Live Load Response of Short Span Bridges with Parallam® Decks

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Abstract

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By
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Structural Composite Lumber (SCL) is reconstituted with high grade presorted veneers to enhance properties including higher and more uniform strength and stiffness than conventional lumber. Parallel Strand Lumber (PSL) is mainly constituted of wood strands of up to 66 ft long bonded together using an adhesive under pressure. Different structural elements including plates and beams can be produced from PSL. PSL is free of natural wood defects such as checks, knots and decay, and less susceptible to water since the adhesive used in the manufacturing process is water resistant.

The mechanical characterization of Parallam was performed through various testing methods that included bending, shear, aging and fatigue. CFC-WVU developed a design procedure for bridge decks utilizing Parallam composite wood panels, manufactured by Trus Joist, Buckhannon, WV. Spring connectors were designed and tested before using them as mechanical means to connect the Parallam deck to steel stringers.

After establishing the mechanical properties of the Parallam, CFC utilized its expertise to design two short span bridges: Peel Tree and Hackers Creek bridges.

Peel Tree Bridge is simply supported with a span of 29 ft. It carries a single traffic lane on county road 20/3 in Barbour County, WV consisting of five W14 × 90 stringers spaced at 2'-9" and the bridge width is 15 ft. The bridge was built in FY2005.

Hackers Creek Bridge is simply supported with a span of 31 ft. It carries a single traffic lane on county road 119/2 in Barbour County, WV. The bridge consists of three W24x94 steel stringers with 6 ft c/c spacing. Hackers Creek Bridge was built in FY2005. Both the bridges are designed to carry AASHTO HS-25 truck load.

After construction, CFC-WVU started a monitoring program to evaluate the load carrying capacity and serviceability of the two bridges. Four live load tests (static and dynamic) were conducted on each bridge using dump trucks provided by the WVDOH. The tests were performed mainly to evaluate the Dynamic Load Allowance, Transverse Load Distribution, Live Load Deflection as well as the stresses under service conditions. Also, visual inspection and moisture measurements in decks were performed to evaluate the overall service condition of the two bridge decks and the deck-to-stringer connectors.

Static deflections of the two bridges were within the allowable limits under a truck load 15% higher than HS-25 AASHTO load. Also, the dynamic as well as the static strains were within the allowable limits under the test truck load.

Visual inspection of Peel Tree Bridge revealed wear problems at the top of the deck due to lack of wearing surface. Hackers Creek Bridge had nine failed deck-to-stringer connectors. Moisture content measurements were taken on the decks of both bridges. The average moisture content of both decks was below the 19% limit provided by AASHTO bridge design specifications (LRFD & Standard).

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I dedicate this study to god and my family especially my parents for all the love and support

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Chapter1

Introduction

1.1Background

Bridge owners and designers are faced with the challenges of accelerating rates of deterioration in bridge decks, which would affect structural integrity and safety for traveling public. The deterioration rates are factors of: 1) load intensity, 2) load frequency, 3) environmental considerations, 4) design and construction quality, 5) material type and resistance and 6) many other factors. It is important to recognize the effect of time dependent bridge deck damage on its serviceability. Time dependent damage can be caused by the cyclic wheel load effect, leading to fatigue related problems. Also, atmospheric conditions such as freezing and thawing, sudden changes in temperature over different hours of a day, and chemical (pH variation) attacks on bridge deck materials can lead to significant reductions in the service life of a bridge deck. Hence, the need arises to develop new materials of high performance. These new materials must be less susceptible to weathering while simultaneously maintaining minimum strength and stiffness over a deck's service life.

The Constructed Facilities Center (CFC) at West Virginia University (WVU) suggested an alternative high performance material for bridge deck replacement utilizing Parallam® or PSL panels manufactured by Trus Joist, Inc., Buckhannon, WV. CFC-WVU has done extensive testing on PSL engineered wood (Parallam®) beams and decks (Smith, 2003). The mechanical characterization of parallam® was performed under bending, shear, aging and fatigue. CFC-WVU has developed a standard design procedure for bridge decks utilizing Parallam® composite wood panels. Spring connectors were designed and tested as mechanical means to connect the Parallam deck to steel stringers in addition to dowels as deck to deck connectors (Smith, 2003). Figures 1 and 2 show a schematic of the spring connector designed by CFC-WVU.

After validating the mechanical properties of the Parallam®, CFC-WVU utilized it in the deck design of many short span bridges including Peel Tree and Hackers Creek bridges. After their construction, CFC-WVU started a monitoring program to evaluate the load carrying capacity, aging and the serviceability responses of these two bridges.

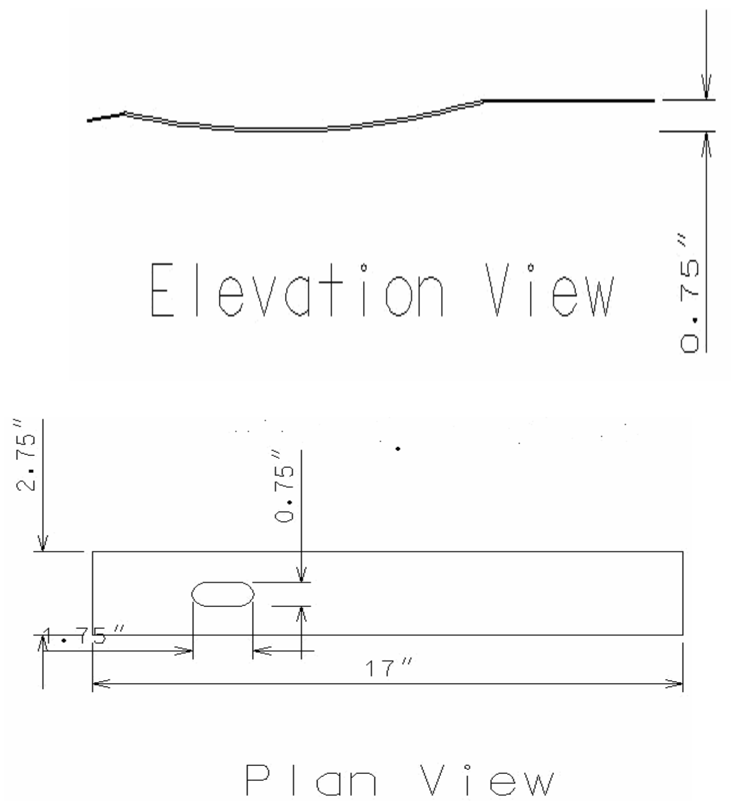


Figure 1: WVU Spring connector (Smith, 2003)

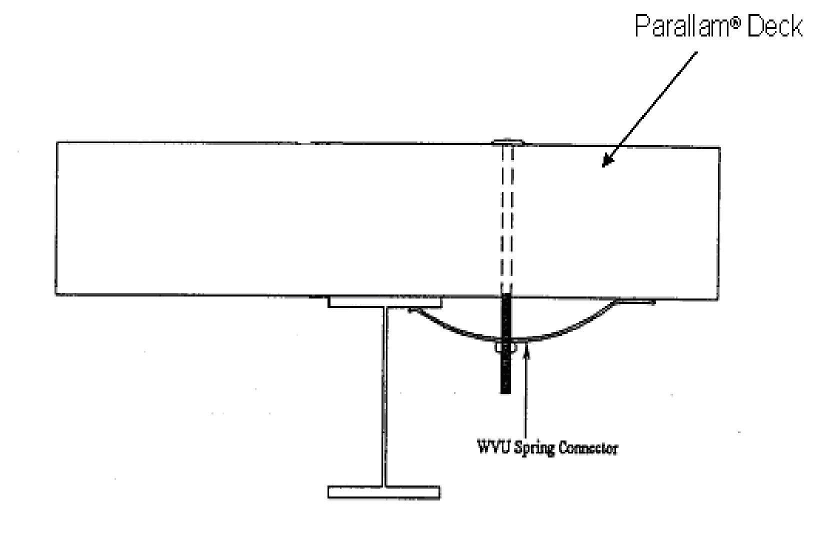


Figure 2: CFC-WVU Spring Connector attaching Parallam deck to stringer (Smith, 2003)

1.2 Objective

The objective of this study is to evaluate the performance of Peel Tree and Hackers Creek bridges under service conditions. The evaluation process aims at determining the load carrying capacity including dynamic load allowance, and the serviceability aspects of the two bridges. These serviceability issues include: load induced stresses and strains, deflections, visual distress, creep and deck rutting. The analyses quantify several important bridge design parameters including: Dynamic Load Allowance (DLA), Transverse Load Distribution Factors (TLDF), deck and stringer deflections and the actual stringer and deck strains/stresses, moisture up-take, creep and any deck surface distress.

1.3 Scope

The second chapter of this report starts with a literature review of the Dynamic Load Allowance provisions in the old and current bridge design specifications. Existing work done on dynamic and static testing of bridges is also reviewed. Moreover, chapter two discusses the work done by CFC-WVU engineers to validate the mechanical properties of the structural composite lumber (Parallam®).

The third chapter discusses in detail the field monitoring program conducted by CFC-WVU to evaluate the performance of Peel Tree and Hackers Creek bridges. This chapter presents a detailed description of the two bridges, instrumentation, test procedures and the field test data and evaluation. The monitoring program is performed through a series of live load tests using dump trucks provided by WVDOH. Also, the monitoring program included visual inspection, deck moisture content measurements, and deck creep measurements. The two bridges were instrumented with strain gages and Linear Variable Differential Transducers (LVDTs) at preselected locations. The acquired field test data have been used as basis for their performance evaluation.

Chapter 2
Literature Review

2.1 Introduction

This chapter presents a critical review of the Dynamic Load Allowance in bridge design codes as well as a review of the available literature on the static and dynamic load testing of bridges and interpretation of field data. General discussion is presented about the Structural Composite Lumber (SCL) in addition to a summary of mechanical properties of Parallam® (Smith, 2003).

2.2 Dynamic load Allowance in Bridge Design Codes

Design codes paid great deal of attention to dynamic load effects of moving vehicles on bridges. As early as 1927 a joint committee of the American Association of State Highway Transportation Officials (AASHTO) and the American Railway Engineering Association (AREA) recommended the use of an impact factor (I) computed as a function of span length, which is:

$$I = \frac{50}{(L + 125)} \qquad (2.1)$$

Where, I= the impact factor not to exceed 30%, L= length in feet for the portion of the span that is loaded to produce maximum stress in a member. The AASHTO LRFD bridge design specifications mandated the use of "the Dynamic Load Allowance (IM) as an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles" and it recommends a DLA value of 0.75 for deck joints, 0.15 for fatigue and fracture limit states and

0.33 for all other limit states. The AASHTO LRFD procedure attributed the effects of a moving vehicle to two sources:

1. Hammering effect is the dynamic response of the wheel assembly to riding surface discontinuities, such as deck joints, cracks, potholes and delaminations."

2." Dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle."

The Canadian government conducted a series of full scale dynamic tests on bridges in 1956-1957 and in 1969-1971 to correlate the DLA to the first flexural frequency of a bridge. The study revealed an increase in the DLA for bridges with a fundamental frequency in the range of 2-5 Hz which happens to be the pitch and bounce frequency for the test trucks. The experimental findings of the study were published in the 1979 edition of the Ontario Highway Bridge Design Code (OHBDC, 1979) as a relationship between the DLA values and the fundamental flexural frequency of a bridge. Further testing was performed to calibrate the results of the previous study to modern bridges and vehicles. It was concluded that reductions could be made to DLA factors of 1979, and new provisions were published in the 1983 edition of the OHBDC.

2.3 Dynamic Load Allowance Evaluation from Field Testing

Dynamic bridge testing has been used for a long time to evaluate the dynamic properties of highway bridges (Bakht and Pinjarkar, 1989). Natural frequencies, mode shapes, damping ratios and impact factors are the targeted parameters in most tests performed by different researchers. However, there is a great deal of confusion about how to obtain DLA from test data. This confusion as reported by Bakht and Pinjarkar (1989) comes from the fact that "There is no uniformity in the manner by which this increment is calculated from test data". They compared the DLA values calculated from different definitions for the same set of data and concluded that" The definition of *I* is far from axiomatic" where *I* refers to the impact factor. They also arrived at the following conclusions:

- The most accurate definition of the impact increment is :

$$I = \frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \quad (2.2)$$

Where: I = impact increment of deflection, δ_{dyn} = maximum deflection under the vehicle traveling at normal speed, δ_{stat} = maximum deflection under the vehicle traveling at crawling speed.

This conclusion is based on the fact that *I* computed from equation (2.2) returned the same value of δ_{dyn} as that measured in the field when substituted into equation (2.3):

$$\delta_{dyn} = \delta_{stat}(1 + I) \quad (2.3)$$

- "The impact factor is not a tangible entity susceptible to deterministic evaluation; it can be accounted for in the design by a probabilistic approach" (Bakht and Pinjarkar, 1989).

- The maximum dynamic response and the maximum static response do not occur at the same load position.

Neely et al. (2004) conducted in-service evaluation of a two lane Fiber Reinforced Polymer (FRP) bridge superstructure. The study involved a series of load tests using three axle dump trucks passing at 20 mph, 25 mph and 40 mph. The results of the load tests showed that a DLA value as high as 0.90 was obtained from the FRP girders deflection measurements which was higher than the strain based DLA values. This result is inconsistent with the results obtained from testing both Peel Tree Bridge and Hackers Creek Bridge which showed higher strain based DLA values. The study also concluded that “there is little, if any, dynamic amplification of an applied load on the bridge due to a vehicle moving at 25 mph. However, at 40 mph, the composite bridge exhibits a relatively large dynamic response”. This conclusion suggests that it is reasonable to consider the response of a bridge to a vehicle passing at 20 mph or 25 mph as a static type response.

Laman et al (1999) evaluated the static and dynamic stresses induced in three through Trus bridges made of steel. Dynamic strain data were collected under controlled and normal traffic conditions for different structural components of the three bridges. The authors concluded that the “ DLA is dependent on truck location, component location, component type, and component peak static stress but appears to be nearly independent of the truck speed”.

Aluri et al (2005) investigated the dynamic response of three FRP bridge decks stiffened with steel stringers. The authors conducted a series of static and dynamic tests on the three bridges and collected deflection, strain and deck acceleration data to evaluate DLA’s, natural frequencies and damping ratios. The test results showed a DLA value as high as 0.93 which is higher than the AASHTO recommended value of 0.33. The authors noted that the response of a bridge at 2mph cannot be considered as a true static response in all cases. This conclusion is based on an observation of a significant amplification in strains at 2mph when compared to the static strains (i.e. the test vehicle is not moving). The authors also reported excessive vibration problems that had been attributed to the low damping of FRP bridge decks and to rough bridge approach conditions.

Nassif et al (2003) compared DLA obtained from a 3D finite element model of a simple span bridge under variable truck load conditions (i.e. variable speed, truck weight, loaded lanes) to DLA measured from load test data. The authors observed that the DLA decreases with the increase in static stress. Also, they concluded that exterior girders exhibited higher DLA due to relatively smaller static load effects. A recommendation was made to take the design DLA as that of the heavily loaded interior girders. However, the authors suggested that the higher DLA values shall be used for design cases where fatigue behavior is dominant.

Wipf et al. (1999) investigated the response of glue laminated timber girder bridges under heavy truck loads. The investigation involved field tests of 16 different bridges where deflection measurements were collected. The study revealed that bridges with rough approach conditions exhibited a DLA value as high as 0.60, where as bridges with relatively smooth approach conditions exhibited a maximum DLA value of 0.18.

2.4 Structural Composite Lumber

Structural Composite Lumber (SCL) refers to a group of engineered wood products that utilize wood veneer or strands in combination with a structural adhesive to produce full scale structural members. Since its introduction in the market, SCL has achieved increasing gains due to its superiority over the natural wood. SCL has proven to be a versatile material since it can be produced virtually in any required structural shape. Also, SCL is free of the defects associated with natural wood such as checks, knots and decay. Moreover, SCL has shown less variability in strength characteristics; thus leading to a higher strength consistency and reliability.

A very common type of SCL is the Laminated Veneer Lumber which consists of continuous veneer sheets or laminae joined together using an adhesive such as Phenol-formaldehyde. The wood veneer is bonded in such a way that the grain is parallel to the longitudinal direction of the member. Other types of SCL include Parallel Strand Lumber (PSL) and Laminated Strand Lumber (LSL) manufactured using bonded long wood strands. The only difference between PSL and LSL is that the later utilizes thinner and wider wood strands.

The type of engineered wood considered herein is called Parallam® which is manufactured by Trus Joist, Inc. at a plant in Buckhannon, West Virginia. The primary species used to manufacture Parallam PSL at the Buckhannon plant is Yellow Poplar.

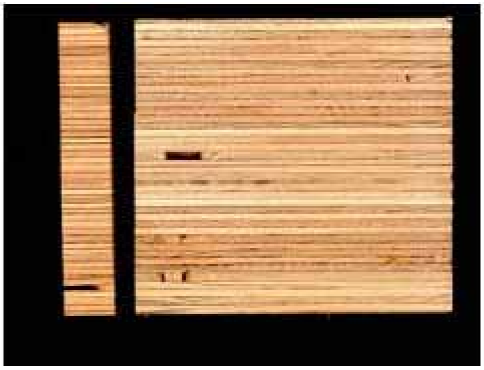
A manual provided by Trus Joist reported that the recomposing of the wood eliminates or minimizes the strength reducing characteristics such as knots and low density. Also, Trus Joist, Inc. provided mechanical material properties based on statistical reliability. The manual states that 95% of all the material must meet or exceed the given performance levels. Moreover, the manual provided a comparison of design stresses between sawn lumber, Glulam and Parallam (Table 2.1). It is very clear that Parallam out performs sawn lumber and glulam. In addition, Trus Joist manual provided wet service strength reduction factors (Table 2.2). This issue is very critical since moisture fluctuations affect the mechanical properties of wood materials, especially in bridge applications.

Table 2.1: Mechanical Properties Comparison (Trus Joist, Inc. Manual)

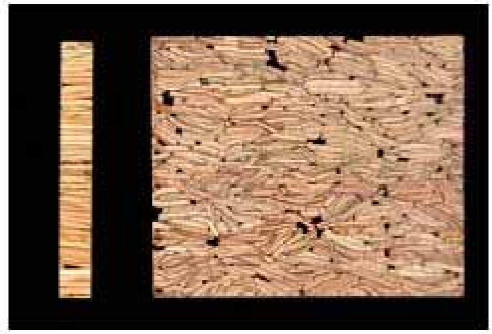
	Douglas-fir#1 beam and stringer	Glulam 24f-V11	Parallam PSL
Max. Bending Stress (psi)	1,300	2,450	2,900
Max. Shear Stress (psi)	85	155	290
MOE × 10^6 (psi)	1.6	1.7	2.0

Table 2.2: Wet Service Factors for Parallam PSL (Trus Joist, Inc. Manual)

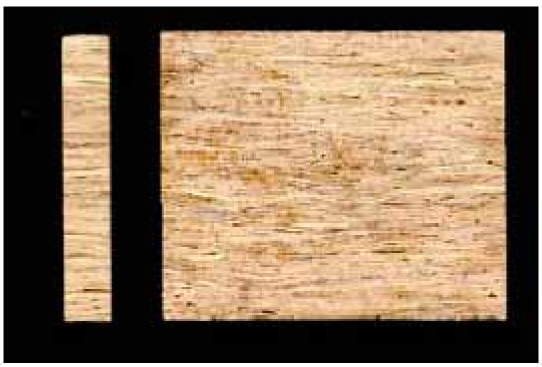
Bending Stress	Shear Stress	MOE
0.8	0.878	0.833



(a) Laminated Veneer Lumber.



(b) Parallel Strand Lumber.



(c) Laminated Strand Lumber.

Figure 2.1: Types of structural composite lumber. (http://courses.forestry.ubc.ca)

2.5 Laboratory Testing of Parallam® Decks

The Constructed Facilities Center at West Virginia University (CFC-WVU) investigated the mechanical properties of the Parallam bridge deck panels. Smith (2003) conducted laboratory testing to evaluate the design material properties, aging and long term performance of Parallam®. This section of the literature review is a summary of the work done by Smith (2003).

2.5.1 Strand Orientation

Parallam® is manufactured with the strands oriented in one direction, thus it is very important to understand the variation in the mechanical properties with strand direction. Figure 2.2 defines the X, Y and Z directions for the cross-section of a Parallam® beam. It can be seen from Figure 2.2 that the X direction is parallel to the wide face of the strands, the Y direction is perpendicular to the wide face of the strands and Z direction runs along the beam parallel to the strand direction.

