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INVESTIGATION OF THE PERFORMANCE OF CROSS-LAMINATED TIMBER DECK PANELS ON LONGITUDINAL STEEL BRIDGE GIRDERS

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ABSTRACT: The use of cross-laminated timber (CLT) has become more popular, with many advances stemming from research and construction projects in Europe. CLT has been utilized in vertical construction projects where many inherent features have been maximized. CLT is prefabricated, relatively lightweight, dimensionally stable, and sustainable. Despite these advances, the use of CLT in bridge structures has been limited, and the adoption of CLT into governing bridge design codes has been slow in North America. Results of a laboratory investigation assessing the feasibility of CLT as a primary structural material for highway bridge deck applications are reported.The investigation focused on the strength and serviceability of a transverse deck panel system supported by longitudinal steel girders, which was tested under service loading to determine the structural behavior. Results of longitudinal deck panel tests are reported in Dahlberg et al. [1] and Dahlberg et al. [2].

KEYWORDS: cross-laminated-timber (CLT), laboratory tests, bridge deck, transverse panels

1 – INTRODUCTION

Modern cross-laminated timber (CLT) originated from collaborative research efforts between industry and academia in Austria during the mid-1990s as an alternative to traditional building materials such as concrete, masonry, and steel. Adoption of CLT grew rapidly in Europe in the early 2000s, driven by the green building movement, improved product efficiencies, approvals, and enhanced marketing and distribution networks. Today hundreds of European mid-rise and high-rise projects have been successfully completed using CLT.

While CLT initially become well-established in Europe, its implementation in the United States and Canada only began gaining momentum in the 2010s. For instance, the U.S. edition of the *CLT Handbook* was published in 2013 to support the U.S. design and construction industry [3]. Over the past decade, CLT has gained significant popularity in the United States, driven by advancements originating from European research and construction projects, a North American CLT product performance standard (PRG-320 [4]), and adoption in US building codes.

CLT has proven particularly effective in vertical construction projects, where its inherent qualities—such as prefabrication, lightweight properties, dimensional stability, and sustainability—are fully utilized. However, its application in bridge structures remains limited, and its adoption into governing bridge design codes has been slow.

CLT holds promise as a complementary or alternative material for bridge decks. Use of CLT in bridge projects remains rare, particularly in North America, with no notable projects yet completed in the United States. Two examples of CLT use in North America are the Mistissini Bridge (2014) and the Maicasagi Bridge (2011), located in Quebec, Canada. In these projects, CLT was chosen for its locally sourced materials and reduced lead times compared to conventional materials, and both were deemed successful demonstrations of CLT's capabilities. Nevertheless, CLT is seldom considered for bridges despite advantages such as its strength-to-weight ratio comparable to concrete, dimensional stability, and the potential use of underutilized timber species. To gain wider acceptance, additional research and proof-of-concept projects are essential.

Characterizing the structural behavior of CLT panels used for bridge decks is essential to support their integration into North American bridge design codes. Currently, CLT is not recognized in the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* [5]. In contrast, glue-laminated timber has been acknowledged by AASHTO for use in treated bridge structures for several decades. Given the similarities between these materials, continued research and data generation could support CLT's viability and eventual inclusion in AASHTO bridge design standards, paving the way for its broader adoption in the bridge industry.

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These tests directly measured the structural performance of the CLT panels, providing valuable data to advance their adoption in bridge design standards.

2 – RESEARCH SURVEY

A primary task for this investigation was to gather information on advanced technologies and design practices related to the implementation of CLT, not only in the United States but also in European countries and Canada, where CLT technology has seen earlier adoption and greater advancements. To achieve this objective, the project team documented existing design details for CLT, including its development in North America and Europe, methods for connecting CLT decks to girders, typical spans between girders, and connections between adjacent CLT panels. Additionally, a survey was conducted among CLT manufacturers and pressure treatment facilities to assess the size limits of CLT panels that can undergo single-pressure treatment.

2.1 – CLT DESIGN PRACTICES

While CLT has become well-established in Europe, its implementation in the United States and Canada only began gaining momentum in the 2010s. The U.S. edition of the *CLT Handbook* was published in 2013 to support the U.S. design and construction industry [3].

In North America, CLT design values are established in accordance with the ANSI/APA PRG 320 standard [4] and are certified by approved agencies based on qualification and mechanical test requirements specified in the standard. Annex A of PRG-320 outlines seven representative layups to assist manufacturers in validating calculations, though it does not mandate using these layups. Manufacturers are free to develop and certify their own layup configurations.

