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QUASI-STATIC OUT-OF-PLANE TESTING OF REINFORCED CROSS-LAMINATED TIMBER

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ABSTRACT:

There is an increasing desire to use wood products, which are environmentally sustainable, in protective design. Although cross-laminated timber (CLT) panels can be designed to mitigate blast threats, softwood CLT needs some form of reinforcement to defeat typical design basis ballistic threats. The inclusion of thin steel plates within a CLT panel's layup, which has previously been shown to be feasible from both a cost and ballistic performance perspective at small scale, could conceivably be used to transform CLT into both a blast and ballistic resistant panel. To evaluate the response of such a reinforced CLT (RCLT) panel at full scale, an analytical methodology was developed to predict the performance of RCLT panels to out-of-plane loading. The methodology was used to develop three unique RCLT layups that varied both the lumber grade and ply orientation to achieve targeted failure mechanisms. Twelve RCLT panels were fabricated and subjected to quasi-static four-point bending. Tested specimens showed high levels of ductility, excellent wood-to-steel adhesion, and significant post-peak load carrying capacity. Dynamic analyses indicate that the 7-ply RCLT panels perform better than unreinforced 7-ply and 9-ply CLT panels.

KEYWORDS: Cross-Laminated Timber, CLT, Reinforced CLT, RCLT, Enhanced CLT, Protective Design, Blast

1 INTRODUCTION

Buildings used by the U.S. Department of State (DOS) and U.S. Department of Defense (DOD) often must meet blast, ballistic resistance, and forced entry (FE) design requirements to mitigate the hazardous effects associated with terrorism [1]. Historically, DOS and DOD buildings exposed to these threats have been constructed using concrete and steel. A significant amount of testing has been performed to demonstrate the ability of these building materials to resist blast, ballistic, and FE threats. A relatively smaller number of tests have been performed on wood construction for similar threats. At least part of this stems from the relative difficulty of designing light-frame wood construction to resist these threats efficiently and economically. However, the emergence of mass timber construction, particularly cross-laminated timber (CLT), presents a sustainable, modular, and cost-effective alternative building material for high-security infrastructure. Recent blast testing of cross-laminated timber [2] has informed the development of design guidance in the United States [3]. Additional ballistic testing has shown the feasibility of incorporating thin steel plates within a CLT panel's layup to defeat typical design basis ballistic threats [4]. It is hypothesized that by detailing the ballistic reinforcing layers to act compositely with the wooden CLT panel, this reinforced CLT (RCLT) panel may also demonstrate improved blast performance.

2 OBJECTIVE

An analytical method was developed to estimate the resistance function (i.e., the out-of-plane (flatwise) load versus displacement relationship) of RCLT panels with arbitrary (user defined) layups. This methodology was used to select three distinct RCLT layups that were fabricated by SmartLam North America and subsequently tested under quasi-static out-of-plane loading at the University of Maine. The objectives of this effort were to (a) develop a viable methodology to predict the performance of RCLT panels to out-of-plane loading, (b) generate test data to validate the methodology, and (c) demonstrate the post-peak behavior of RCLT panels.

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3 ANALYTICAL METHOD

An analysis methodology was developed to predict the out-of-plane bending resistance of RCLT panel designs. The methodology is similar to the shear analogy method used for standard CLT panels [3],[5],[6], but the relevant equations are modified to permit the inclusion of non-wood (e.g., steel) laminations. As a verification effort, the methodology was used to estimate the ASD design values for several standard CLT panels [7]. The comparison showed good agreement for most of the design values, with the methodology predicting an approximately 9% higher shear strength than published values. This difference was found to stem from different shear capacity equations used in the methodology [6] and by ANSI/APA PRG-320 [8]. The PRG-320 shear equation implicitly assumes a uniform wood cross section, resulting in the well-known equation for peak shear stress in a rectangular section, shown in Equation (1):

$$\tau = 1.5\frac{V}{A} \tag{1}$$

In contrast, the developed methodology based on the shear analogy method uses a shear flow calculation to estimate the shear capacity of the CLT/RCLT panels. Due to the alternating, orthogonal wood layers the shear stress through the panel thickness is non-parabolic, contrary to that assumed by Equation (1). The effect is that the shear flow calculation estimates a slightly lower peak shear stress demand in the panel for an applied unit shear, resulting in a larger estimate of the panel's shear force capacity. An analogy to the design of steel W-shapes is appropriate. The flanges in a steel W-shape drive the shear stress distribution in the web to be nearly uniform. Similarly, although to a lesser degree, the strongly oriented outer wood layers (in a CLT panel) and steel inserts (in an RCLT panel) result in a more uniform shear stress distribution through the panel thickness (relative to the parabolic shear stress distribution assumed in PRG-320), leading to a lower peak shear stress in the panel, and a larger shear force capacity of the panel.

