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LABORATORY INVESTIGATION OF CROSS-LAMINATED DECKS FOR BRIDGE APPLICATIONS

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ABSTRACT: The use of cross-laminated timber (CLT) has gained popularity during the past decade in North America, with many advances stemming from completed research and construction projects located in other countries. In particular, CLT has been utilized in vertical construction projects where many of its inherent features have been maximized. Despite these advances, the use of CLT in bridge structures has been limited and CLT has not been adopted into governing bridge design codes.

This paper reports the results and conclusions of the laboratory investigation of the feasibility of CLT as a primary structural material for highway bridge deck applications. Two common timber bridge superstructure systems are used in the United States: (1) longitudinal deck panels spanning the bridge abutments, and (2) transverse deck panels across the bridge width supported by longitudinal girders. The subject of this paper is the longitudinal CLT bridge deck system, which was tested under service loading to determine the structural behavior under static loading.

KEYWORDS: Cross-Laminated Timber, Bridge, Laboratory Investigation, Longitudinal Deck

1 INTRODUCTION

The use of cross-laminated timber (CLT) has gained popularity during the past decade, with many advances stemming from completed research and construction projects located in Europe. In particular, CLT has been utilized in vertical construction projects where many of its inherent features have been maximized. To name a few, CLT is prefabricated, relatively lightweight, dimensionally stable, and environmentally sustainable. North American design resources and standards have been developed for CLT in the form of PRG 320 [1], CLT Handbook [2], and the National Design Specification (NDS) [3] but these effectively limit the use of CLT to indoor applications where the moisture content can be maintained below 16%. The use of CLT in bridge structures has been limited and the adoption of CLT into governing bridge design codes has been slow-going.

CLT shows promise as a complementary or alternative construction material in bridge decks, and additional research evaluating the structural performance of these decks would further guide the appropriate use in bridge projects. This paper provides a summary of research conducted to characterize the structural characteristics of CLT bridge decks subjected to typical traffic loads.

CLT was first introduced in Europe in the 1990s to provide a potential alternative to more commonly used building materials such as concrete, masonry, and steel. Since that time, CLT and its production and distribution have vastly improved allowing for rapid growth and use. The building industry, in particular, has seen hundreds of projects completed using CLT, with mid-rise and high-rise buildings constituting the bulk of these projects.

The use of CLT for bridge projects has been very limited with very few in North America and none in the United States. In fact, there have been only a few notable bridge projects to incorporate CLT including the Mistissini Bridge located in Quebec, Canada, constructed in 2014 [4] and the Maicasagi Bridge located north of Quebec constructed in 2011 [5]. Both were long span bridges and the CLT was composite with other materials. Regardless, in each case, CLT was selected for its locally sourced material and shorter lead time compared with more commonly used materials. Both of these projects were considered a success and provide an example of CLT capabilities. Despite this fact, CLT is rarely considered for use in bridge applications, even with several inherent advantages. Additional research and proof of concept is

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required to help make CLT a viable bridge construction material to be used by designers and engineers.

CLT is not currently recognized in the American Association of State Highway and Transportation Officials (AASHTO) load and resistance factor design (LRFD) bridge design specifications (BDS) [6]. Glue-laminated timber has been recognized by AASHTO for use in treated bridge structures for several decades. The inherent similarities between each bridge type along with continued proof-of-concept research should provide supporting data for CLT viability and acceptance by AASHTO in the bridge industry.

2 RESEARCH METHODS

2.1 PRE-TEST INVESTIGATION

The research team conducted a literature review of bridge designs using CLT in the United States, Canada, and European countries where CLT technology has been more quickly advanced and implemented. A review of deck materials, including species, size, and layup, was conducted. Additionally, the viability of CLT with respect to economics, strength, and serviceability was reviewed. Existing details for CLT from any available bridge plans and vertical construction were reviewed for viability and adoption to bridge structure use. CLT manufacturers were surveyed to identify the capabilities of pressure treatment facilities and assess the size limits of CLT panels that can be singly pressure treated.