CLT panels are composed of multiple layers of lumber boards stacked crosswise (typically at 90 degrees) and bonded together on their wide faces, with occasional bonding on the narrow faces. A typical CLT panel consists of at least three glued layers of boards arranged orthogonally, with each layer alternating orientation relative to its neighbors. Lumber piece thickness ranges from 16 mm to 51 mm, while widths typically observed in production range from 61 mm to 241 mm. Boards are finger-jointed with structural adhesive and kiln-dried, with visual grading or machine stress rating applied. Panel sizes vary by manufacturer; common widths are 0.6, 1.2, 2.4, and 3.0 m, with lengths up to 18.3 m and thicknesses up to 508 mm [3].

Advantages of CLT are its high high in-plane and out-of-plane strength and stiffness, enabling two-way action similar to a reinforced concrete slab. The cross-lamination provides a "reinforcement" effect, significantly enhancing splitting resistance for certain connection systems. For floor and roof systems, the outer layers are oriented parallel to the primary span direction to optimize load capacity.

2.2 – PRESSURE TREATMENT OF CLT

Like other timber bridge types, protecting CLT panels for exterior use is crucial to ensuring long-term serviceability. However, the ability to pressure-treat full CLT panels is constrained by the limitations of current manufacturing capabilities. In the United States, standard chamber sizes typically restrict panel widths to no more than 2.1 meters. Additionally, pressure-treating individual members before assembling them into a CLT panel is not cost-effective, presenting further challenges. Recognizing that effective protection measures are key to the broader acceptance of CLT panels, researchers recommend further investigation to determine the best strategies for safeguarding CLT panels from exposure to outdoor elements.

3 – EXPERIMENTAL PROCEDURES

In this project, CLT deck panels spanning transversely across steel girders were designed following the load requirements outlined in the *American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (BDS)* [5]. Full-scale laboratory tests were conducted to evaluate both the strength and serviceability of the panels under typical highway-type loads. Instrumentation, including strain and deflection gauges, was used to collect data for structural characterization and comparison, with loading applied at specific deck locations to simulate highway vehicle traffic. These tests directly measured the structural performance of the CLT panels, providing valuable data to advance their adoption in bridge design standards.

3.1 – CONSTRUCTION AND MATERIALS

At the Iowa State University Structures Laboratory, the project team constructed and tested a portion of a full-scale bridge assemblies comprised of steel girders and transverse CLT panels.

The steel girders used for both bridge assemblies were W610x82 (W24x55 AISC) steel girders (σ_y=345 MPa) with a clear span of 7.62 m. Two girder configurations were tested to evaluate the CLT deck performance under varying girder spacing. The first configuration used 4 girders spaced 1.40 m apart, while the second configutation used 3 girders spaced 2.10 m apart. The total width of the assembly was 4.88 m, representing approximately half the width of a typical in-service bridge, while the joined panels totaled 7.31 m in the longitudinal direction.

The CLT deck for each assembly was comprised of 5-ply panels made from two grades of Douglas fir: select structural for longitudinal layers (F_b = 10.3 MPa,) and No. 2 for transverse layers (F_b = 6.2 MPa). The deck edge profile is shown in Figure 1. Each layer was 35 mm thick. The layup of the panels was determined based on the loads provided to the CLT manufacturer and is not a basic layup as described in PRG 320, nor is it certified for future production.

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Figure 1 Panel Edges Joined with Half-Lap Joint

Edges of the panels were joined using a 76 mm wide half-lap joint, a common connection in vertical construction and nail-laminated timber bridge deck panels. The joints were fastened with screws: vertical screws (ASSY VG CYL, 8 mm diameter, 159 mm long) and inclined screws (ASSY VG CSK, 10 mm diameter, 219 mm long). The screw configuration along the joint is shown in Figure 2. The inclined screws were chosen for their stiffer connections and higher load capacities.

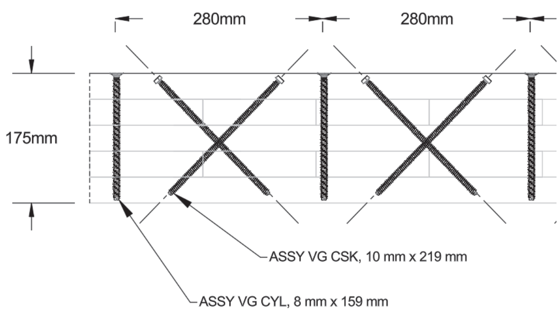

Figure 2 Half-Lap Joint Screw Pattern

The panels spanned transversely across the girders for both assemblies and were fastened to the top flanges of the steel girders and a 38x191 mm solid-sawn board laid flatwise along the flange. Dome-head through bolts (13 mm diameter) were spaced 305 mm apart on each side of the flange to secure the panels as shown in Figure 3.