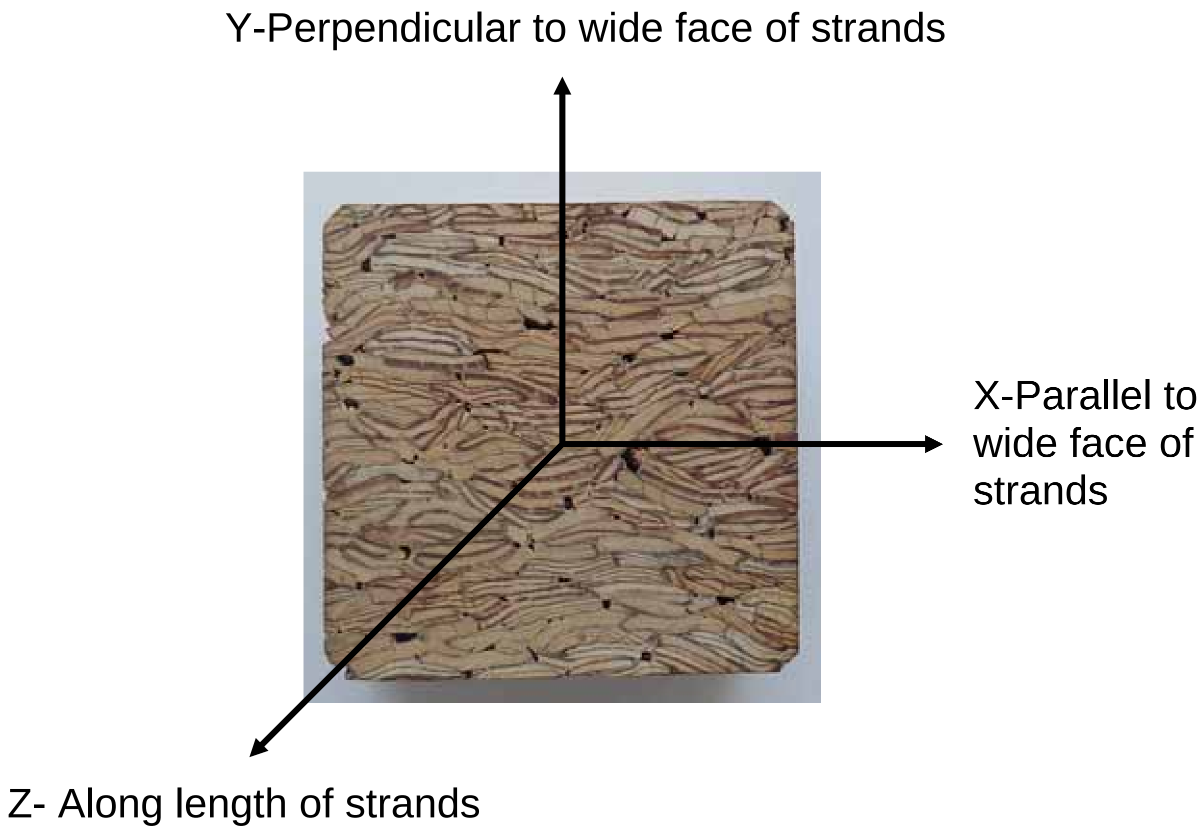


Figure 2.2: Strand orientation in Parallam® (Smith, 2003).

2.5.2 Aging of Test Samples

It is very significant to account for the aging process of any material used for the design of civil infrastructures. Freezing, thawing, chemical attack and temperature gradient are factors that contribute to the degradation in the mechanical properties of the material under service conditions. Smith (2003) developed an accelerated aging procedure based on ASTM standard 1101 to simulate 20-25 years of service life conditions. The primary purpose of this accelerated aging technique is to establish the durability of the bond-line of the phenol-formaldehyde adhesive between the Parallam® strands.

The aged samples have been subjected to six cycles of swelling and shrinkage. Each cycles consisted of the following steps:

1. Soaking of samples in water at a vacuum of 25 inch Hg at room temperature for 30 minutes;
2. Soaking of samples in water at a pressure of 100 Psi for 30 minutes;
3. Freezing at a temperature of 15 °F for 2 hours; and
4. Oven drying at a temperature of about 150 °F for 8 hours.

The vacuum soaking and the pressure soaking creates differential expansion of the strands and the bond-line leading to severe shrinkage and swelling.

2.5.3 Small Scale Bending Tests

Smith tested a total of (24) samples under three point-bending using an Instron Machine. The samples dimensions were (1/2”× 1/2" × 10”). Twelve (12) of the twenty four (24) samples were creosote treated, and the remaining twelve were untreated. Six (6) treated samples and six (6) untreated sample were subjected to the aging process described earlier. Load-Deflection curves were plotted to extract the modulus of elasticity (MOE) and the modulus of rupture (MOR). Table 2.3 summarizes the test results in terms of the modulus of elasticity and the modulus of rupture (i.e. failure stress in bending for the outer most fibers).

Table 2.3 Average modulus of elasticity and modulus of rupture (smith, 2003)

	Avg MOR (psi)	MOE (psi)
Aged Creosote-Treated Y-Direction	8727	1.37E+06
Aged Creosote-Treated X-Direction	7506	1.74E+06
Aged Non-Treated Y-Direction	11023	1.62E+06
Aged Non-Treated X-Direction	11274	N/A
Non-aged Creosote-Treated Y-Direction	9682	1.41E+06
Non-aged Creosote-Treated X-Direction	8663	1.87E+06
Non-aged Non-Treated Y-Direction	13057	1.72E+06
Non-aged Non-Treated X-Direction	12229	2.61E+06

Trus Joist, Inc. reported a maximum bending stress and modulus of elasticity of 2,900 psi and 2,000,000 psi respectively. A comparison with the test data shows that the actual failure stress is higher than that provided by Trus Joist, Inc for both aged and no-aged samples. Moreover, the comparison shows that the modulus of elasticity from the test data is in very good agreement with the modulus of elasticity reported by Trus Joist for the non-aged samples.

2.5.4 Small Scale Shear Tests

The shear strength of the Parallam in both the X and Y directions was evaluated according to ASTM D 143 using the shear block method. The shear blocks are placed in a specific jig, where an equal pressure is applied across the length of the step raising at a constant rate until fracture is obtained. The machine then produces the maximum stress sustained by the specimen. The results of the shear strength are summarized in Table 2.4.

Table 2.4 Shear strength values from shear block (smith, 2003)

	Shear Strength (psi)
Shear Parallel to the Strand (X-Direction)	738
Shear Parallel to the Strand (Y-Direction)	635

Twenty four (24) small scale beams (1.75”×1.75” × 11”) were tested to failure under three point bending to evaluate the shear strength. Half of the beams were aged and the remaining half was un-aged. The average failure loads and average shear strength is shown in table 2.5.

Table 2.5: Average failure load and shear strength (smith, 2003).

Group	Average Failure Load P (kips)	Average Shear Strength τ (psi)
Control (unaged)	3074	761
Aged	1763	433

The shear strength in the X direction from the shear block test and the three point bending compare well (738 and 761 respectively). An aging factor of 0.60 was derived based on the results of the aged and the un-aged samples as follows:

$$c_e = \text{Aging factor} = \frac{433 \text{ psi}}{761 \text{ psi}} = 0.58 \Rightarrow \text{use } 0.6 \qquad (2.4)$$

It should be noted that Trus Joist, Inc did not provide any factors to address the aging issue.

2.5.5 Full-Scale Bending Tests

Four beams with dimensions (7”× 7” × 108”) were tested under four point bending to establish the modulus of elasticity and the modulus of rupture.

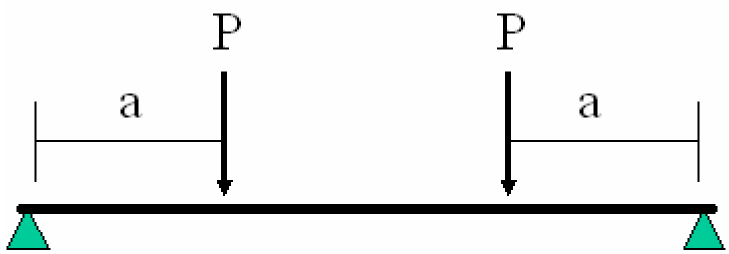


Figure 2.3: Four point bending setup (smith, 2003).

The first three beams were tested in a configuration to promote a bending failure, while the fourth beam was tested in a configuration to promote shear failure. Beams 1 and 3 exhibited bending failure mode, while beams 2 and 4 exhibited horizontal shear mode. Table 2.6 shows the failure load, the maximum bending stress and the maximum shear stress for the beams.

Table 2.6: Max bending and shear stress (Smith, 2003)

Beam	a (in.)	P (kips)	Bending Stress(psi)	Shear Stress(psi)
B1	24	17	7136.6	520.4
B2	24	21.25	8920.8	650.5
B3	24	18.75	7871.3	574.0
B4	18	19.2	6045.1	587.8

2.6 Deck Design Procedure

Smith (2003) presented preliminary design calculations for the Parallam® deck of the Woodville Beam Span Bridge located in District 2 of WVDOH. The bridge was designed jointly by West Virginia Division of Highways (WVDOH) and the Constructed Facilities Center of West Virginia University.

The deck design follows procedures developed by AASHTO Standard Specifications for Highway Bridges, 16th Edition, 1996 which is developed for non-connected glulam panels. The allowable stress values for Parallam® were obtained from AASHTO Table 13.5.4B. Modification factors for wet service were also obtained from AASHTO Table 13.5.4B and aging strength reduction factor of Ce= 0.61 was used.

To arrive at a conservative design method, the following assumptions were made:

1. There is no composite action between the deck panels and the steel stringers.
2. The deck panel is assumed to behave as a simple span beam between the steel stringers.
3. The deck panel is designed to resist the stresses induced by the traffic loads in the direction perpendicular to the traffic.
4. There is no panel to panel connection, i.e., does not account for the plate action induced by connecting the deck panels together.
5. The stresses induced in the panel in the traffic direction are insignificant and therefore not checked.