CLT panels consist of an odd number of layers of flatwise lumber boards or structural composite boards stacked crosswise (typically at 90 degrees) and glued together under pressure. Modern CLT initially resulted from industry–academia joint research efforts in Austria in the middle 1990s. The use of CLT increased significantly in Europe in the early 2000s, partially driven by the green building movement and better efficiencies, product approvals, and improved marketing and distribution channels.

While this product is well established in Europe, the work toward the implementation of CLT products and systems has only recently begun in the United States and Canada. The U.S. edition of the CLT Handbook [2] and first CLT performance standard, PRG 320 [1] were published in 2013 with the intent to assist the U.S. building design and construction industry. The CLT Handbook [2] describes only glued CLT; nonglued CLT products are outside the scope of the publication. In 2015, CLT design procedures were introduced in the National Design Specification for Wood Construction, and in 2021, the International Building Code (IBC) [7] incorporated a new class of wood building materials, mass timber, which included CLT. The IBC also limited the construction of mass timber structures to 18 stories high

A cross section of a CLT element has at least three glued layers of boards placed orthogonally, each layer in an alternating orientation to the neighboring layers. Thickness of individual lumber pieces varies from 16 to 51 mm, and the minimum width is 1.75 times the thickness in the major strength direction and 3.5 times the thickness in the minor strength direction. Lumber laminations are visually graded or machine stress rated and dried to 12% moisture content. Panel sizes vary by manufacturer, and typical widths are 0.6, 1.2, 2.4, and 3 m while length can be up to 18 m and thickness can be up to 508 mm [1].

CLT provides relatively high in-plane and out-of-plane strength and stiffness properties, giving it two-way action capabilities similar to a reinforced concrete slab. The 'reinforcement' effect provided by the cross-lamination in CLT also considerably increases the splitting resistance of CLT for certain types of connection systems. Likewise, for floor and roof systems, the outer layers run parallel to the major span direction to maximize the flexural load capacity.

In North America, CLT design values are developed in accordance with the ANSI/APA PRG 320 [1] standard and qualified by an approved agency in accordance with the qualification and mechanical test requirements specified in the standard. PRG-320 Annex A contains seven layups to give the manufacture a means to validate calculations but these are not required layups. Rather, they are representative of what could be manufactured. Each manufacturer is free to develop and certify their own layup configuration.

2.2 PRESSURE TREATMENT

Similar to other timber bridge types, protection of CLT panels for exterior use is essential for long-term serviceability. The ability to pressure-treat full panels is limited by current manufacturer capabilities. Common chamber sizes in the United States would effectively limit the panel sizes to no wider than 2.1 m. Furthermore, pressure treating individual members to be used in a final CLT configuration is disadvantaged by the overall cost. The researchers understand that CLT panel protection measures are critical to their overall acceptance. Further research is recommended to identify best options to protect CLT panels from outdoor elements.

2.3 LABORATORY TESTING

At the Iowa State University structures laboratory, the project team conducted load tests using CLT panels in two configurations. The first configuration was a longitudinal deck panel superstructure system. It used two 314-mm-deep x 2.48-m-wide, 9-lam-layup (Figure 1), single-span panels, which spanned 7.62 m longitudinally between two deck concrete blocks serving as abutments (Figure 2). The second configuration was a transverse deck panel superstructure system. It used three 175-mm-deep x 2.49-m-wide, 5-lam-layup panels spanning transversely across longitudinal steel simply supported W-shape girders. For both bridge configurations tested, untreated Douglas-fir (*Pseudotsuga menziesii*) lumber was utilized for all layers in the CLT bridge deck panels. In the direction of the

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primary span, select-structural graded lumber was used. For the corresponding orthogonal layers, lumber was graded for #2 and better. For brevity, this paper will focus on the 9-lam, longitudinal deck panel configuration, testing, and results.