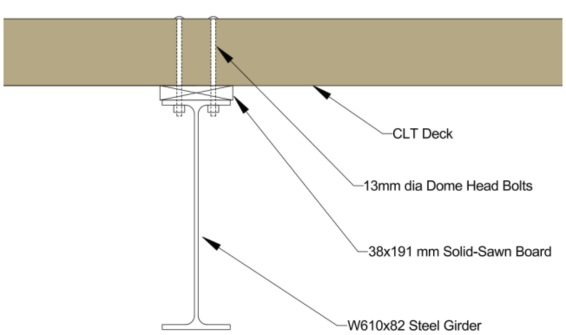

Figure 3 CLT Panel Connection to Girder

3.2 – LOAD CASES

Four static load cases were conducted on each of the four-girder and three-girder assemblies. The load cases simulated the rear tandem axle of a dump truck, with loads applied at four specific locations on the deck to replicate the AASHTO HL-93 vehicular live loading design tandem notional load. Each axle was represented with a maximum load of 111 kN (55.5 kN per tire), and the load locations were selected to maximize shear and bending reactions within the panels. Figure 4 and Figure 5 show examples of a test configuration and load placement.

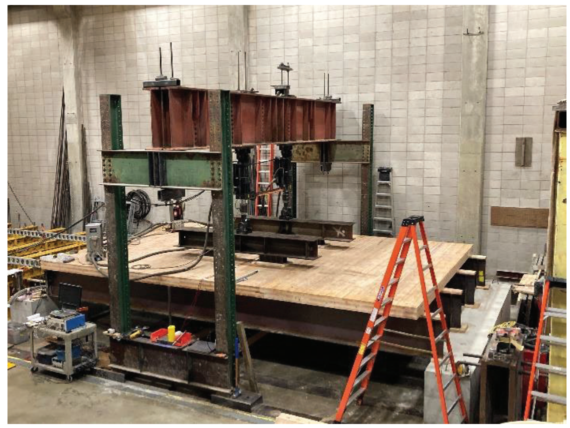

Figure 4 Example of Transverse Panel Test Configuration

Figure 5 Load Placement used for Transverse Panel Deck Tests

After the initial static tests, the three-girder assembly underwent an additional 500,000 cyclic loads to simulate a lifetime of service, followed by a final static test. Comparing the midspan strain and deflection values from the pre- and post-cyclic static tests provided an approximate assessment of potential changes in structural behavior. Both the cyclic and final static test on the three-girder assembly used only Load Case 2 positions.

3.3 – INSTRUMENTATION

Deflection and strain gauges were strategically placed to capture key performance metrics. Deflection gauges were positioned beneath each girder at midspan. Strain gauges were installed on the bottom flange of the steel girders at

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midspan and on the top and bottom of the CLT panels in their strong direction, equally spaced between the girders.

The strain and deflection data collected at the midspan of the girder give a good indication of the transverse distribution of the applied load.

The deflection gages at the girder midspan also allow a direct comparison of the global deflection to the recommended maximum of L/425 for wooden bridges or L/800 for steel bridges provided in AASHTO LRFD BDS [5]. Given the mixed use of materials in this investigation, both recommended limits are given and the reader is encouraged to make comparisons to both. It is important to note that AASHTO does not require deflection limits be met aside from a few structure types.

From a structural perspective, excessive deflections in wood components can lead to loosening of fasteners and the cracking or failure of brittle materials such as asphalt pavement. Noticeable sagging not only creates an undesirable visual appearance but may also raise public concerns about structural adequacy. Additionally, deflections from moving vehicle loads can induce vertical motion and vibrations, which may be disruptive to motorists. While not required, proper deflection control is essential to ensure both long-term performance and public confidence in timber bridge structures.

The strain data collected on the top and bottom of the CLT panel in the major strength direction are converted to stress to generally compare to the reference design values for allowable bending stress (F_b) of select structural Douglas fir. Note that, for design, the reference design values are modified by adjustment factors for moisture, flat-use, time effect, etc.

Figure 6 illustrates the configurations and instrumentation plans for the four-girder and three-girder assemblies, respectively. Figure 7 details the load case placement for each assembly, showing the transverse gauge position (0 m) at the deck edge nearest to Load Cases 1 and 3.

