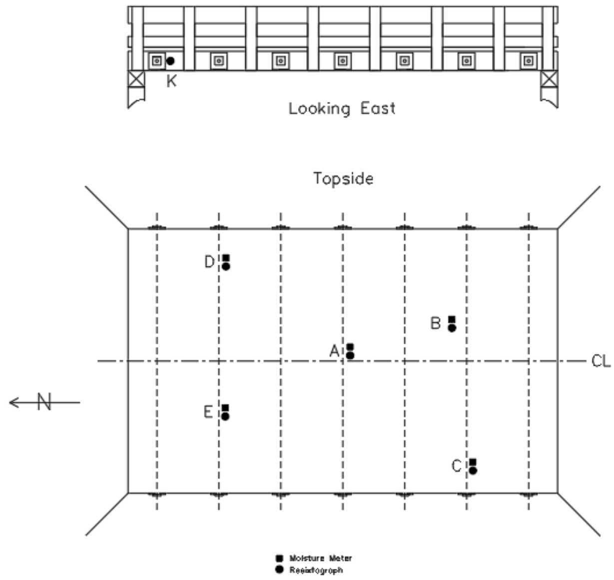

Figure 21 *Hoffman Run moisture and micro-drill resistance locations*

6.3.1 Moisture Measurements

The moisture content was measured at several locations and depths 38 mm and 76 mm (1.5 in and 3 in) on each bridge. Table 1 summarizes the measurements for each bridge. The results for the Hibbsville bridge indicate the timber is below the fiber saturation point of approximately 25 to 30 percent. Conversely, most measurements taken at the Hoffman Run bridge exceed the fiber saturation point.

Table 3 *Moisture measurement results*

Location	Adjusted Moisture Content (%)			
	Hibbsville		Hoffman Run	
Probe Depth	38 mm (1.5 in)	76 mm (3 in)	38 mm (1.5 in)	76 mm (3 in)
A	20.7	22.5	28.0	35.0
B	21.9	23.2	28.0	33.0
C	22.4	23.6	25.0	27.0
D	-	-	32.0	31.0
E	-	-	38.0	39.0
F	-	-	35.9	32.7
G	-	-	20.7	22.9
H	-	-	27.7	30.6
I	-	-	32.6	35.4
J	-	-	30.0	31.2

The previous monitoring studies also recorded moisture percentage at several locations. The average moisture percentage is presented in the final reports. The values and qualifying information follow in the short excerpts taken from those reports.

<u>Hibbsville</u>

The average lamination moisture content of the Hibbsville bridge was approximately 24 percent at the beginning and end of monitoring. Measurements indicated that the moisture content level remained relatively stable throughout the monitoring period, although there were fluctuations of 2 to 3 percent in the measurement zone as a result of seasonal climatic changes. The primary contributing factor to the continued high moisture content was the absence of a watertight membrane over the surface of the deck. Water directly contacts the timber deck. Furthermore, drying of the deck is inhibited by dirt and debris accumulations on the deck. [1]

<u>Hoffman Run</u>

Average lamination moisture content of the Hoffman Run Bridge was approximately 42 percent in December 1990, 2 months after installation. From that time, the bridge experienced a gradual decrease in moisture content to approximately 32 percent at the conclusion of the monitoring. The 10 percent decrease in moisture content occurred at a relatively constant rate during the 32 months, although there were fluctuations of 2 to 3 percent in the measurement zone as a result of seasonal climate changes. It is expected that the moisture content in the interior of the laminations was greater than the values obtained in the measurement zone as a result of slower moisture migration through the lamination depth.

Because of the high moisture content at the time of installation, the deck moisture content remained above the fiber saturation moisture content, which is approximately 25 to 30 percent. Above this level, changes in the moisture content do not cause dimensional changes in the wood laminations. In time, the moisture content will gradually decrease and eventually stabilize below the fiber saturation point. When the moisture content of the bridge falls below fiber saturation, subsequent moisture loss will result in shrinkage of the laminations and could significantly influence bar force retention. [2]

6.3.2 Micro-drill Resistance Results

Micro-drilling resistance measurements were taken at several locations at the Hibbsville and Hoffman Run bridges. The results are plotted in Figure 22 showing the relative resistance through the depth of the drill bit penetration. The depth of drilling and drilling resistance are plotted along the horizontal and vertical axes, respectively. One should note any abrupt changes to the drilling resistance and assume these areas to be voids (if no resistance) or decayed (if reduced resistance).

The results for each bridge generally show the timber is sound with few areas of reduced density. Specifically, Hoffman Run locations E, K, and L show areas where the density changes. The orientation of the drilling at location E is vertical (from the top side downward) which indicates the bit is penetrating a single timber lamination across its depth. This is pointed out by the arrows. A small area may be experiencing the onset of decay. The orientation of the drilling at locations K and L is horizontal (from outside inward at deck edge) which indicates the bit is penetrating multiple timber laminations across their widths. The reduction in density pointed out by the arrow is likely a result of reduced density in a single timber lamination.