Figure 1 *CLT Panel for Load Testing*

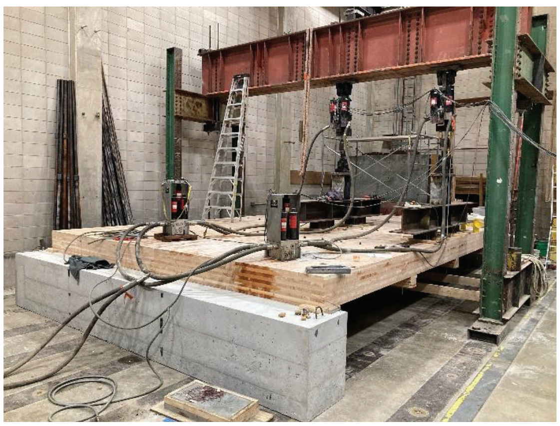

Figure 2 *Longitudinal Panel Test Configuration*

The configuration of the laboratory test setup included two equal-width panels set side-by-side spanning longitudinally as shown in Figure 3. The total width of the CLT deck was 4.88 m. This width constitutes approximately half of an actual in-service bridge. Three built-up spreader beams made up of five 38-mm x 191-mm boards connected with screws and measuring 191 x 216 mm in total were connected to the underside of the deck using 19-mm threaded thru-rods spaced at 305 mm on center, which is consistent with current AASHTO specifications.

Each of the panels were joined along their edge using a shiplap joint, which is a commonly fabricated joint used in the vertical construction industry and in nail-laminated timber bridge deck panels (Figure 4). The overlap width measured 76 mm, and the joint was stitched together using screws in the configuration shown in Figure 5. The vertical screws used were ASSY VG CYL 8 x 300 mm, and the inclined screws used were ASSY VG CSK 10 x 400 mm. This screw configuration was used because inclined screws tend to produce a stiffer connection with higher load capacities.

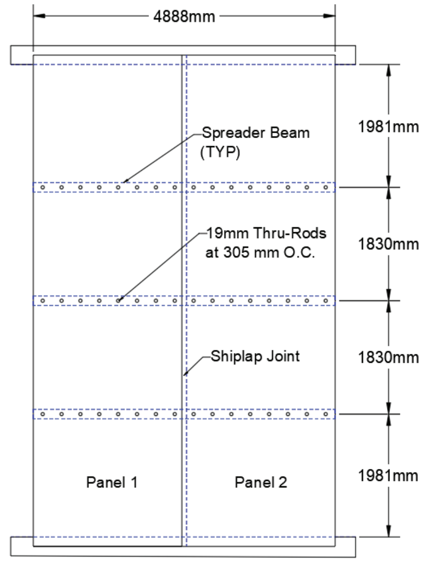

Figure 3 *Plan View of Longitudinal Deck Panel Configuration*

Figure 4 *Shiplap Connection used for Longitudinal Panel Deck*

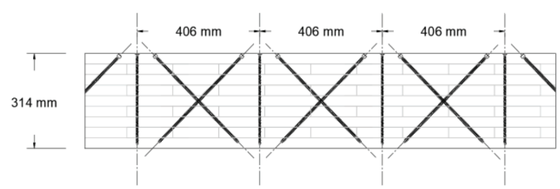

Figure 5 *Screw Pattern along Longitudinal Panel Shiplap Joint*

The tests were completed using typical highway service loads to simulate highway vehicle traffic. The panels were subjected to four load cases (LC), which simulated the rear tandem axle of a dump truck by placing loads at four specific locations as shown in Figure 6. The load, axle configuration, and tire spacing are consistent with the HL-93 AASHTO vehicular live loading design tandem. The locations were selected to maximize the shear and bending reactions within the panels and to determine how the applied force was distributed transversely across the panels, with a specific interest in how the longitudinal joint affected load distribution. The maximum applied load was 111 kN per axle, or 56 kN per tire location.

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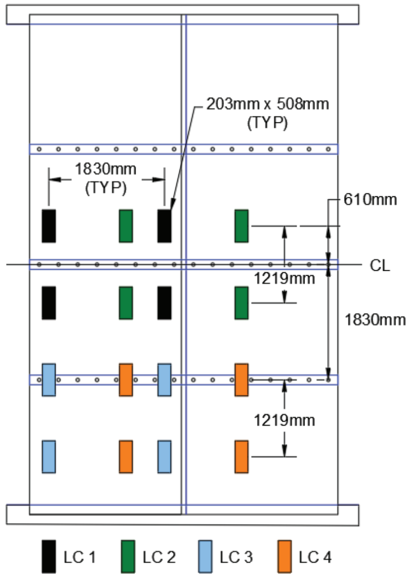