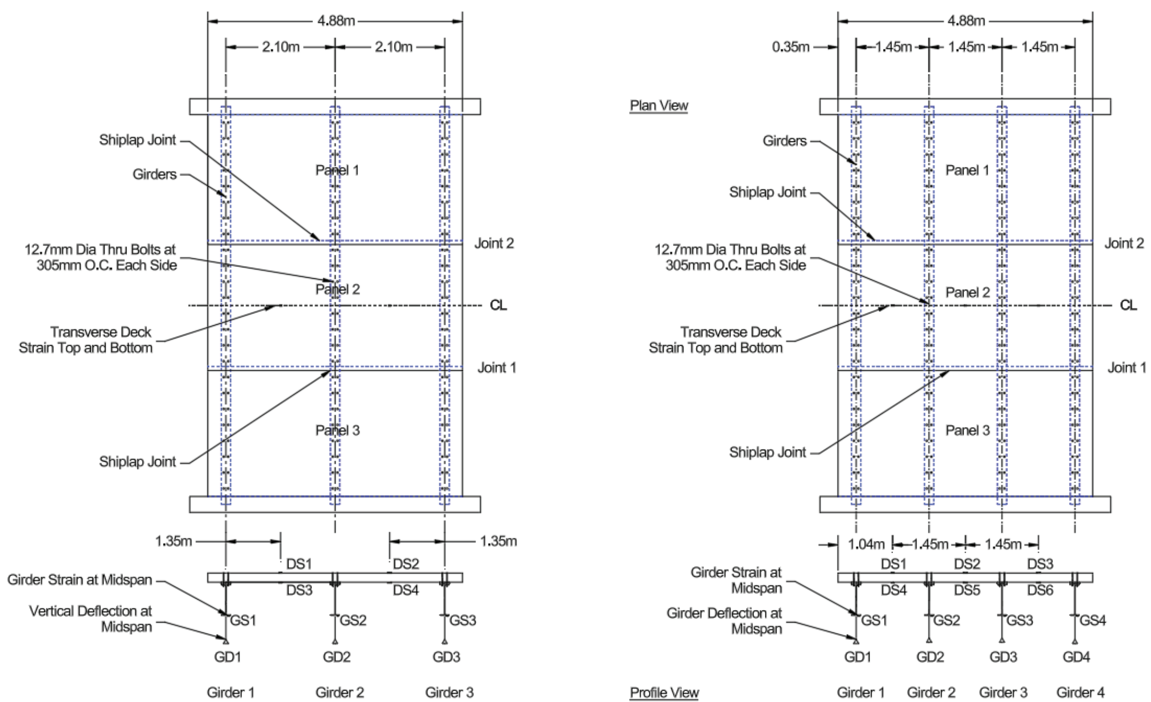

Figure 6 Transverse deck panel and instrumentation configuration for 3- and 4-girder assemblies

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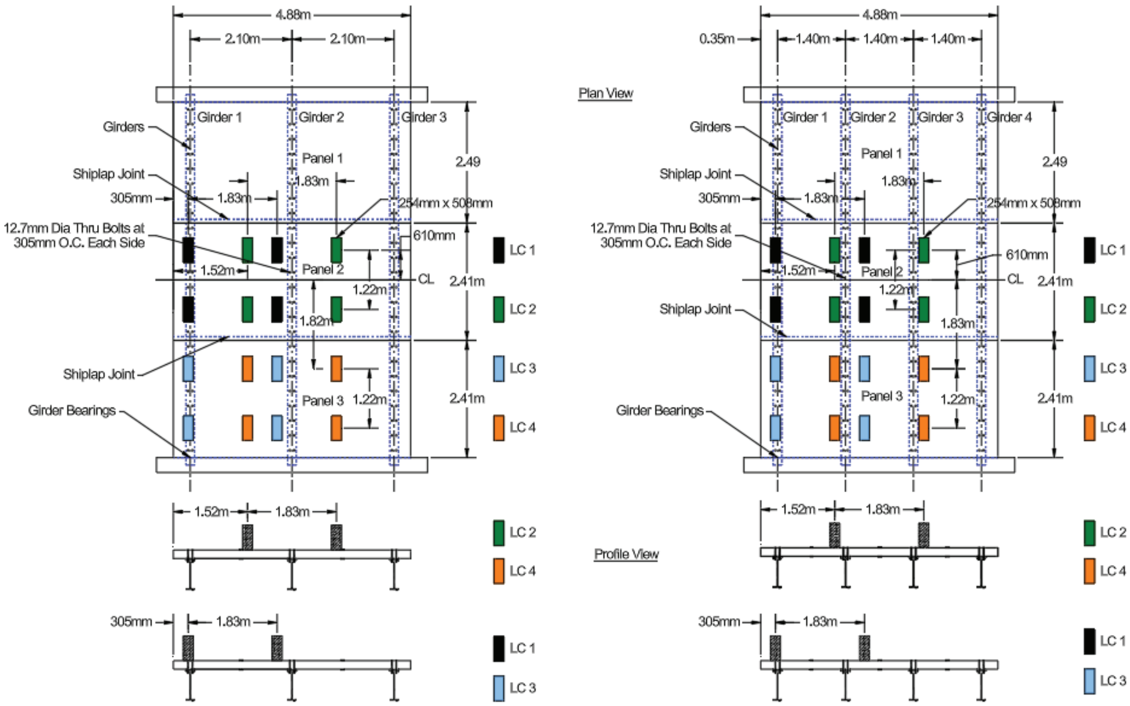

Figure 7 Location of rear-axle tandems for each load case for 3- and 4-girder assemblies

4 – RESULTS

4.1 – FOUR GIRDER ASSEMBLY

Midspan Girder Strain and Deflection

For Load Cases 1 through 4, the midspan girder deflection and corresponding span-to-deflection (L/D) ratio were measured and calculated, along with the midspan girder strains and their corresponding maximum stresses. The results are summarized as follows:

Load Case 1 (Figure 8): Maximum deflection: 7.1 mm; tensile strain: 415 microstrain. Corresponding (L/D): 1028; live-load steel stress: 83 MPa.