The following is the design procedure for Peel Tree Bridge:

1. Determine the bridge configuration and loading:

Span: 29 ft

Stringer spacing 33 inches

Roadway Width (out-to-out): 15 ft

Parallam® Panel Width: 19"

Parallam® Panel Length: 15 ft (equal to roadway width)

AASHTO Loading: P= 15,000 lb HS-25 wheel load (1.25* 12,000)

Determine the material properties form AASHTO Table 13.5.4B

$F'_{by} = F_{by} .C_m = 2900$ psi *0.8 = 2320 psi

$E' = E .C_m = 2.0*106 *0.85 =1.7 *106$ psi

$F'_{vy} = F_{vy}. Ce = 210$psi * 0.71 =149.1 psi

γ=50 lb/ ft3 (Creosote Treated)

Where, Fby = Tabulated allowable bending stress, psi

F_{vy} = Tabulated allowable horizontal shear stress, psi

E = Modulus of elasticity, psi

C_m = Adjustment factor for moisture

C_e = Adjustment factor for durability which includes moisture effects

W 14 X 90 Steel stringers are used. b_f= 14.520”

2. Wheel load distribution area (length and width)

P= wheel load for HS-25 truck = 15,000

b_t = wheel load distribution width in the direction of deck span

$b_t = \sqrt{0.0025 * P}$ = 19.36 in. (AASHTO 3.30)

b_d =wheel load distribution width normal to the direction of deck span

b_d= 15+ (2X deck thickness) ≤ Panel width (19 in.) (AASHTO 3.25.11)

b_d= = 15+ (2X7) = 29 in. ≤ 19 in.

b_d= 19 in.

3. Determine the panel section properties:

$A = b_d \times t = 19 \times 3.5 = 66.5 \text{ in}^2.$

$S = b_d \times t^2 / 6 = 19 \times 3.5^2 / 6 = 38.79 \text{ in}^3.$

$I = b_d \times t^3 / 12 = 19 \times 3.5^3 / 12 = 67.89 \text{ in}^4.$

4. Determine the effective deck span as per AASHTO 3.25.1.2:

s = Effective deck span taken as the clear distance (CD) between stringers plus one half the width (or flange width) of one stringer but not to exceed the clear span plus the assumed panel thickness.

s = CD + (0.5*stringer flange width) ≤ CD +panel thickness

s = (18.48) (0.5*14.52) =25.75 ≤ 18.48+ 3.5 = 22 in OK.

s =22 in.

5. Compute the dead load from the weight of the panel and the wearing surface:

$W_{DL} = W_{panel} + W_{asphalt}$

$W_{panel} = 50pcf \times 19$ in. $\times 3.5$ in / 1728 = 1.94 lb/in.

$W_{asphalt} = 150$ pcf $\times 2.0$ in. $\times 19$ in. /1728 = 3.3 lb/in.

$W_{DL} = 1.94 + 3.3 = 5.24$ lb/in.

6. Check bending stresses:

Dead Load Moment = $M_{DL} = W_{DL} * s^2 / 8 = 5.24 * 22^2 / 8 = 317$ in.lb

Live Load Moment = $M_{LL} = P * s / 4 = 15{,}000 * 22 / 4 = 82{,}500$ in.lb

Total Moment = $M_T = M_{DL} + M_{LL} = 82{,}817$ in.lb

$S_{Req.} = 0.8 * M_T / F_{by} = 82{,}817 / 2320 = 35.70 \leq 38.79$ OK. (0.8 factor for continuity)

7. Check for horizontal shear stress:

Horizontal shear in transverse Parallam® deck panels shall be based on the maximum vertical shear occurring at a distance from the supporting beams equal to the deck thickness (AASHTO 3.25.1.3).

Dead Load Shear = $V_{DL} = W_{DL} * (s/2 – t) = 5.24 * (22 /2 – 3.5) = 39.3$ lb.

Live load shear is computed by placing the edge of the wheel load distribution width (b_t) at a distance equal to the deck thickness. The resultant of the wheel load acts through the center of the distribution width.

Live Load Shear = $V_{LL} = \dfrac{s - t - b_t/2}{s} * P = \dfrac{22 - 3.5 - 19.36/2}{22} * 15{,}000 = 6014$ lb

Total Shear = V_T = 6,053 lb

$F_{VYREQ} = \dfrac{3 * V_T}{2 * A} = \dfrac{3 * 6053}{2 * 66.5} = 136.5$ psi < $F'_{VY} = 149.1$ psi OK.

8. Check for live load deflection:

Δ_{max}= AASHTO HS-25 Loading*s^3/ (48*E'*Iy)

Δ_{max}= 15,000* 22^3/ (48*1,700,000*67.89) = 0.0288" < s/500 = 0.044 ok

2.7 Bridge Construction Procedure

The construction procedure involves the following steps:

1. Acquire appropriate borings for piles. (Geotechnical Subcontractor)

2. Drive piles into bedrock and assemble abutment rebar.

3. Pour abutments, leaving some rebar exposed in order to pour backwall.

4. Place steel girders (fascia girders welded with posts for railing system).

5. Place Parallam deck pieces on steel girders, attaching each to the other by joint stiffener.

6. Attach deck pieces to steel girders using WVU Spring Connectors.

7. Pour backwall.

8. Place waterproofing membrane onto Parallam deck.

9. Pour asphalt overlay for decking.

10. Assemble guide rail system with blockouts and rails.

11. Finish approach work in order to ensure alignment of roadway and bridge way.

CHAPTER 3

Live Load Testing of Peel Tree and Hackers Creek Bridges

3.1 Introduction

The dynamic response of moving traffic on a bridge deck is as important as the static load effects. The moving load that crosses a bridge leads to higher induced stresses and deflections. The increase in stresses and deflections under moving loads with respect to the static stresses and deflections is referred to as the Dynamic Load Allowance (DLA) or Load Magnification Factor. Also, it is a very important design parameter to evaluate or quantify the Transverse Load Distribution Factors for a given bridge system. In this chapter, data are reported with reference to Peel Tree and Hackers Creek bridges. The test data have been used to establish the Dynamic Load Allowance (DLA) and the Transverse Load Distribution Factors (TLDF) under service conditions through both the live load deflection data as well as strain data. The West Virginia Department of Transportation-Division of Highways provided two axle dump trucks for field testing. Strain and deflection responses of Peel Tree and Hackers Creek bridges were measured under static conditions as well as variable speeds of fully loaded dump trucks.

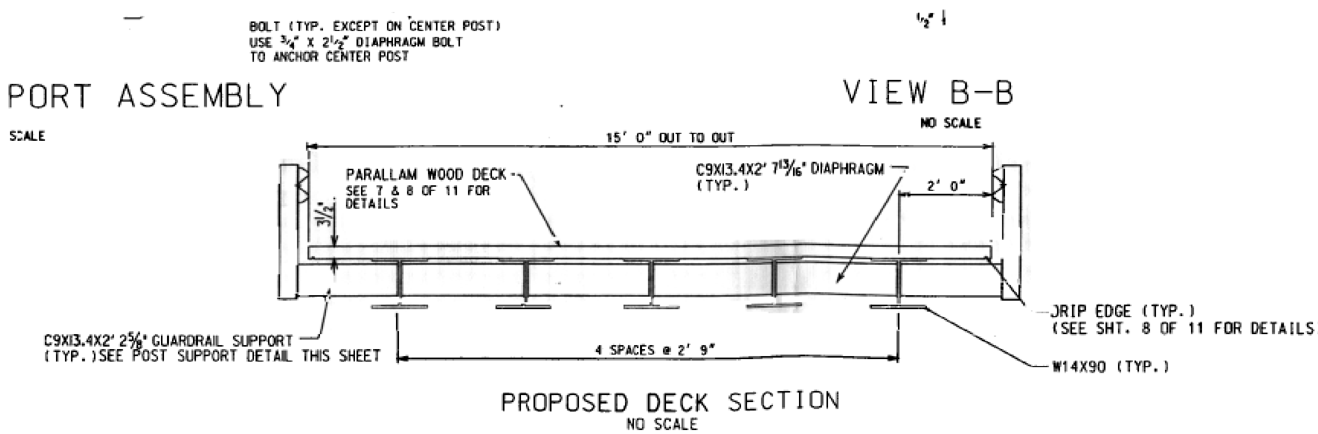
3.2 Bridge Description and Instrumentation

The two bridges (Peel Tree & Hackers Creek) were built using Parallel Strand Lumber (PSL) or Parallam® deck stiffened by steel stringers. The design was provided by CFC-WVU. The two bridges were instrumented with uni-axial strain gages and LVDTs which were used to measure strains and deflections respectively. The detailed description of each bridge and the instrumentation are discussed in the following sections.