4 TEST SPECIMENS

4.1 LAYUP SELECTION

The analysis methodology was used to select RCLT panel layups for quasi-static testing. Several dozen wood species and layups were considered. In each case the steel layers were placed near the exterior of the panel to maximize their contribution to bending strength and their effect on redistributing the shear stress through the panel. The steel layers were ASTM A1011 sheets with a specified minimum yield strength of 250 MPa. Based on the calculated capacity estimates and economy of material, three wood species/grades were selected: Spruce-Pine-Fir South (SPF-S) No.2, Southern Pine (SP) No.2, and 2400F-2.0E Southern Pine MSR (SP MSR). These three wood grades were oriented in three specific layups to target desired failure limit states.

Layup number 1 (Table 1) utilized SPF-S No.2 in each wood layer and acted as a baseline case (note that using SPF-S No.2 in an unreinforced CLT panel is a standard 'V4' grade CLT panel). SPF-S No.2 has a relatively low density, hardness, and toughness when compared to other wood species and thus served as a reasonable estimate for the lower bound case. The analysis methodology predicted layup number 1 would fail in rolling shear.

Layup number 2 (Table 2) utilized SP No.2 in each 0-degree layer (the major strength direction) and SP MSR in each 90-degree layer (the minor strength direction). SP provides a higher rolling shear capacity then SPF-S, which increases the overall shear capacity of the panel. The analysis methodology predicted the steel layers would yield nearly simultaneously with rupture of the outer wood layers.

Layup number 3 (Table 3) is similar to layup number 2, except the outer wood layers (wood plies 1 and 7) are oriented in the 90-degree (minor) direction. Changing the orientation of the outer layers only slightly changes the shear capacity of the panel but greatly reduces the panel's flexural capacity. Thus, the intent was a panel in which the steel plates would yield prior to either a wood flexural rupture or rolling shear failure. The analysis methodology predicted the steel layers would yield first, followed by rupture of the inner wood core.

Analogous unreinforced CLT panels (i.e., baseline cases) were not tested due to cost considerations.

4.2 SPECIMEN FABRICATION

For each of the three layups, four replicate specimens were fabricated by SmartLam North America (twelve specimens total). The fabrication process started by sandblasting the steel plates to remove oils and mill scale. The lumber was acclimated and processed according to normal CLT operations and PRG-320 requirements. Steel plates were placed by hand during the pressing operation to facilitate correct alignment. Adhesive, similar to that used for normal CLT production, was used. All specimens were approximately 1.22 m wide x 4.27 m long x 250 mm thick. Figure 1 shows the installation of the steel plates during the layup process. Figure 2 shows some of the finished panels.

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Table 1: *Layup number 1*

Grade	Orientation	Thickness [mm]
SPF-S No.2	0-deg	35
A1011 Gr.36 Steel	-	3.4
SPF-S No.2	90-deg	35
SPF-S No.2	0-deg	35
SPF-S No.2	90-deg	35
SPF-S No.2	0-deg	35
SPF-S No.2	90-deg	35
A1011 Gr.36 Steel	-	3.4
SPF-S No.2	0-deg	35

25.4mm = 1 inch

Table 2: *Layup number 2*

Grade	Orientation	Thickness [mm]
SP No.2	0-deg	35
A1011 Gr.36 Steel	-	3.4
2400F-2.0 SP MSR	90-deg	35
SP No.2	0-deg	35
2400F-2.0 SP MSR	90-deg	35
SP No.2	0-deg	35
2400F-2.0 SP MSR	90-deg	35
A1011 Gr.36 Steel	-	3.4
SP No.2	0-deg	35

25.4mm = 1 inch

Table 3: *Layup number 3*

Grade	Orientation	Thickness [mm]
SP No.2	90-deg	35
A1011 Gr.36 Steel	-	3.4
2400F-2.0 SP MSR	90-deg	35
SP No.2	0-deg	35
2400F-2.0 SP MSR	90-deg	35
SP No.2	0-deg	35
2400F-2.0 SP MSR	90-deg	35
A1011 Gr.36 Steel	-	3.4
SP No.2	90-deg	35