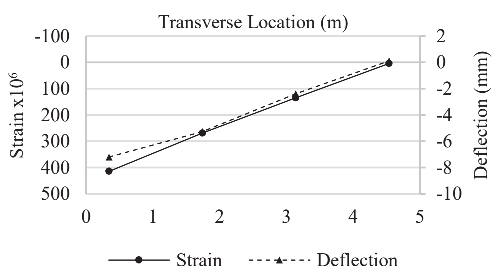

Figure 8 Load Case 1 Strain and Deflection – 4-Girders

Load Case 2 (Figure 9): Maximum deflection: 5.3 mm; tensile strain: 265 microstrain. Corresponding L/D: 1371; live-load steel stress: 55 MPa.

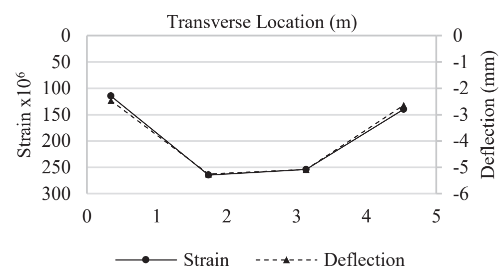

Figure 9 Load Case 2 Strain and Deflection – 4-Girders

Load Case 3 (Figure 10): Maximum deflection: 3.8 mm; tensile strain: 175 microstrain. Corresponding L/D: 1920; live-load steel stress: 35 MPa.

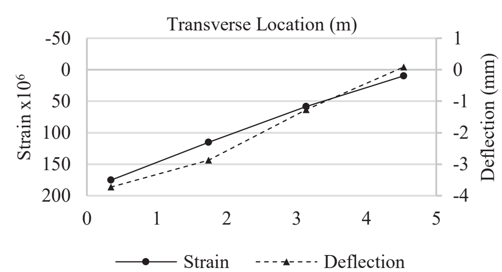

Figure 10 Load Case 3 Strain and Deflection – 4-Girders

Load Case 4 (Figure 11): Maximum deflection: 2.8 mm; tensile strain: 98 microstrain. Corresponding L/D: 2618; live-load steel stress: 21 MPa.

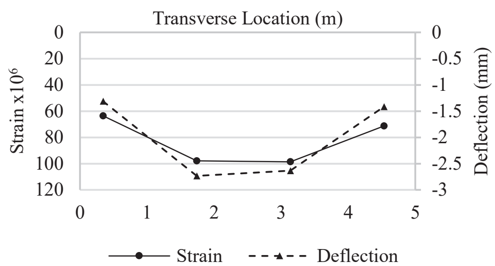

Figure 11 Load Case 4 Strain and Deflection – 4-Girders

In all cases, the span-to-deflection ratios and live-load steel stresses were within acceptable limits, indicating satisfactory performance relative to the recommended deflection criteria and the yield stress of the steel.

Midspan CLT Panel Strain

The midspan strain values and corresponding stresses assuming a modulus of elasticity of 11,700 MPa for select structural Douglas Fir for the top and bottom of the CLT panels, measured in the transverse (strong) direction, are summarized as follows:

Load Case 1 (Figure 12): Tensile strain: 98 microstrain; compressive strain: 83 microstrain. Corresponding stresses: 1.15 MPa (tension), 0.97 MPa (compression).

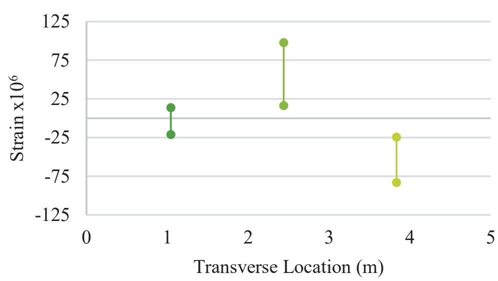

Figure 12 CLT Deck Strain at Maximum Load – Load Case 1, 4-Girders

Load Case 2 (Figure 13): Tensile strain: 165 microstrain; compressive strain: 134 microstrain. Corresponding stresses: 1.94 MPa (tension), 1.57 MPa (compression).

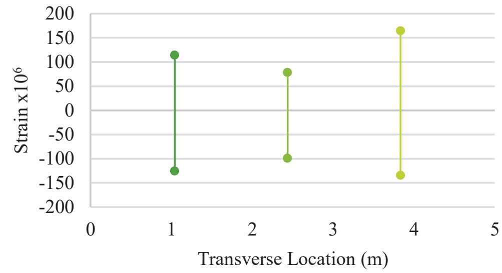

Figure 13 Deck Strain at Maximum Load – Load Case 2, 4-Girders

Load Case 3 (Figure 14): Tensile strain: 10 microstrain; compressive strain: 17 microstrain. Corresponding stresses: 0.14 MPa (tension), 0.21 MPa (compression).