Figure 6 *Location of Rear-Axle Tandems for each Load Case*

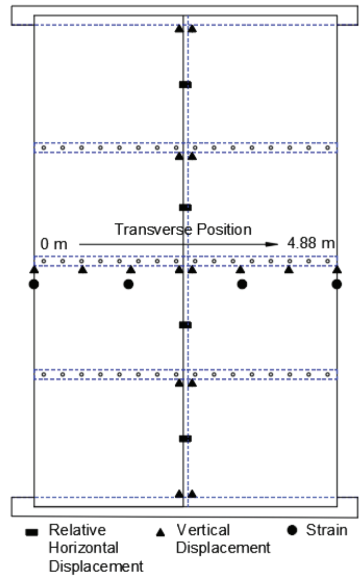

Figure 7 *Deflection and Strain Gage Placement*

2.4 DATA ACQUISITION

Strain and deflection gages were used to gather data for the purposes of structural characterization and comparison.

A series of deflection gages were placed transversely at midspan and along the longitudinal joint to capture the vertical deflection. Additional deflection gages were placed horizontally along the longitudinal joint to capture any relative movement or joint opening between the two panels.

Strain gages were placed at the top and bottom of the panel edges. During the manufacturing process, because of the overall panel width specifications, the outermost longitudinal boards were only 10 mm wide. This portion of the uppermost and lowest laminations was removed to allow the strain gage to engage the first full-width board. Top and bottom strain gages were also placed at approximately 1500 mm from each panel edge. The gage locations are shown in Figure 7.

At one of the panel edges (0-m transverse position), additional strain gages were placed on each of the longitudinal laminations. These gages were intended to determine the strain profile of the panel when loaded.

One horizontal displacement gage was placed at the lowermost transverse lamination to determine if any gapping occurs between individual members. The gages on the edge of panel are shown in Figure 8 and Figure 9.

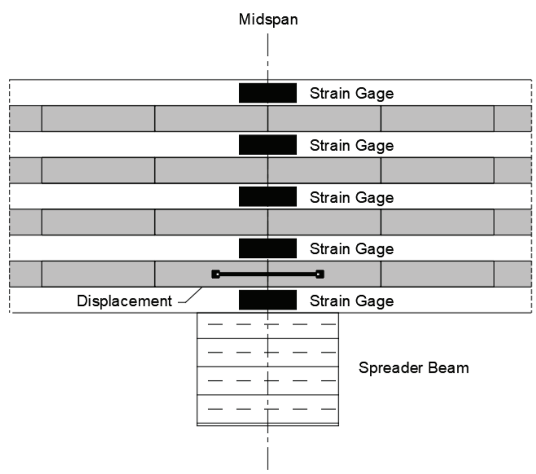

Figure 8 *Schematic of Strain Gages at Panel Edge*

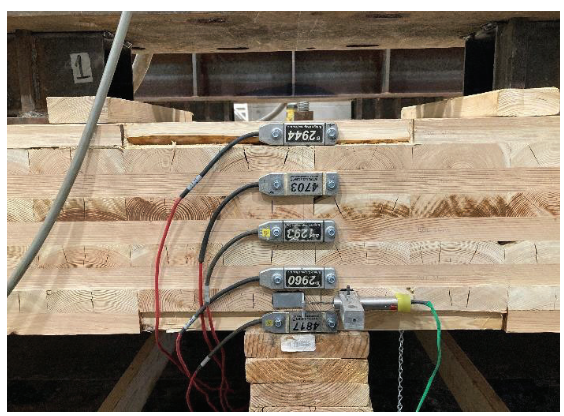

Figure 9 *Strain Gages at Panel Edge*

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3 CLT PANEL EVALUTION

3.1 STRUCTURAL CHARACTERIZATION

Photographs of the reaction frame and load position for each load case location are shown in Figure 10 through Figure 13. Plots of the corresponding strain and deflection behaviour at midspan at the time of maximum load for each of the four load cases are represented in Figure 14 through Figure 17.