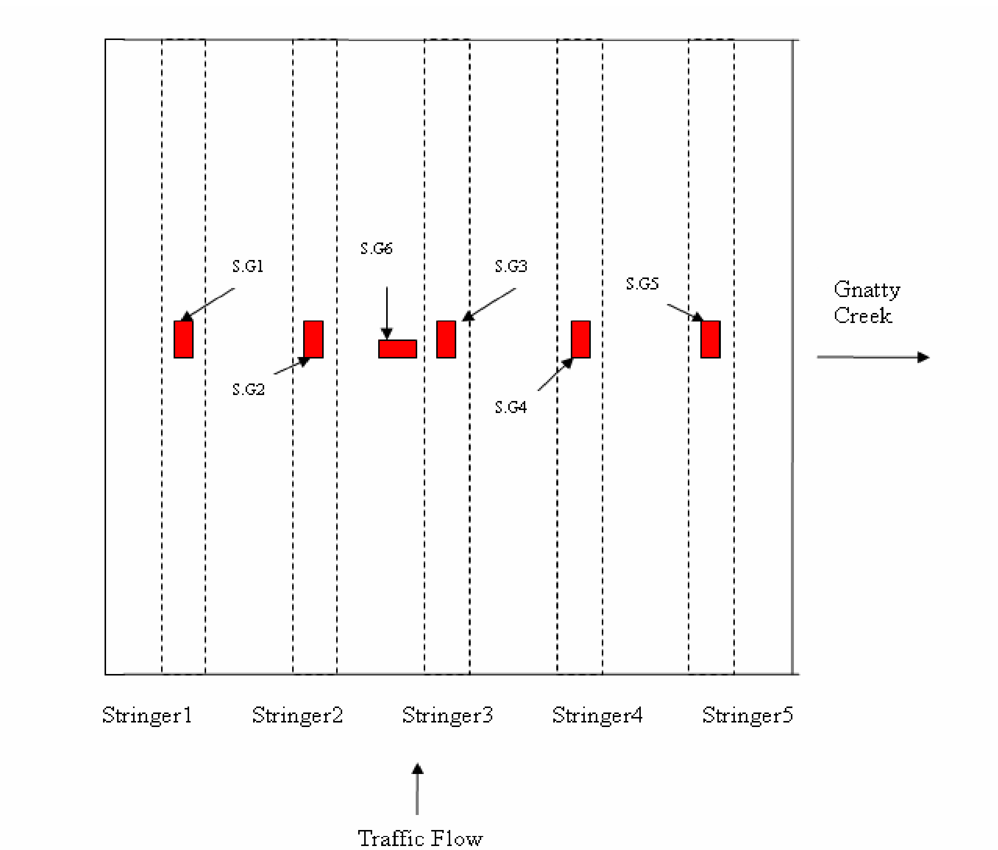
3.2.1 Peel Tree Bridge

Peel Tree Bridge is a 29 feet long span bridge located along road 20/3 in Barbour County, WV with a single traffic lane. The bridge consists of five W14×90 stringers spaced at 2'-9" centers with a 2' deck overhang. The total width of the bridge is 15'. The deck system utilized 19"×180" Parallam® panels with a deck thickness of 3.50". Plate clips are used to connect the panels and the stringers (see figure 3.11). No surface overlay was used on top of the Parallam® panels. The bridge was designed to carry a HS-25 AASHTO loading. The bridge was instrumented with five strain gages (gages 1,2,3,4 and 5) in the span direction. The gages were located at the center of the bottom flange of each steel stringer. Also, an additional strain gage (6) was installed on the bottom of the deck adjacent to the top flange of the central stringer to measure strains in the transverse direction. During the first two tests, scales were attached to stringers 1 and 3 to measure the static deflection by taking two measurements, one prior to loading and one after static loading. During the November 2006 test, two LVDTs were used to measure the static and dynamic deflections of stringers 1 and 3. For the field test in June 2007, nine LVDTs were used to measure the static and dynamic deflections of both the deck and stringers. Figure 3.1 shows a typical cross section of the bridge and strain gage locations.

Figure 3.1 (a) and (b): Peel Tree Bridge Cross Section and Strain Gage Locations



(a) Peel Tree Bridge Cross Section

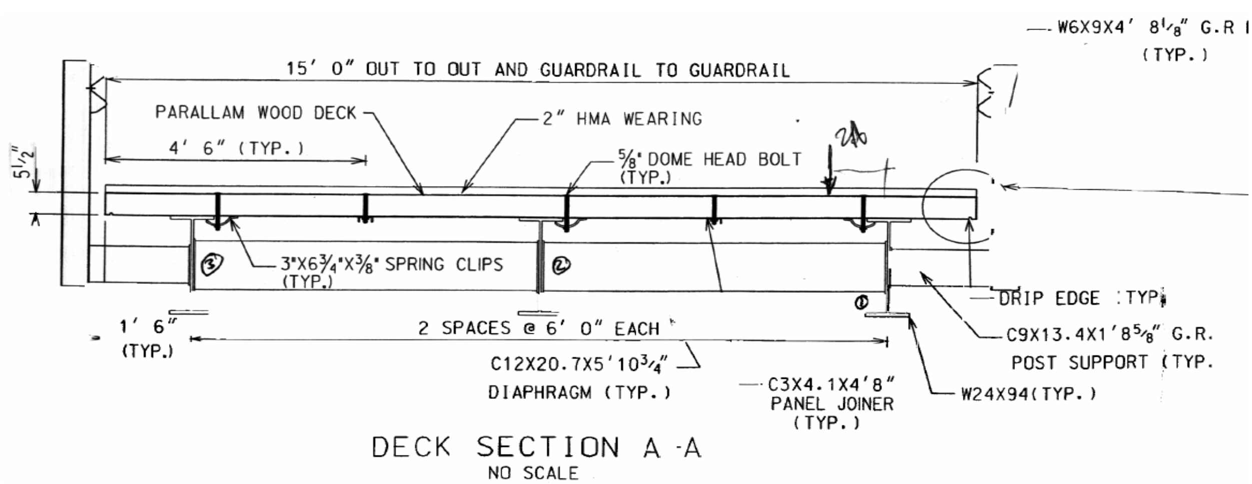


(b) Peel Tree Bridge Strain Gage Layout

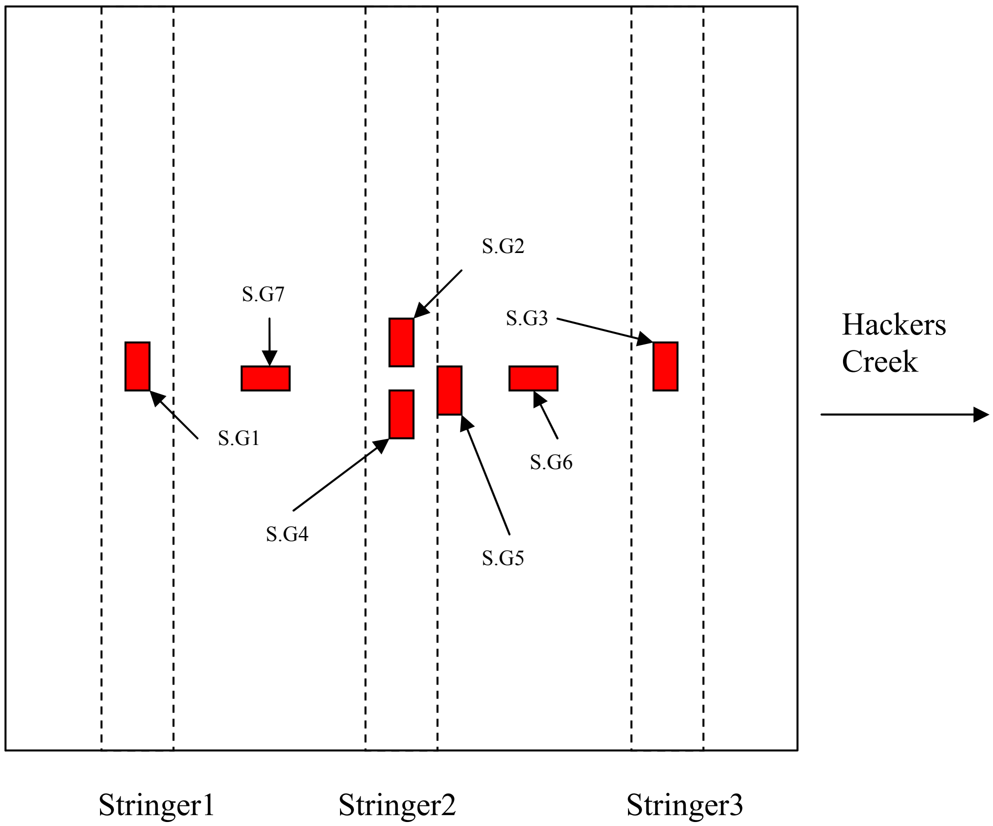
3.2.2 Hackers Creek Bridge

Hackers Creek Bridge has a 31 feet span with simple supports. The bridge consists of three W24×94 steel stringers spaced at 6 feet centers with 1'-6" deck overhang, resulting in a width of 15 feet. The deck system is similar to the one described for Peel Tree Bridge except that the deck thickness is 5.50" with 1" asphalt wearing surface. The bridge serves a single traffic lane on road 119/2 in Barbour County, WV. The bridge was instrumented with a total of 7 strain gages at the midspan, one at the bottom flange of each stringer in the span direction (gages 1, 2and3), one at the bottom of the top flange of the central stringer (gage 4), one on the bottom of deck adjacent to the top flange of the central stringer in the span direction (gage 5) and two gages on the bottom of the deck in the transverse direction of the bridge (gages 6 and 7). LVDTs were used to measure both the static and the dynamic deflections at the midspan for both the deck and the stringers. Figure 3.2 shows a typical cross section of the bridge and the strain gage locations.

Figure 3.2 (a) and (b): Hackers Creek Bridge Layout and Strain Gage Locations



(a) Hackers Creek Bridge Cross Section



(b) Hackers Creek Bridge Strain Gage Locations

3.3 Test Procedure

Peel Tree Bridge was tested on four separate occasions: September 2005, September 2006, November 2006 and June 2007. A dump truck provided by WVDOH passed over the bridge at different speeds: 2 mph, 10 mph and 15 mph with the exception of tests conducted in November 2006 and June 2007. During testing in November 2006 and June 2007, the truck passed over the deck at 20 mph. An additional measurement was taken at 10 mph with a 2”×4” wooden plank located just before the midspan section to represent an impact load. Also, a set of data was acquired with the center of gravity of the rear axle of the truck located at the midspan. The strain and deflection data have been acquired using a Vishay Micro-Measurements’ System 5100 scanner. The scanner is controlled from a laptop computer. Because of the approach conditions, the test procedure was slightly different for Hackers Creek Bridge from Peel Tree Bridge testing. The response of Hackers Creek Bridge was measured with the center of gravity of the rear axle at the midspan, at 2 mph, 10 mph and 10 mph with impact. The response could not be taken at higher truck speeds because of a sharp curve at the south end of the bridge.

Figure 3.3: Static Live Load Test of Peel Tree Bridge

Figure 3.4: Static Live Load Test of Hackers Creek Bridge

3.4 Results and Analysis for Peel Tree Bridge

The response of Peel Tree Bridge was evaluated using the strain and deflection data. To calculate the in-service bridge response, maximum strains and deflections at each measurement location were determined. Figure3.5 shows a typical time-strain response for one of the stringers of Peel Tree Bridge.

The maximum stringer strain recorded for Peel Tree Bridge was 302 microstrains, which is well within the yield strain of steel (2000 microstrains) and it corresponds to a stress level of 8758 psi. Also, the maximum recorded deck strain was 195 microstrains which is also well below the allowable strain (1450 microstrains) and it corresponds to a stress level of 390 psi. The maximum strain acquired during each test is reported in tables 3.1, 3.2, 3.3 and 3.4.