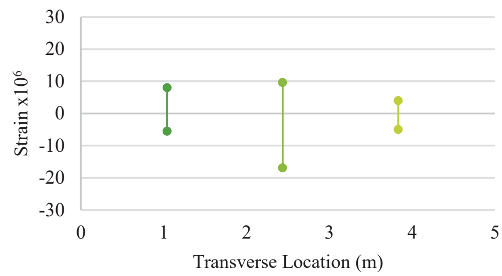

Figure 14 Deck Strain at Maximum Load – Load Case 3, 4-Girders

Load Case 4 (Figure 15) Tensile strain: 52 microstrain; compressive strain: 69 microstrain. Corresponding stresses: 0.61 MPa (tension), 0.83 MPa (compression).

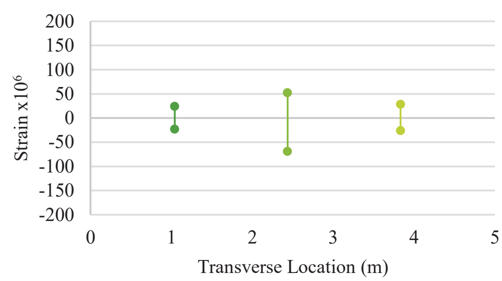

Figure 15 Deck Strain at Maximum Load – Load Case 4, 4-Girders

All strain and stress values remained well within the material's capacity, confirming effective load distribution and structural integrity of the CLT panels.

Four Girder Assembly Summary

The analysis of all load cases demonstrates consistent performance of both the steel girders and CLT panels under various load conditions. Strain, deflection, and stress values for the girders and panels were within recommended limits for serviceability and safety, indicating reliable behavior of the system under applied loads.

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4.2 – THREE GIRDER ASSEMBLY

Midspan Girder Strain and Deflection

For Load Cases 1 through 4, the midspan girder deflection and corresponding span-to-deflection (L/D) ratio were measured and calculated, along with the midspan girder strains and their corresponding maximum stresses. The results are summarized as follows:

Load Case 1 (Figure 16): Deflection: 8.6 mm; bottom-flange tensile strain: 484 microstrain. Corresponding L/D: 847; live-load steel stress: 96.5 MPa.

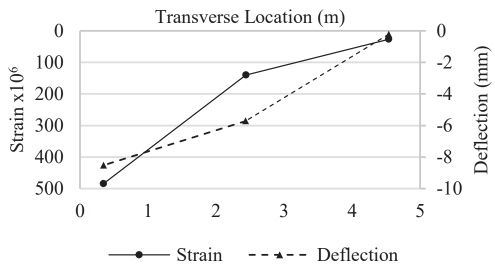

Figure 16 Load Case 1 Strain and Deflection – 3-Girders

Load Case 2 (Figure 17): Deflection: 7.6 mm; bottom-flange tensile strain: 197 microstrain. Corresponding L/D: 960; live-load steel stress: 41.4 MPa.

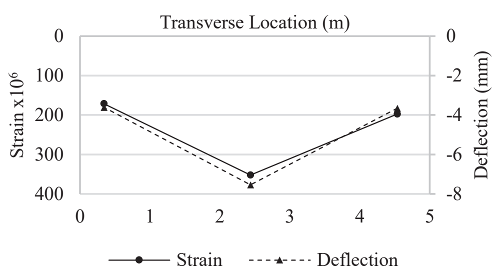

Figure 17 Load Case 2 Strain and Deflection – 3-Girders

Load Case 3 (Figure 18**):** Deflection: 4.6 mm; bottom-flange tensile strain: 212 microstrain. Corresponding L/D: 1600; live-load steel stress: 41.4 MPa.

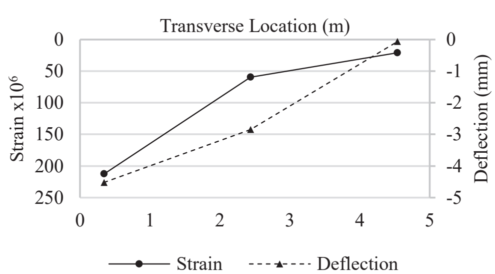

Figure 18 Load Case 3 Strain and Deflection – 3-Girders

Load Case 4 (Figure 19): Deflection: 3.8 mm; bottom-flange tensile strain: 94 microstrain. Corresponding L/D: 1920; live-load steel stress: 18.8 MPa.