For Load Case 1, Figure 15 indicates that when the entirety of the load was placed on Panel 1, the deflection was greatest at the panel edge with approximately 39 mm of deflection. This corresponds to a span-to-deflection ratio (L/D) of 197. The deflection values linearly decreased relative to the transverse position until the minimum value of 1.3 mm was measured at the opposing panel edge. The strain values ranged between 545 and 572 microstrain in compression and tension, respectively, at the panel edge with a relatively linear strain profile through the depth of the panel as indicated by the five strain gages positioned on the longitudinal laminations at the zero position (Figure 9). The strain profile at the panel edge was linear throughout the full load range. The peak strain value of 648 microstrain was measured in compression at the top of the panel at the first interior position. Assuming a modulus of elasticity of 11.7 GPa for select structural Douglas-fir, the corresponding maximum stress is 7.6 MPa.

For Load Case 2, when the load was equally positioned on each panel, Figure 15 indicates fairly symmetric behaviour of the panels under peak load. The deflection was greatest at the longitudinal joint with a measured value of 21.7 mm. This corresponds to an L/D value of 351. The deflection values uniformly decreased from the longitudinal joint to the panel edges but did not significantly decrease overall. A similar behaviour was observed in the strain values. Peak strain values of 401 and 392 microstrain were measured at the interior positions at the top of the panels corresponding to stress values of 4.7 and 4.6 MPa, respectively. The strain values decreased slightly at the panel edges similar to the behaviour observed in the deflection data. As with Load Case 1, the strain profile as indicated by the strain gage series at the panel edge was fairly linear indicating linear strain behaviour.

For Load Case 3, the load was positioned in the same transverse position as Load Case 1 entirely on Panel 1 but near the endspan. The results presented in Figure 16 indicate many similarities in the overall deflection and strain trends among each of these load cases. The deflection was greatest at the panel edge (20.5 mm) with linearly decreasing values corresponding to the transverse position. The deflection on the opposing panel edge was nearly 0 mm. The strain values at midspan are understandably less than the measured values when the load was positioned at midspan. However, it should be noted that the pattern of relative strain magnitudes when comparing adjacent strain locations is similar between Load Cases 1 and 3. The peak strain value is observed at the first interior strain position. The strain profile observed at the panel edge grouping of strain gages further indicates linear behaviour.

For Load Case 4, the load was positioned symmetrically on each panel as with Load Case 2 but near the endspan. The results presented in Figure 17 indicate very similar deflection and strain behaviour as that in Load Case 2, albeit smaller magnitudes due to the longitudinal load position. The deflection was greatest at the longitudinal joint (11.5 mm) and gradually and minimally decreased with distance from the joint toward the panel edge (9.1 to 9.5 mm). Strain magnitudes were also nearly symmetric with respect to the longitudinal joint. Furthermore, the strain profile at the panel edge indicated linear behaviour.

3.2 EQUIVALENT STRIP WIDTH

Load distribution characteristics for each load case can be quantified by calculating the equivalent strip width (ESW). Calculation of the ESW using measured data is typically achieved by 1) numerically integrating the area under the moment distribution curve, and 2) dividing the summation by the estimated maximum moment. However, when assuming uniform stiffness of the panels, deflection data can be used instead (Equation 1).

$$ESW = \frac{\sum_{i=1}^{n}(deflection_i \times d_i)}{deflection_{max}} \tag{1}$$

where, n is the total number of deflection sensors, $deflection_i$ is the deflection reading of the i-th sensor, $deflection_{max}$ is the maximum deflection measured by the sensors, and d_i is the spacing of adjacent deflection gages. The ESW results for each load case are listed in Table 1.

***Table 1:** Equivalent Strip Width*

Load Case	ESW (m)
1	2.48
2	4.30
3	2.50
4	4.40

It is evident that when the load is positioned on both panels rather than on one panel only, the ESW is approximately 70% greater indicating that a greater width of the bridge is engaged in the load resistance. The transverse load distribution is significantly less when the load is entirely on one panel despite the uniformity of the deflection trend. A stiffer spreader beam may aid the transverse load distribution characteristics. As a means of comparison, ESW values calculated using deflection data from recently completed load tests of in-service, longitudinal nail-laminated bridge decks range from 1.8 to 4.2 m [8]. It can be observed that the ESW values calculated from the CLT tests and nail-laminated bridge decks, which traditionally have one-way action, are similar.