25.4mm = 1 inch

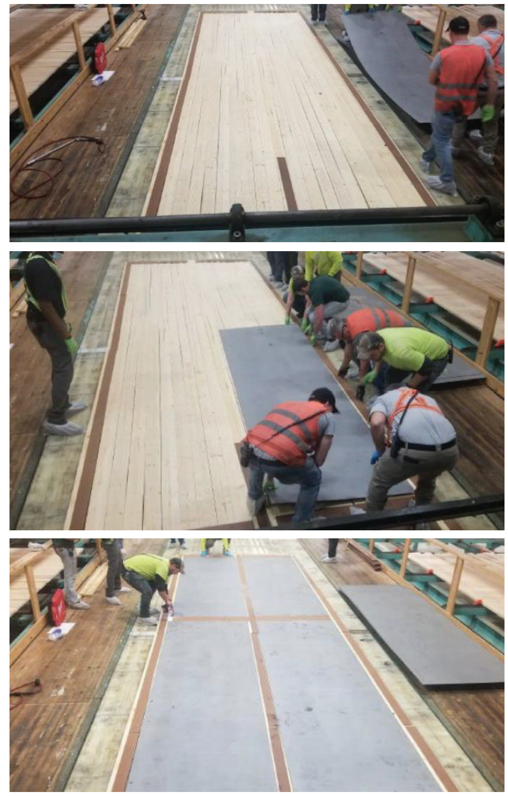

Figure 1: *RCLT steel plate installation*

Figure 2: *Finished RCLT panels*

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5 TEST CONFIGURATION

The twelve RCLT specimens were tested at the University of Maine under quasi-static four-point bending. A schematic of the test setup is shown in Figure 3 with Figure 4 showing a corresponding isometric view of the actual test setup. For each test, a 1300-kN actuator loaded a steel spreader/cross I-beam which then loaded two steel W-shapes running orthogonal to the span length. The cross beam-to-W-shape connection was detailed to allow rotation of the W-shapes. The RCLT panels were supported on hollow structural sections (HSS) across the full width of the panels. The HSS sections were supported on concrete supports and were permitted to rotate. Between the W-shape-to-RCLT panel and RCLT panel-to-HSS section were lubricated sheets of high-density polyethylene (HDPE) to reduce frictional constraint of the RCLT panel. The test span length was 3.66 m. Three displacement gages recorded the midspan displacement while four video cameras oriented at the four panel corners (i.e., North-West, North-East, South-West, South-East) recorded the testing. The applied load was recorded via a calibrated load cell at the actuator. Testing was performed well beyond the maximum load carrying capacity of the panel.

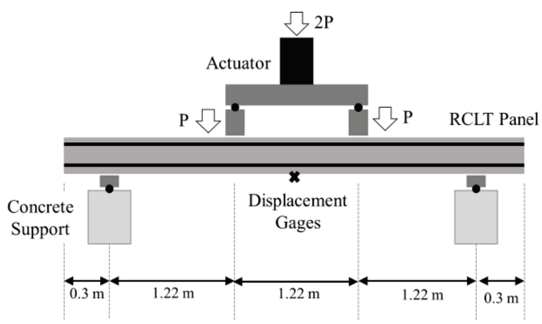

Figure 3: *Schematic test configuration (elevation)*

Figure 4: *Test configuration*

6 TEST RESULTS

6.1 LAYUP NUMBER 1

Figure 5 shows the force-displacement curves of the four specimens. On average, the layup displayed a rolling shear failure near 465 kN with a corresponding deflection of 25 mm. There is a sharp drop in load carrying capacity after the rolling shear failure, but a significant residual capacity remains. This residual capacity is attributed to the two steel plates, each acting compositely with the immediately adjacent (intact) wood layers, and quasi-compositely with the other steel plate. This adds ductility to the system, even after a rolling shear failure has occurred. Figure 6 shows specimen 1 under various levels of loading. The images correspond to (A) the specimen just after a rolling shear failure, (B) midspan flexural (tension) rupture of the wood, and (C) the post-test state of the panel.