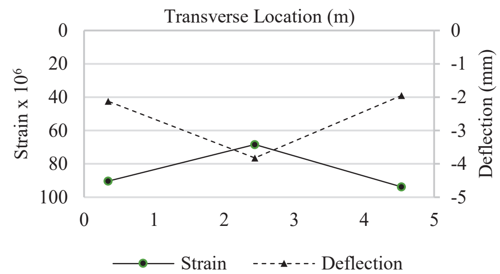

Figure 19 Load Case 4 Strain and Deflection – 3-Girders

For all cases, deflection and live-load steel stresses remained well within acceptable limits, indicating satisfactory performance relative to recommended service deflection criteria and the steel's yield stress.

Midspan CLT Panel Strain

The midspan strain range for the top and bottom of the CLT panels, measured in the transverse (strong) direction, is summarized below:

Load Case 1 (Figure 20): Peak tensile strain: 129 microstrain; compressive strain: 106 microstrain. Corresponding stresses: 1.52 MPa (tension), 1.24 MPa (compression).

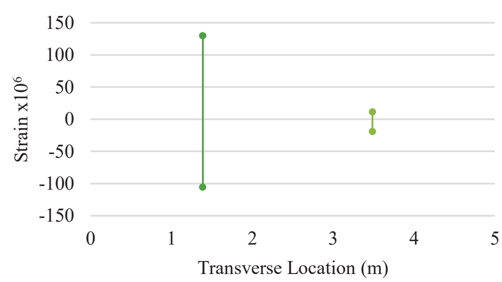

Figure 20 Deck Strain at Maximum Load – Load Case 1, 3-Girders

Load Case 2 (Figure 21): Peak tensile strain: 443 microstrain; compressive strain: 301 microstrain. Corresponding stresses: 5.17 MPa (tension), 3.52 MPa (compression).

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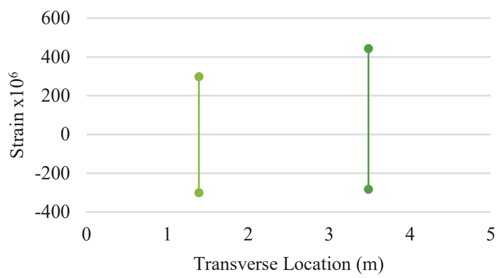

Figure 21 Deck Strain at Maximum Load – Load Case 2, 3-Girders

Load Case 3 (Figure 22): Peak tensile strain: 23 microstrain; compressive strain: 13 microstrain. Corresponding stresses: 0.28 MPa (tension), 0.14 MPa (compression).

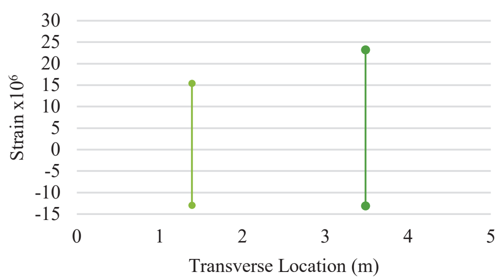

Figure 22 Deck Strain at Maximum Load – Load Case 3, 3-Girders

Load Case 4 (Figure 23): Peak tensile strain: 67 microstrain; compressive strain: 49 microstrain. Corresponding stresses: 0.76 MPa (tension), 0.55 MPa (compression).

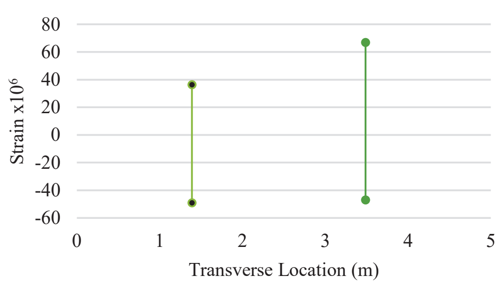

Figure 23 Deck Strain at Maximum Load – Load Case 4, 3-Girders

Across all cases, the strain and stress values were within the material's capacity, demonstrating effective load distribution and structural integrity of the CLT panels.

Three Girder Assembly Summary

The results from Load Cases 1–4 confirm that the girders and CLT panels performed well under applied loads. Deflection, strain, and stress values for both components remained within recommended serviceability and safety limits, indicating reliable behavior of the bridge configuration under maximum loading conditions.

4.3 – LOAD DISTRIBUTION COMPARISON

Figure 24 and Figure 25 compare the midspan girder deflection values for the three-girder and four-girder assemblies during Load Case 1 and Load Case 2 (midspan loading), respectively. These plots illustrate the transverse load distribution across the bridge and highlight differences in deflection magnitude and load-sharing behavior between the two configurations. The recommended deflection limit of L/425 equates to a maximum deflection of 17.2 mm which is more than twice the magnitude of any of the recorded deflection values.