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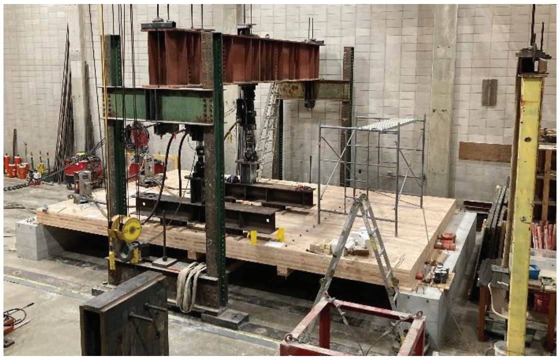

Figure 10 *Load Case 1*

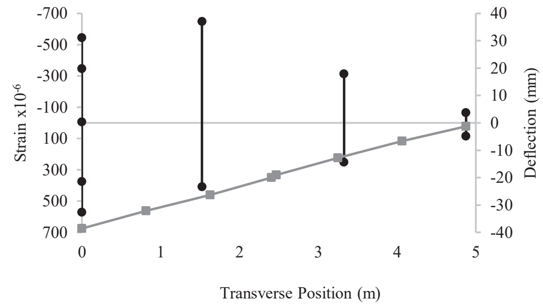

Figure 14 *Strain and Deflection at Max Load for Load Case 1*

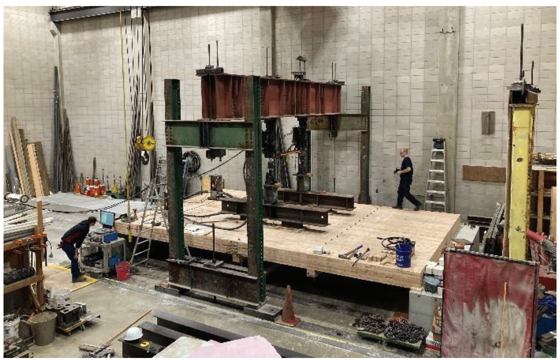

Figure 11 *Load Case 2*

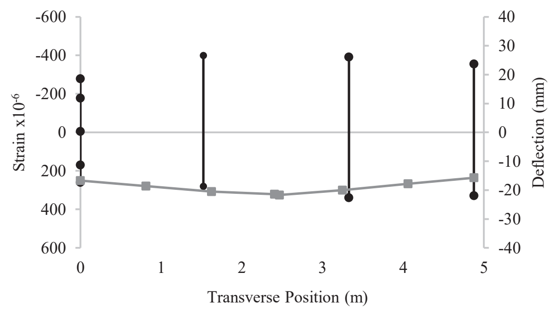

Figure 15 *Strain and Deflection at Max Load for Load Case 2*

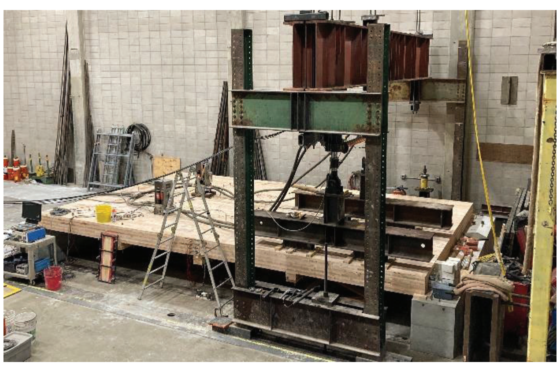

Figure 12 *Load Case 3*

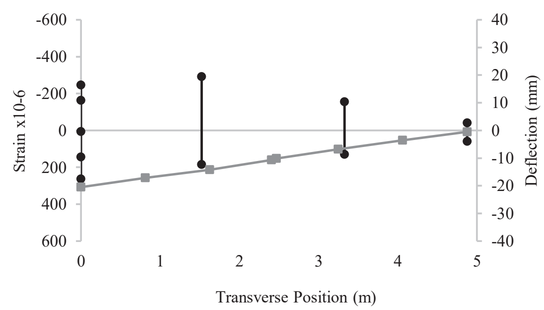

Figure 16 *Strain and Deflection at Max Load for Load Case 3*

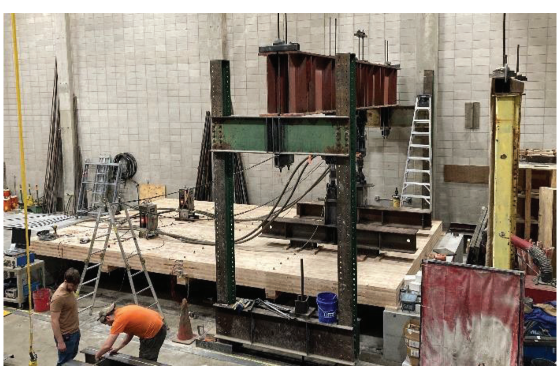

Figure 13 *Load Case 4*

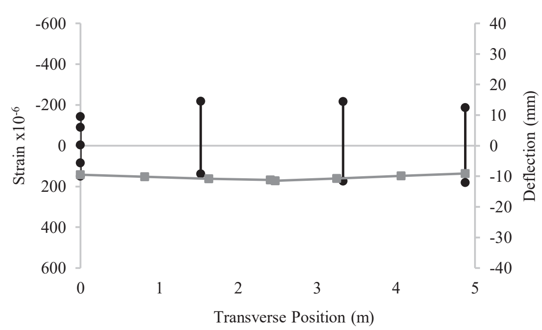

Figure 17 *Strain and Deflection at Max Load for Load Case 4*

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4 SUMMARY AND CONCLUSIONS

Cross-laminated timber panels have gained popularity in recent history and have been widely used in the vertical construction industry. Despite this fact, there have been few instances where CLT panels have been used for bridge deck applications in North America. The impediments to using them are not dissimilar to the use of other timber structures in exterior environments. The ability to pressure-treat full panels is limited by current manufacturer capabilities. Furthermore, pressure-treating individual members to be used in a final CLT configuration is disadvantaged by the overall cost. Further research is recommended to identify best options to protect CLT panels from outdoor elements.

CLT has been used in ways that maximize many of its inherent features. CLT is prefabricated, relatively lightweight, dimensionally stable, and environmentally sustainable. North American design resources and standards have been developed for CLT in the form of PRG 320 [1], CLT Handbook [2], and the National Design Specification (NDS) [3], but these effectively limit the use of CLT to indoor applications where the moisture content can be maintained below 16%. The use of CLT in bridge structures has been limited, and the adoption of CLT into governing design codes has been slow-going.