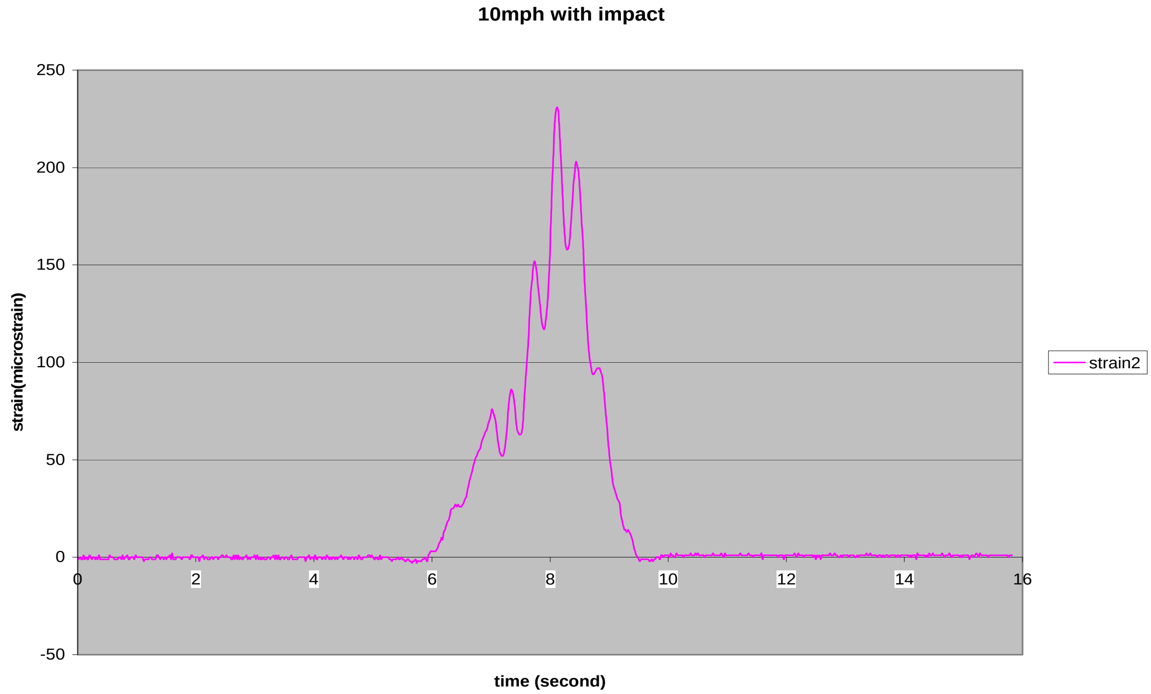


Figure 3.5: Strain-Time Response of Stringer 2 in Peel Tree Bridge

Load Case	Stringer1	Stringer2	Stringer3	Stringer4	Stringer5	Deck
Static	153	207	218	205	173	173
2mph	173	215	206	186	152	153
10mph	155	204	205	189	148	162
15mph	137	189	200	194	178	178
10mph+Impact	250	302	291	248	193	193

Table 3.1: Maximum Measured Strains (microstrains) for Peel Tree Bridge-September 2005

Load Case	Stringer1	Stringer2	Stringer3	Stringer4	Stringer5	Deck
Static	174	206	N.A	167	N.A	29.6
2mph	161	207	N.A	190	N.A	-138
10mph	144	210	N.A	220	N.A	-132
15mph	135	205	N.A	207	N.A	-129
10mph+Impact	131	230	N.A	232	N.A	-116

Table 3.2: Maximum Measured Strains (microstrains) for Peel Tree Bridge-September 2006

Load Case	Stringer1	Stringer2	Stringer3	Stringer4	Stringer5	Deck
Static	162	184	142	157	112	184
2mph	152	183	185	157	121	183
10mph	157	193	202	176	145	193
20mph	138	185	213	189	185	185
10mph+Impact	150	195	199	175	146	195

Table 3.3: Maximum Measured Strains (microstrains) for Peel Tree Bridge-November 2006

Load Case	Stringer1	Stringer2	Stringer3	Stringer4	Stringer5	Deck
Static	124	N.A	171	166	142	-19
2mph	145	N.A	212	217	190	-133
10mph	143	N.A	200	194	168	-117
20mph	191	N.A	247	240	202	-126
10mph+Impact	107	N.A	212	238	250	-121

Table 3.4: Maximum Measured Strains (microstrains) for Peel Tree Bridge -June 2007

3.4.1 Dynamic Load Allowance for Peel Tree Bridge

The Dynamic load allowance was computed from strain data using field information from all four tests performed on Peel Tree Bridge. Also, the Dynamic Load Allowance from deflection was computed from November 2006 test data. The maximum recorded strain was used to compute the DLA. Also, the response at 2 mph was used as a baseline to compute the DLA in the case of Peel Tree Bridge.

Based on strain measurements, the highest DLA value for Peel Tree Bridge was found to be 53% which occurred during the November 2006 test at 20 mph. This value exceeds the 33% suggested by AASHTO. The strain based DLA values for the deck and the stringers were as low as 1% and as high as 53%. The average DLA was 16% with a standard deviation of 13.13 for the full data range. Also, the average DLA was 12% and the standard deviation was 8.4 for the data lower than 33% (as per AASHTO LRFD). As for the data higher than 33%, the average DLA was 42% and the standard deviation was 5.7. Also, it was observed that the DLA for the same measurement location at the same speed from other tests at different dates was different. For example, the DLA for stringer 4 at 10 mph was 33%, 22% and 11% from the first, second and third tests respectively. This observation supports the conclusion made by Bakht and Pinjarkar (1989) that a probabilistic approach should be adopted to evaluate the DLA. Moreover, there is no clear trend in the relationship between the DLA and the test vehicle speed. It should be noted that a negative DLA value was observed in some cases which could be attributed to transverse position of the test truck (off-center) when it enters the bridge.

From the maximum deflection data, the DLA value was only 9% which occurred during the November 2006 test at 10mph with impact.

3.4.2 Transverse Load Distribution Factors (TLDF) for Peel Tree Bridge

The Transverse Load Distribution Factors (TLDF) are computed from the field data to determine the fraction of the wheel load carried by a beam or a girder. The TLDF is computed based on the stringer midspan strain data under the static load case (i.e. the center of gravity of the rear axle is at the midspan). The TLDF of a given stringer is computed as the static strain of that stringer divided by the sum of the static strains of all stringers. Equation (3.1) is used to calculate the load fraction of the stringers:

$$\text{Load Fraction} = \frac{\varepsilon_i}{\sum_{j=1}^{k} \varepsilon_j} \tag{3.1}$$

Where e_i is the measured static strain for stringer i and Σe_j is the summation of measured static strains in all stringers. The load fraction is based on the number of loaded lanes which is one for Peel Tree Bridge. To compare this value to the TLDF suggested by AASHTO for timber bridge decks which is S/5 (S is the stringer spacing), the stringer spacing is divided by the number resulting from the above formula. For Peel Tree Bridge the maximum TLDF was S/11.30 which is very conservative compared to the design TLDF of S/5. Tables 3.5 and 3.6 summarize the TLDF for Peel Tree Bridge from September 2005 and November 2006 respectively.

Table 3.5: Stringers TLDF values for Peel Tree Bridge-September 2005

	TLDF as Ratio of S
Stringer 1	S/17.2
Stringer 2	S/12.7
Stringer 3	S/11.4
Stringer 4	S/12.9
Stringer 5	S/15.2

Table 3.6: Stringers TLDF values for Peel Tree Bridge-November 2006

	TLDF as Ratio of S
Stringer 1	S/12.9
Stringer 2	S/11.3
Stringer 3	S/14.6
Stringer 4	S/13.3
Stringer 5	S/18.6

3.4.3 Live Load Deflection for Peel Tree Bridge

The static live load deflections were measured for stringers 1 and 3 in the first three tests, and are given in table 3.7. It is clear that the data from the first and the last test compare well, which gives enough reason to believe that the data from the second test have some sort of human error while taking the measurements. Deflections of both the deck and stringers were measured on June 12, 2007. The deck deflection was computed as the difference between the total deck deflection and the average of the deflections of the stringers supporting the bay under consideration. Table 3.8 shows the static live load deflections from the test data taken in June 2007.

Comparing these values to the live load deflection limits of L/800= 0.435" for the stringer and S_{eff}/500= 0.044" for the deck panels set by CFC-WVU engineers during the design process, It is clear that actual live load deflections are within the limit and is valid for cases without applying the DLA factor to deflection value.

Table 3.7: Measured Static Live Load Deflection for Stringers 1 and 3 from the first three tests

	Deflection of Stringer 1	Deflection of Stringer 3
September 2005	0.3125"	0.2187"
September 2006	0.59"	0.40"
November 2006	0.3225"	0.243"

Table 3.8: Measured Static Live Load Deflections for Peel Tree Bridge from June 18, 2007 test

Component	Stringer 1	Stringer 2	Stinger 3	Stringer 4	Stringer 5	Deck Bay 1	Deck Bay 2	Deck Bay 3	Deck Bay 4
Static deflection	0.175"	0.264"	0.270"	0.277"	0.246"	0.0195"	0.019"	0.0025"	0.032"

3.4.4 Visual Inspection and Bridge Condition

Peel Tree Bridge was visually inspected on 6/1/2007. The purpose of the inspection was to report any serviceability problems such as rutting of the deck under wheel path, joint integrity, and deck moisture content.

The visual inspection revealed wear and rutting of the deck in the wheel path (Figure 3.6). Also, chipping and rolling of wood fibers of the Parallam deck were noticed at the joint between the deck and the approach (Figures 3.7 and 3.8). Moreover, gravel from the roadway, driven onto the deck, has caused indentations (Figure 3.9).

Neither the panel-to-panel connectors (i.e. the inverted channels) nor the panel-to-stringer connectors (i.e. the plate clips) showed any sign of distress. In addition, the bottom of the deck, the stringers, the guardrail and the seats are in very good condition (Figures 3.10, 3.11 and 3.12). A couple of bird nests were found on the diaphragms (Figure 3. 13).

Moisture Content

Moisture content readings were taken on two occasions; September 2005 and June 2007. Ten measurements were taken on the top of the deck in September 2005 and the average moisture content reading was 10.58%. As of June 2007, moisture content measurements were taken at both the top and bottom of the deck. Twenty readings were taken at the top of the deck and the average moisture content was 8.17%. Also, ten readings were taken at the bottom of the deck and the average moisture content was 13.67%. AASHTO LRFD bridge design specifications as well as AASHTO standard bridge design specifications have a limit of 19% on the moisture content for wood structures. Therefore, the average moisture content of the Parallam deck is within the limit.