6.2 LAYUP NUMBER 2

Figure 7 shows the force-displacement curves of the four specimens. On average, the layup first displayed a flexural wood rupture near 750 kN with a corresponding deflection of 35 mm, and later, a rolling shear failure near 76 mm of displacement. Figure 8 shows specimen 1 under various levels of loading. The images correspond to (A) the onset of wood flexural rupture, (B) a rolling shear failure, and (C) the post-test state of the panel.

As seen in Figure 7, specimen 2 displayed significant load carrying capacity through 250 mm of displacement. Unlike the other three specimens, specimen 2 did not display a rolling shear failure. Instead, after the outer wood ply on the underside of the panel ruptured, the core of the panel displayed an adhesive bond failure between the tension steel and wood core, as well as a flexural rupture of the wood core layers near midspan. The significant drop in load carrying capacity (around 250 mm) is due to this bond failure eventually propagating to the support. The force-displacement curve for specimen 2 is an ideal outcome for blast design as the area under the curve represents significant energy dissipation potential. Figure 9 shows the failure sequence from the specimen 2 test.

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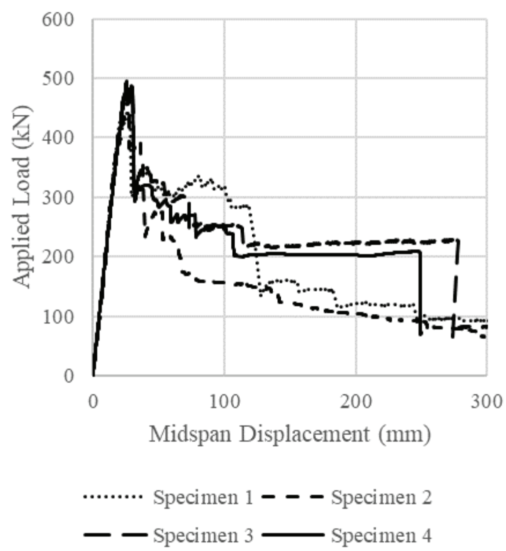