During this study, two nontreated, longitudinal CLT panels were subjected to highway-type service loads to characterize their structural performance and behaviour. The panels were made up of two grades of Douglas-fir, select structural for longitudinal layers and No. 2 for transverse layers, and were connected together using a shiplap joint, screws, and spreader beams. The simply supported, single-span panels were equipped with deflection and strain gages placed at various locations on the structure to quantify the performance under load. Four load cases were used to place the loads at distinctive locations at midspan and near the end span. In two cases, the entirety of the load was applied to only one panel, whereas in the other two cases the load was equally applied to both panels.

The equivalent strip width (ESW) was calculated for each load case. It was observed that when a single panel was loaded (Load Cases 1 and 3), the ESW decreased by about 40% when compared with the ESW when both panels were simultaneously loaded (Load Cases 2 and 4). This phenomenon is similar to the ESWs observed during in-service, slab-type timber bridge tests previously completed by this research team. Improvements to the transverse load distribution characteristics of longitudinal CLT panel bridges decks would benefit the design and rating of these bridges.

Because of their structural characteristics, CLT panels lend themselves well to being used for highway bridge structures. The data prove their performance to be uniform and predictable. Strain profiles through the depth of the panel indicate the panel remains linear through the full load range. The data also prove the performance to be within the prescribed guidance by the AASHTO Load and Resistance Factor Design [6] for timber bridge structures with the exception of the recommended deflection limits of L/425. This recommended limit is to aid the bridge deck serviceability, not to set limits for structural adequacy. The performance of overlay wearing surfaces are most sensitive to higher deflection magnitudes, which can result in maintenance costs for the bridge owner. An increase in the panel depth, improved transverse load distribution, or shorter span lengths can simply reduce the panel deflection.

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