Creep

In an attempt to measure deck creep, a micrometer has been used to measure the elevations at four locations in the bottom of the deck between the top flanges of the stringers (see Figure 3.14). Four discrete measurement points were taken: at 4” and at 29” from the edge of the top flange and to the left and to the right of the inverted channel. The maximum difference in elevation was about 1/8” which suggests that the creep is virtually zero (Figure 3.15). There is sudden change in the slope of the lines to the left and right of the channel. A possible explanation is that one side of the inverted channel dug into the deck while tightening the bolts. It should be noted that the points on the positive side of the Y axis of Figure 3.15 indicate camber (upward) while the points on the negative side indicate a downward deflection.

Figure 3.6: Wheel Path Rutting of the Deck on Peel Tree Bridge

Figure 3.7: Parallam Deck Chipping on Peel Tree Bridge

Figure 3.8: Parallam Deck Rolling on Peel Tree Bridge

Figure 3.9: Indentation in the Deck Due to Gravel Driven in by the Traffic

Figure 3.10: Guardrail Condition on Peel Tree Bridge

Figure 3.11: Conditions at the Bottom of Peel Tree Bridge

Figure 3.12: Conditions at the Bottom of Peel Tree Bridge

Figure 3.13: Bird Nest on the Diaphragm

Figure 3.14: Using the Micrometer to Take Creep Measurements

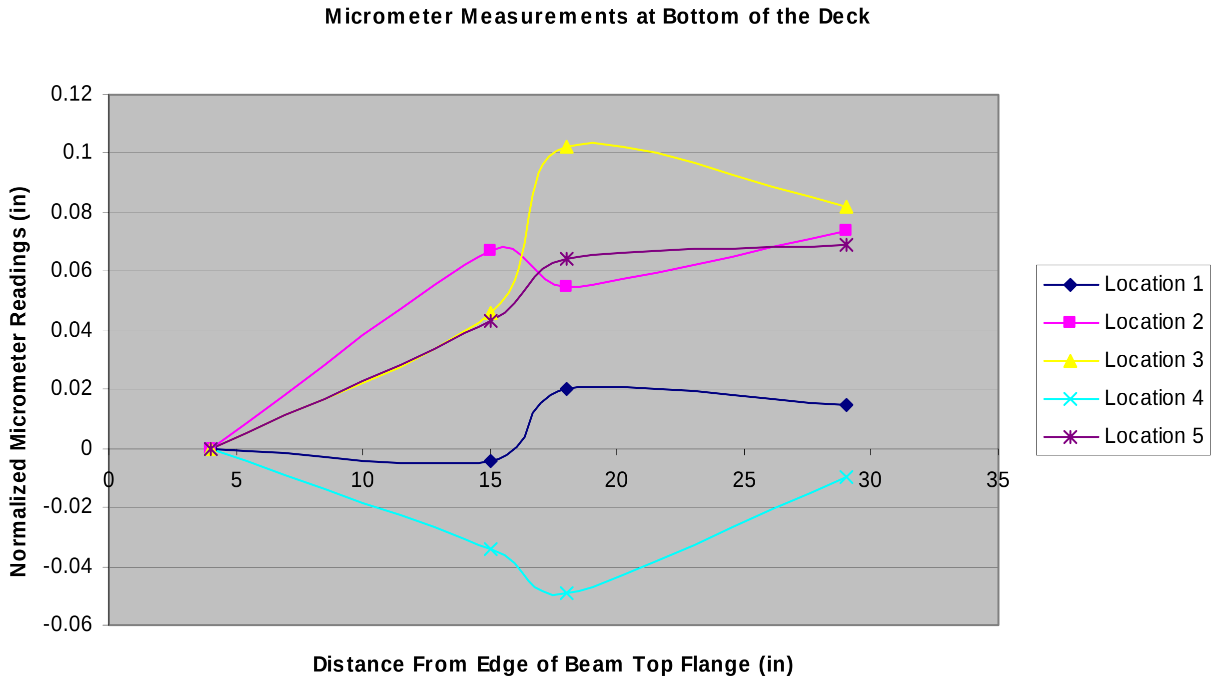


Figure 3.15: Normalized Micrometer Readings at the Bottom of the Deck

3.5 Results and Analysis for Hackers Creek Bridge

The response of Hackers Creek Bridge was evaluated using the strain and deflection data. To evaluate the in-service bridge response, maximum strains and deflections at each measurement location were determined.

For Hackers Creek Bridge, the maximum recorded stringer strain was 413 microstrains, which is below the yield strain of steel (2000 microstrains) and it occurred on September 2006. The maximum recorded deck strain was 651 microstrains on May 2005 which is about 56% of the allowable strain. The strain value of 651 microstrains corresponds to about 1300 psi of bending stress which is also well within the allowable limit of 2900 psi. It should be noted that Trus Joist, Inc. suggested a wet service factor of 0.80 to be applied to the allowable bending stress of 2900 psi. Also, it has to be emphasized that the weight of the rear axle of test truck is at least 15% higher than that of HS-25 truck.

The data acquired during each of the three tests are reported in this section. Tables 3.9, 3.10, 3.11 and 3.12 provide a summary of the maximum recorded strains for the stringers and the deck components of Hackers Creek Bridge.

Load Case	Gage1	Gage2	Gage3	Gage4	Gage5	Gage6	Gage7
Static	182	245	154	-215	N.A	538	532
2mph	157	187	187	-228	N.A	574	552
10mph	169	240	152	-199	N.A	432	509
10mph +Impact	157	256	187	-228	N.A	574	651

Table 3.9: Maximum Measured Strains (microstrains) for Hackers Creek Bridge-May 2005

Load Case	Gage 1	Gage2	Gage3	Gage4	Gage5	Gage6	Gage7
Static	175	198	N.A	N.A	N.A	189	236
2mph	173	206	N.A	N.A	N.A	391	476
10mph	151	413	N.A	N.A	N.A	339	417
10mph +Impact	154	176	N.A	N.A	N.A	363	491

Table 3.10: Maximum Measured Strains (microstrains) for Hackers Creek Bridge-September 2006

Load Case	Gage1	Gage2	Gage3	Gage4	Gage5	Gage6	Gage7
Static	122	188	165	-178	-166	207	195
2mph	116	188	143	-178	-145	383	329
10mph	117	181	N.A	-128	-161	393	302
10mph +Impact	147	224	N.A	-204	-186	378	294

Table 3.11: Maximum Measured Strains (microstrains) for Hackers Creek Bridge-November 2006

Load Case	Gage1	Gage2	Gage3	Gage4	Gage5	Gage6	Gage7
Static	19	210	132	188	N.A	191	210
2mph	116	214	146	423	N.A	524	214
10mph	140	210	144	385	N.A	359	210
10mph +Impact	196	236	157	386	N.A	445	236

Table 3.12: Maximum Measured Strains (microstrains) for Hackers Creek Bridge-June 2007

3.5.1 Dynamic Load Allowance for Hackers Creek Bridge

The Dynamic Load Allowance (DLA) is defined as the increase in induced static strain or deflection due to vehicle movement across a bridge. The Dynamic Load Allowance was computed from strain data using field information from all tests performed on Hackers Creek Bridge. The Dynamic Load Allowance from deflection was computed from the last two sets of test data, i.e. from November 2006 and June 2007 test data. The maximum recorded strain was used to compute the DLA. However, the static response was taken as base line to compute the DLA for Hackers Creek Bridge because there was a significant dynamic deck response at 2 mph.

The highest DLA value was 174% which occurred at 2 mph on June 2007 test. This was taken at the bottom of the deck. Also, the analysis results showed that the DLA values ranged from as low as 1% to as high as 174% which shows the importance of following a statistical approach for evaluating the DLA. The average DLA was 52% and the standard deviation was 47.54 for the full spectrum of data. Also, the average DLA was 11% and the standard deviation was 7 for the data lower than 33% (as per AASHTO LRFD). As for the data higher than 33%, the average DLA was 97% and the standard deviation was 28.4. Our data did not reveal a clear trend between the DLA and the vehicle speed.

The maximum DLA from deflection data was 34% and it occurred at 10 mph with impact (truck traveling over 2”x4” plank) on June 2007. In general, the DLA from deflection is significantly lower than the DLA from strain. It should be noted that no representative design DLA value is suggested herein due to very limited deflection data.

3.5.2 Transverse Load Distribution Factors for Hackers Creek Bridge

The Transverse Load Distribution Factors (TLDF) for Hackers Creek Bridge are evaluated using the stringers static strain and deflection responses. First, the load fraction for each stringer is computed using equation 3.1. Then, the stringer spacing S is divided by the resulting number to find the TLDF value. It is quite clear that the actual TLDF values are far less than the design TLDF of S/5. Table 3.13 shows the TLDF values for Hackers Creek Bridge measured in November 2006 and June 2007 tests.

Table 3.13: Stringers TLDF values for Hackers Creek Bridge from November 2006 Test

	TLDF as Ratio of S (November 2006)	TLDF as Ratio of S (June 2007)
Stringer 1	S/23.35	S/23.7
Stringer 2	S/15.15	S/13.3
Stringer 3	S/17.30	S/20.4

3.5.3 Live Load Deflection for Hackers Creek Bridge

During the November 2006 test, LVDTs were used to measure deflections of stringers 2and 3. The measured deflections under static load were 0.235” and 0.176” for stringers 2 and 3 respectively. Both the deck and stringer deflections were measured during the June 2007 test. The relative deck deflections were computed using the same procedure as Peel Tree Bridge. Table 3.14 summarizes the static live load deflection data.

Comparing these values to the live load deflection limits of L/800= 0.465" for the stringer and S_{eff}/500= 0.135" for the deck panels set by CFC-WVU engineers during the design process, It is clear that actual live load deflections are well within the limits. It should be noted that the weight on the rear axle of the test truck is at least 15% higher than that of HS-25 truck.

	Stringer 1	Stringer 2	Stringer 3	Deck Bay 1	Deck Bay 2
Nov. 2006	N.A	0.235"	0.176"	N.A	N.A
June, 2007	0.135"	0.241"	0.157"	0.0445"	0.0315"

Table 3.14: Static Live Load Deflections for Hackers Creek Bridge

3.5.4 Visual Inspection and Bridge Condition

A visual inspection was performed on Hackers Creek Bridge on 6/1/2007 revealed: 1) the bottom side of the deck was in a very good condition; 2) no leaching of preservative was noticed; and 3) vertical movement of the Parallam panels relative to one another was noticed under traffic load.

The inspection also revealed the stringer-to-panel connector failure. These connectors exhibited a brittle mode of failure at the bolt location as seen in Figures 3.16 and 3.17. This failure could be due to one or a combination of the following factors: 1) the connectors were over stressed due to excessive torque on the bolts, 2) hydrogen embrittlement during galvanization process, 3) manufacturing faults, and 4) excess traffic induced fatigue stresses. At least nine connectors were broken or missing.