Figure 5: *Layup number 1 force-displacement curves*

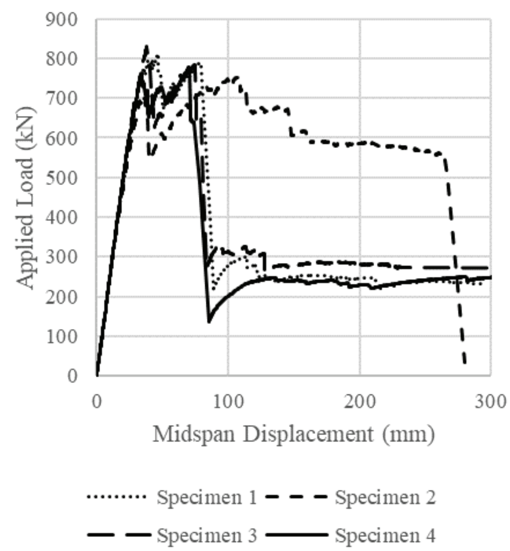

Figure 7: *Layup number 2 force-displacement curves*

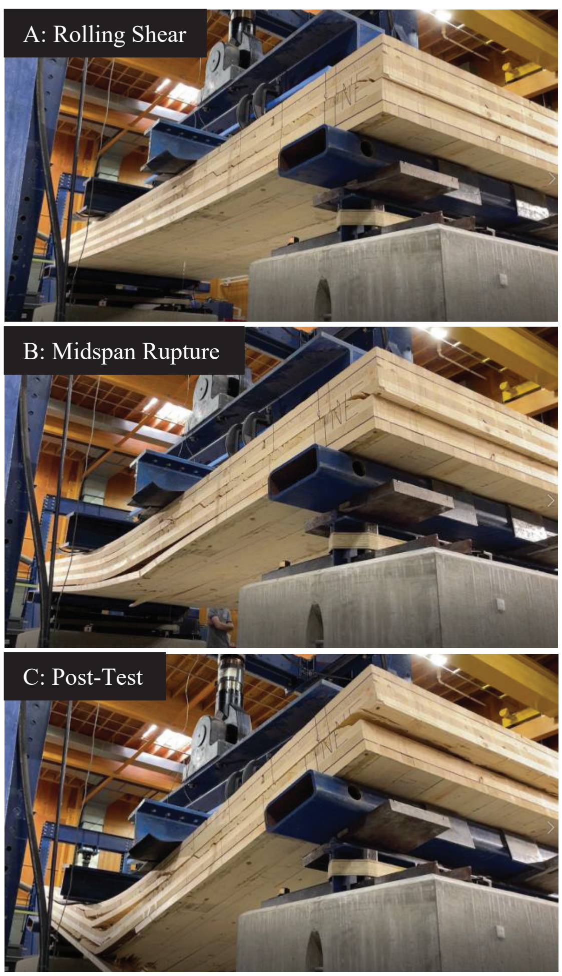

Figure 6: *Layup number 1, specimen 1, under loading*

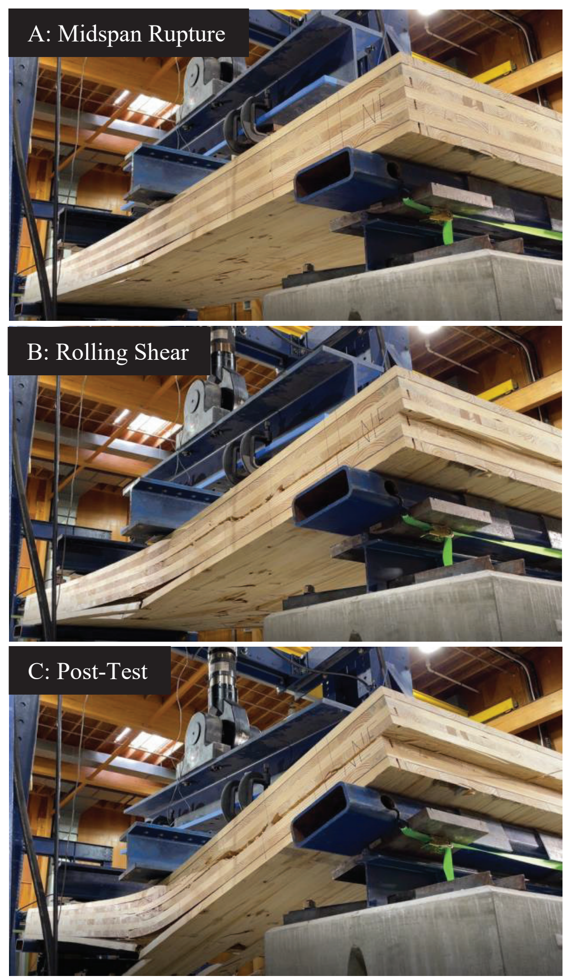

Figure 8: *Layup number 2, specimen 1, under loading*

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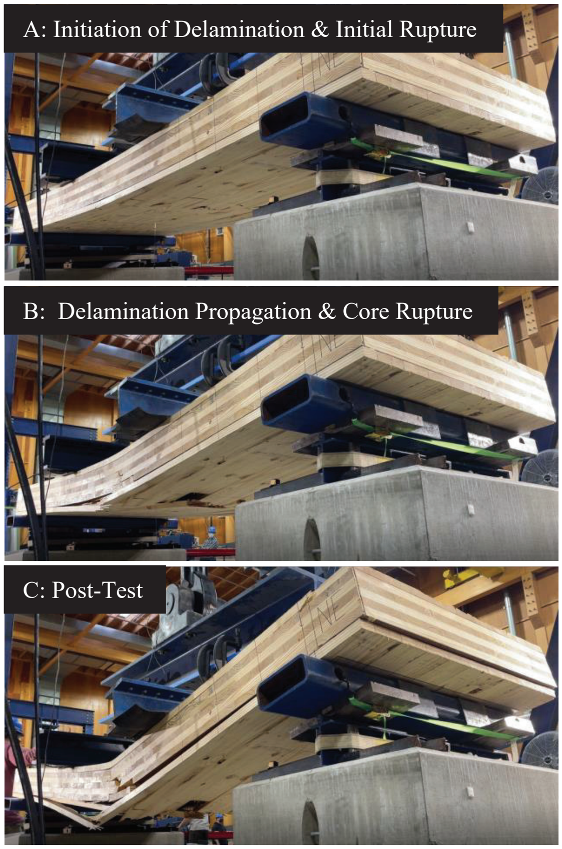

Figure 9: *Layup number 2, specimen 2, under loading*

6.3 LAYUP NUMBER 3

Figure 10 shows the force-displacement curves of the four specimens. On average, the layup displayed steel yielding behavior near 320 kN (23 mm corresponding displacement) and a rolling shear failure near 560 kN and 94 mm of displacement. Figure 11 shows specimen 1 under various levels of loading. The images correspond to (A) the specimen just before a rolling shear failure, (B) a rolling shear failure, and (C) the post-test state of the panel.