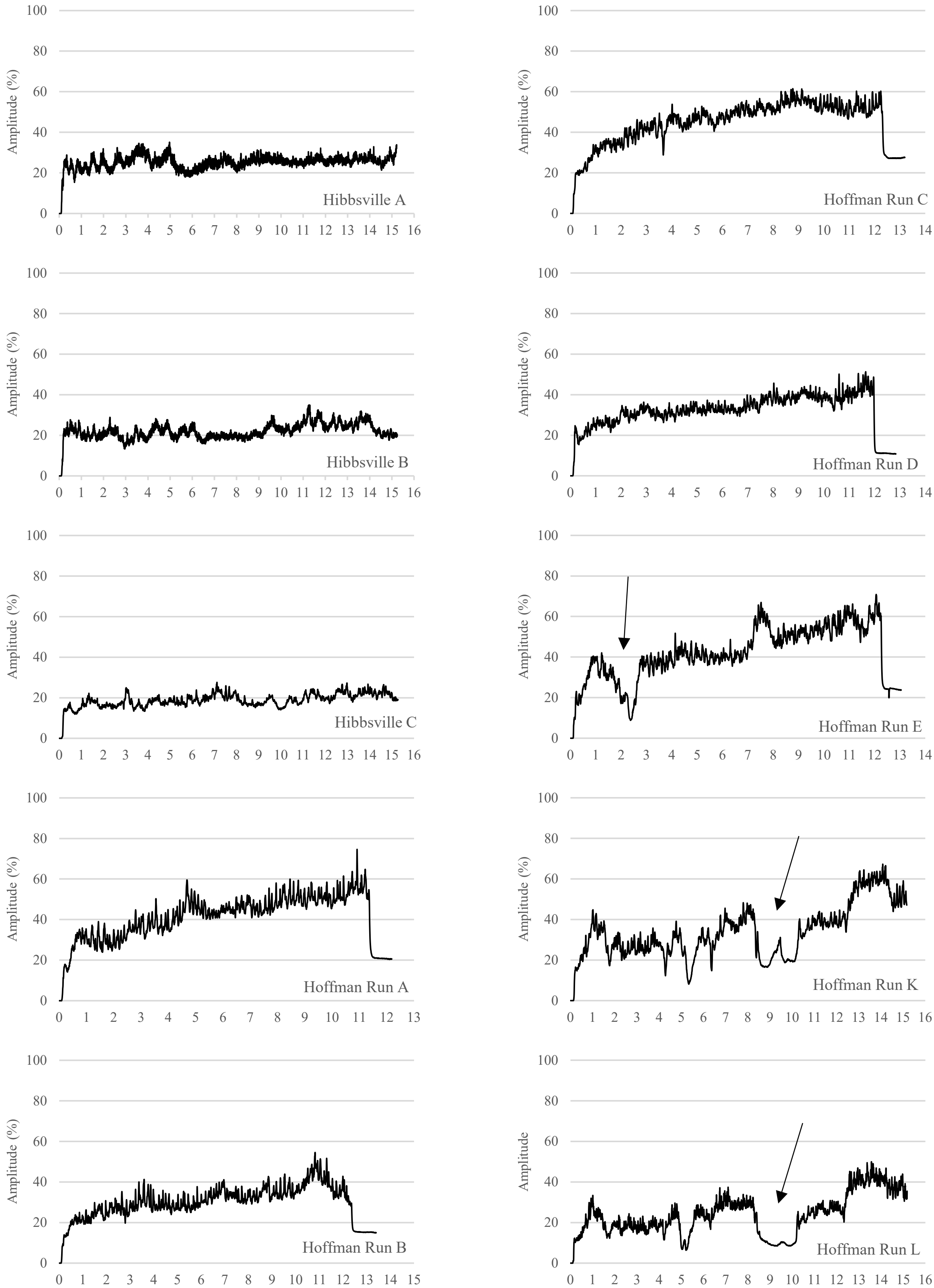

***Figure 22** Hibbsville and Hoffman Run micro-drill resistance measurements versus depth (inches) of penetration (1 in = 25.4 mm)*

7 CONCLUSIONS

The following conclusions are based on inspections completed during June of 2019 and with respect to previous inspections and load tests completed on these same bridges more than 20 years earlier.

The field performance of the seven stress-laminated deck bridges can be categorized as having reached varying levels of unsatisfactory performance. Maintenance has been lacking and attention to these items is important to their overall field performance. Previous studies indicate a reduction in bar tension force within the couple of years following construction. It is recommended the bars be re-tensioned periodically (every 2 to 5 years) to ensure interlaminar stresses are great enough to ensure loads are distributed across the bridge deck in an efficient manner. Guidance and instruction for retensioning the bars was produced in the form of a video and is included along with the comprehensive report for this project [4].

When bar forces were measured, the in-situ forces were lower than the design force. The lower threshold for lamination slippage is at 40 percent of the full design force levels. The measured forces for the Hibbsville and Hoffman Run bridges ranged from 37 to 56 percent, which is at or near the accepted threshold. For this reason, retensioning of the stressing bars is recommended for both bridges. Reduced bar forces may prevent the timber laminations from acting fully as friction members. That is, the interlaminar stress is low enough to allow sliding between laminations on the vertical plane. This can be a potential safety hazard as the structural integrity of the stress-laminated superstructure deck may become compromised.

In several instances, the moisture content measured at various locations on these bridges is elevated above the fiber saturation point. In addition to creating a condition conducive to biological growth, timber with high moisture content is susceptible to an accelerated degradation. Moreover, elevated moisture content in deck laminations has been previously shown to reduce the force within the transverse tensioning bars on similar bridges. Despite these observations, it should be noted the Hibbsville and Hoffman Run bridges are not protected by a waterproof asphalt layer as some of the other bridges in the study were. For this reason it is expected the moisture content measurements would be higher. Generally, the measurements were approximately the same as at the time of construction with slightly dryer readings internally at 76 mm (3 in).

As a general maintenance practice it is advised to remove debris from the deck surface and curb lines. Debris can impede deck drainage and trap moisture against the deck surface and/or curb line, thereby increasing the moisture content and creating an environment more suitable to accelerated degradation. Due to advanced degradation, it is recommended the curbs at Hoffman Run be replaced.

Micro-drill resistance measurements indicated areas of reduced density or voids in some areas. Though the overall condition of the timber deck laminations were not in an advanced state of deterioration, one should note there are a few areas where localized degradation has occurred.

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