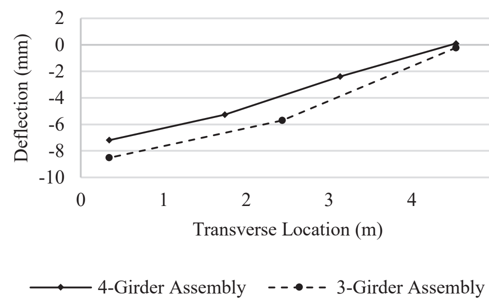

Figure 24 Deflection Comparison for Load Case 1

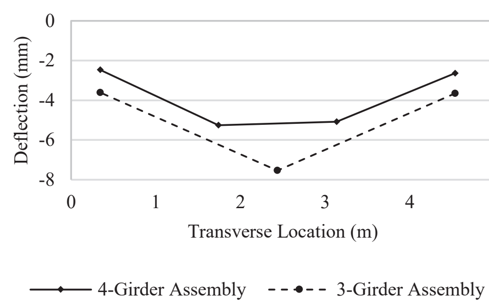

Figure 25 Deflection Comparison for Load Case 2

Table 1 presents the calculated load distribution factors for Load Cases 1 and 2**,** derived from girder deflection data. It is important to note that edge girders typically exhibit lower stiffness compared to interior girders, as they support a smaller deck area. However, this variation in stiffness has not been accounted for in the values presented in the table.

Table 1 Calculated Load Distribution Factor for Load Cases 1 and 2

	Load Distribution Factor			
	Load Case 1		Load Case 2	
	3-Girder	4-Girder	3-Girder	4-Girder
Girder 1	0.59	0.48	0.37	0.16
Girder 2	0.40	0.35	0.25	0.34
Girder 3	0.02	0.16	0.38	0.33
Girder 4	--	0.01	--	0.17

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4.4 – CYCLIC TEST RESULTS

The three-girder assembly underwent a cyclic load test to evaluate whether structural performance was affected by repeated loading and unloading, simulating years of use and truck crossings. The test applied 500,000 cycles at the Load Case 2 position, with a total load of approximately 222 kN per cycle, distributed as 55.5 kN at each point of contact.

Grider deflection for all three girders, the girder tensile strain, and the top and bottom deck strain values were collected throughout the test duration.

In all cases, the structural performance showed no significant changes over the course of 500,000 cycles. The measured values remained consistent with those obtained during the initial Load Case 2 static test. This consistency indicates that the system maintained its integrity and load-carrying capacity under prolonged cyclic loading.

A static test for Load Case 2 was conducted following the completion of the cyclic load test to evaluate any structural changes resulting from the cyclic loading.

The maximum strain and deflection values observed after the cyclic load test closely matched those recorded before the cyclic test (Figure 26). These results indicate that the cyclic loading had no significant impact on the structure, with its strength and stiffness remaining virtually unchanged.

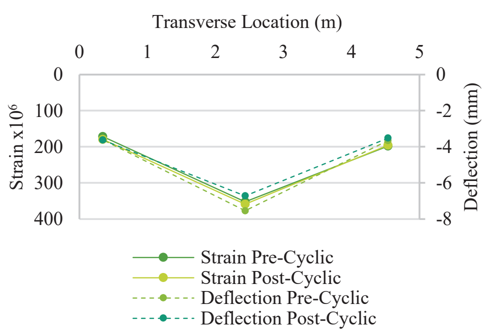

Figure 26 Load Case 2 Strain and Deflection – 3 Girders, Pre- and Post-Cyclic Loading

5 – CONCLUSION

Valued for their prefabrication, lightweight design, dimensional stability, and sustainability, CLT panels are well-established in vertical construction but have seen limited use in North American bridge decks.

While CLT panels perform comparably to other bridge types of similar size, barriers to adoption include the high cost of pressure treatment for durability, lack of waterproof adhesives for wet environments, and design standards like PRG 320 that restrict CLT to indoor conditions with controlled moisture levels. Despite these challenges, CLT panels exhibit strong structural potential for bridge applications.

This study evaluated the performance of two portions of full-scale bridge assemblies comprised of steel girders and transverse CLT panels under highway-type loads. Both assemblies displayed uniform, predictable behavior and met AASHTO Load and Resistance Factored Design [5] guidelines, including deflection limits (L/425), which are critical for serviceability. Transverse load distribution across panels was effective, and cyclic load tests simulating years of truck crossings (500,000 cycles) showed no structural degradation, confirming durability under repeated use.

Overall, CLT panels meet structural requirements for bridge decks, offering performance consistent with existing timber bridge types. Addressing durability challenges and improving transverse load distribution could advance CLT's viability for modern infrastructure, providing a sustainable and versatile alternative for highway bridge construction.

The structural characteristics of the panels lend well to using them for highway bridge structures. The data prove the performance to be uniform and predictable. Overall, the structural performance of CLT panels under highway-type loads is consistent with other allowable bridge types of similar size.

6 – REFERENCES

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