6.4 COMPARISON TO PRE-TEST ESTIMATES

Figure 12 compares pre-test estimates with average load-displacement curves obtained from testing. In all cases the pre-test initial stiffness underestimated the actual stiffness. The pre-test prediction for layup 1 was a rolling shear failure, which was observed in the actual test. The pre-test prediction for layup 2 was steel yielding followed by flexural rupture of the outer wood ply. The pre-test calculation erroneously assumed failure of the outer wood ply was an ultimate limit state, thus leading to the relatively brittle nature of the predicted force-displacement curve. In the actual test the outer ply ruptured, but significant ductility was observed before the final rolling shear failure was observed (the embedded steel plates carry considerable flexural load). The pre-test prediction for layup 3 was steel yielding followed by flexural rupture of the inner wood core. While the pre-test estimate accurately predicted the steel yielding, the ultimate strength and final limit state were inaccurately predicted. It is noted that the pre-test calculated shear strength was ~590 kN, which agrees with the observed peak shear strength of the panel.

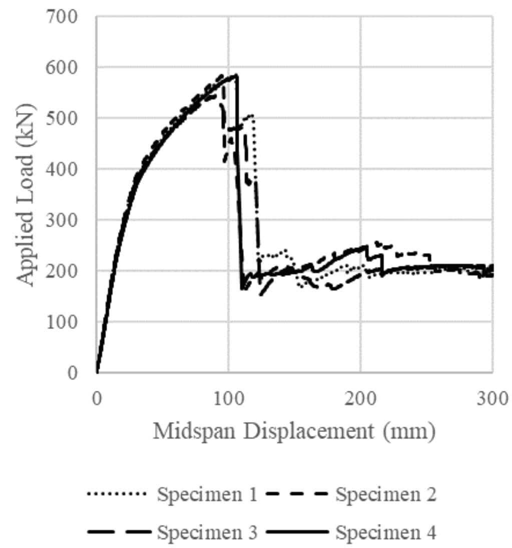

Figure 10: *Layup number 3 force-displacement curves*

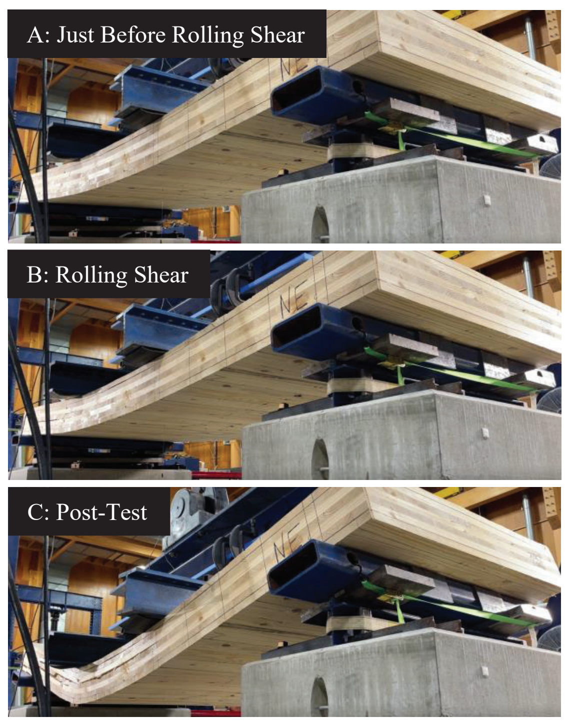

Figure 11: *Layup number 3, specimen 1, under loading*

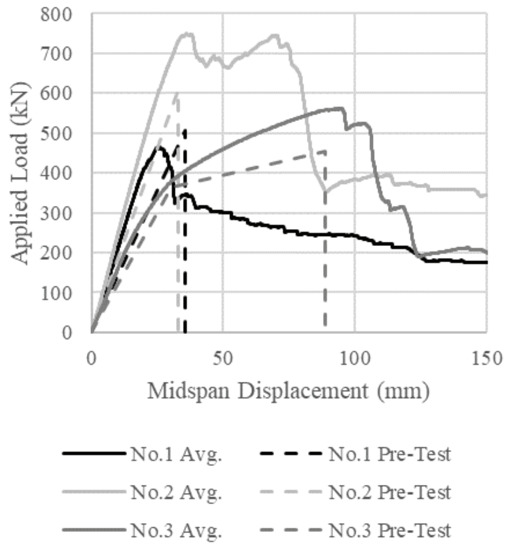

Figure 12: *Average force-displacement curves and pre-test predictions for the three layups*