The stringers, Diaphragms and the guardrail of the bridge are in excellent condition and showed no sign of potential problems. The wearing surface did not show any reflective cracking, however, small dips in the wearing surface are visible (Figure 3.18). These dips coincide with the joints between the Parallam panels. Also, disintegration of abutment concrete was noticed on the north side (Figure 3.19).

Moisture Content

Moisture content measurements were taken at both the top and bottom surfaces of the deck. Ten measurements were taken on the top of the deck and the average moisture content was 10.88%. Also, ten readings were taken at the bottom of the deck and the average moisture content was 12.94% (see Figures 3.20 & 3.21). The average moisture content is below the maximum specified by AASHTO bridge design specifications (LRFD and standard) which is 19%.

Creep

A micrometer was used to detect any creep in the Parallam deck between stringers. The procedure to measure creep is similar to the description given earlier for Peel Tree Bridge. Again, the differences in the micrometer measurements were less than 1/8”, which indicates that deck creep deformations are close to zero. There is sudden change in the slope of the lines to the left and right of the channel. A possible explanation is that one side of the inverted channel dug into the deck while tightening the bolts. It should be noted that the points on the positive side of the Y axis of Figure 3.22 camber, while the points on the negative side indicate deflection.


Figure 3.16: A Failed Stringer to Deck Connector on Hackers Creek Bridge


Figure 3.17: Failure Mode of a Stringer to Deck Connector on Hackers Creek Bridge

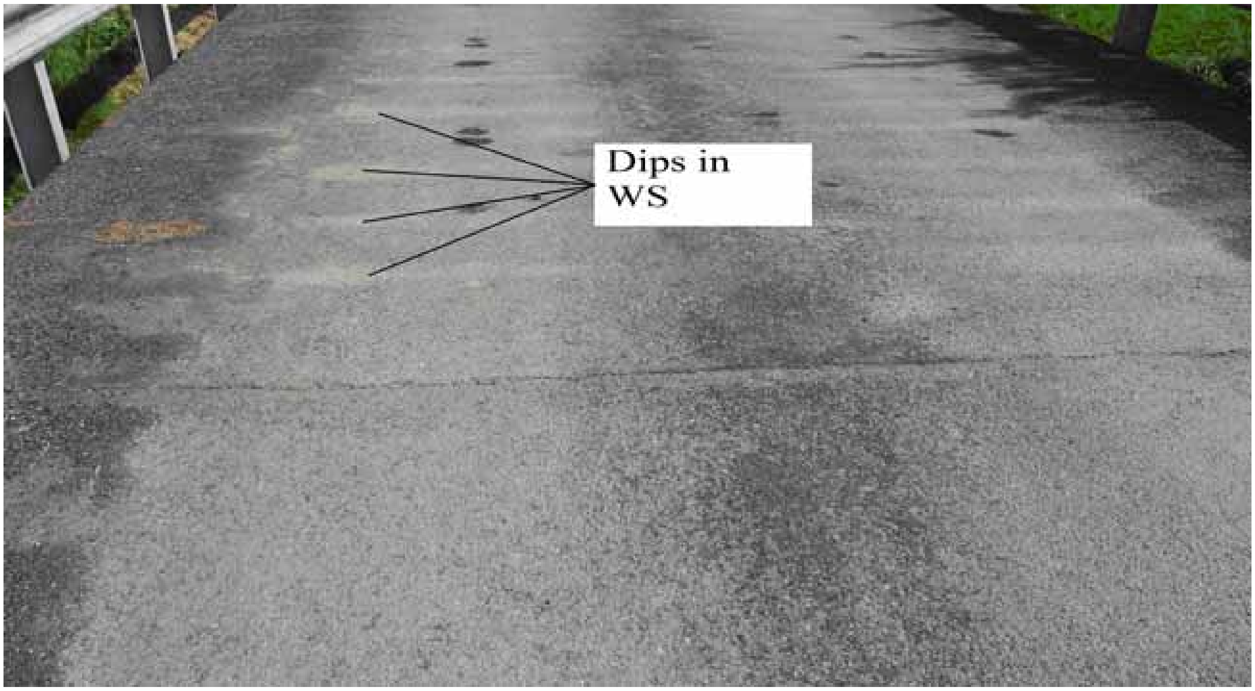


Figure 3.18: Dips in the Wearing Surface of Hackers Creek Bridge


Figure 3.19: Disintegration of Concrete on the north abutment

Figure 3.20: Taking Deck Moisture Content Readings on Hackers Creek Bridge

Figure 3.21: Taking Deck Moisture Content Readings on Hackers Creek Bridge

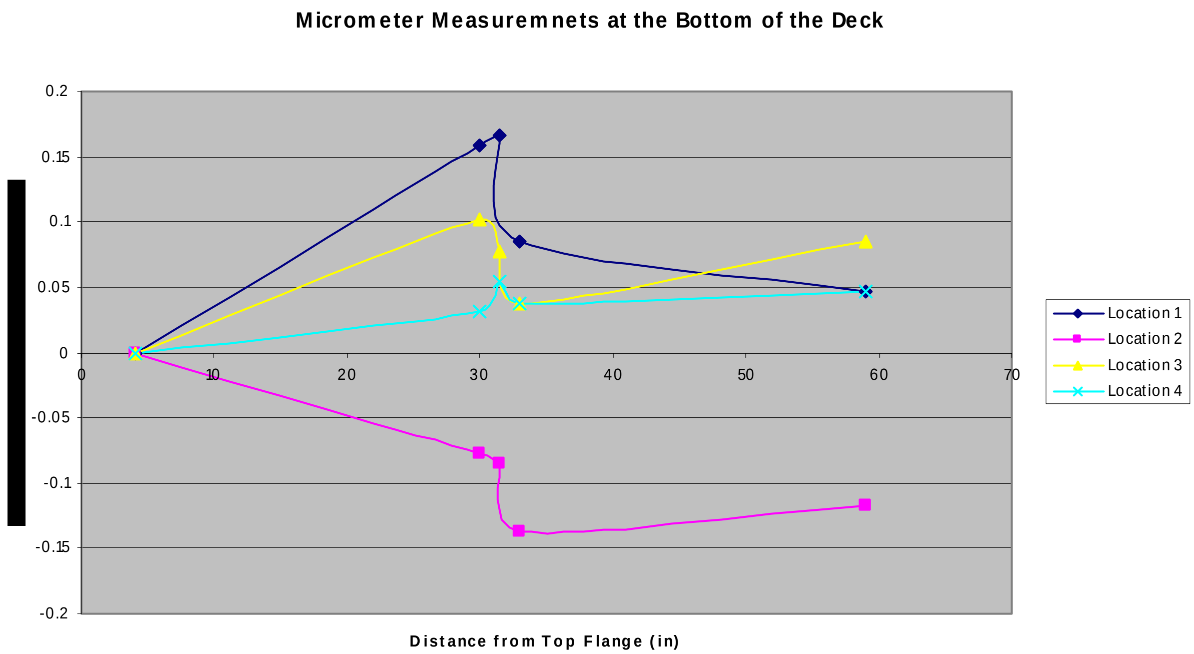


Figure 3.22: Normalized Micrometer Measurements at the Bottom of the Deck

Chapter 4

Summary and Conclusions

4.1 Summary

This research presented the utilization of Parallam® structural composite lumber panels as a bridge deck replacement alternative. CFC-WVU designed two short span bridges in Barbour County, WV and launched a monitoring program through a series of live load tests. The performance of the two bridges was evaluated under heavy truck loads. In-service bridge performance parameters were determined in the field to verify the design assumptions and to ensure a satisfactory performance of the two bridges. Strain and deflection data were used to evaluate the DLA factors, Transverse Load Distribution Factors and truck load deflections.

4.2 Conclusions and Recommendations

<u>Based on the field data collected for the two bridges the following conclusions are drawn:</u>

1. The maximum strain DLA values were 53% and 174% for Peel Tree Bridge and Hackers Creek Bridge respectively, which are much higher than the DLA value of 0.33 recommended by AASHTO LRFD specifications. It is recommended that more testing should be done to arrive at a more accurate DLA for the design.

2. The DLA values for Peel Tree Bridge were as low as 1% and as high as 53% and the DLA values for Hackers Creek Bridge were as low as 1% and as high as 174%. For the same measurement locations, different DLA values were observed at different dates.

From these field data, it can be concluded that a probabilistic approach should be adopted to evaluate the DLA (Bakht and Pinjarkar, 1989).

3. In some cases, negative DLA values were observed for the exterior stringers which could be attributed to the transverse position of the test truck when crossing the bridge. It is recommended that more testing should be done to investigate this behavior.

4. The deck deflections under static live load are within the design limit of $S_{eff}/500$ for both bridges.

5. The maximum global live load deflections under the test truck load (15% higher than AASHTO HS-25) were L/1081 and L/1583 for Peel Tree and Hackers Creek bridges respectively which are lower than the design limit state of L/800.

6. For both bridges, the measured stringer strains and deck component strains are well within the allowable strains under a truck load 15% higher than AASHTO HS-25 load.

7. The S/5 Transverse Load Distribution Value (TLDF) used for the design is very conservative.

The visual inspection of Peel Tree Bridge revealed the Following:

1. The deck of Peel Tree Bridge suffered from wear and rutting in the wheel path.

2. Gravel from the roadway has been driven onto the deck, and then embedded into the deck causing indentations.

3. Chipping and rolling of the Parallam deck was noticed between the deck and the approach.

4. The bottom of the deck, the deck-to-stringer connectors, the panel joiners, the guardrail and the seats are in very good condition.

5. The average moisture content of the deck was below the 19% limit set by AASHTO bridge design specifications.

6. The deck creep is virtually zero.

The visual inspection of Hackers Creek Bridge revealed the Following:

1. The bottom of the deck was in a very good condition.

2. Vertical movement of the deck panels relative to one another was observed under traffic load.

3. Nine deck-to-stringer spring connectors failed at the bolt locations in Hackers Creek Bridge and the mode of failure was brittle.

4. The wearing surface showed no signs of reflective cracking, however, small dips were noticed in the wearing surface that coincide with the deck panels joint locations.

5. The average moisture content of the deck was within the permissible limit of 19%.

6. The creep of the deck panel is close to zero.

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