6.5 POST-TEST ESTIMATES

Following the four-point RCLT panel bending tests, material tensile testing was performed on samples of the steel plates to improve the material properties used in the predictive analysis model. In addition, the analysis methodology was updated to include an incremental calculation approach, which accounts for damaged plies in the flexural calculations. Although still being developed, this updated methodology seems to better capture the evolution of flexural failures as degradation of load carrying capacity due to wood flexural rupture is now being captured, as shown in Figure 13.

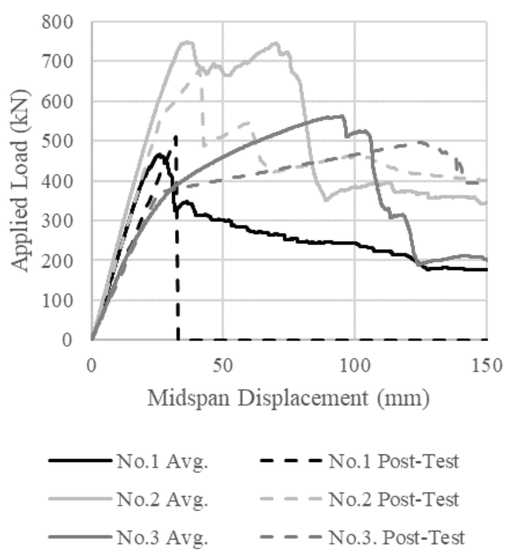

Figure 13: *Average force-displacement curves and updated methodology (post-test) predictions for the three layups*

7 DYNAMIC ANALYSIS

Following the test, calculations were performed to investigate the relative performance of these RCLT layups against unreinforced CLT layups under blast loading. Resistance curves were calculated for standard (unreinforced) CLT panels utilizing SP No.2 in 7-ply and 9-ply layups assuming a 3.66 m simply supported clear span. The layups assumed alternating 0 deg (strong) and 90 deg (weak) layers, with the exterior layers oriented in the 0 deg direction. Resistance functions for the three RCLT panels (layups 1, 2, and 3) were calculated with the updated methodology described in Section 6.5, using high-strain rate dynamic increase factors and assuming a uniform load distribution. The calculated resistance functions for all five panels are shown in Figure 14. The resistance function calculations predict: the two unreinforced CLT panels will display flexural failures near 105 kPa and 155 kPa; layup 1 will fail in shear near 165 kPa; layup 2 will demonstrate steel yielding near 180 kPa, followed by a shear failure near 215 kPa; layup 3 will demonstrate steel yielding near 130 kPa, and rupture of the wood core near 180 kPa. In all five cases the resistance function was set to perfectly plastic at the calculated ultimate resistance, generally following the approach in PDC-TR 18-02 [3].

Pressure-Impulse (P-I) diagrams were generated to compare the relative performance of the five panels. For each panel, a single-degree-of-freedom (SDOF) solver was used to determine the pressure-impulse combination that resulted in a specified ductility demand (i.e., μ = {maximum displacement / yield displacement} = 1, 2, 3, 4). The results are plotted in Figure 15 though Figure 18; note that the scales vary across the four figures. For each P-I curve, P-I combinations to the left or below the curve result in a smaller ductility demand while P-I combinations to the right or above the curve result in a greater ductility demand. Using Figure 15 as an example, the three RCLT panels are expected to sustain larger loads then the 7-ply CLT panel in the elastic range, while layup 2 is expected to sustain larger loads than both the 7-ply and 9-ply panels in the elastic range.

Although useful for comparison purposes, the P-I diagrams fail to account for the ductility capacity of the various panels. PDC-TR 18-02 defines ductility response limits for unreinforced CLT panels where, currently, ductility demands greater than 2 (i.e., $\mu \geq 2$) are expected to result in a 'Blowout' failure of unreinforced panels. On the other hand, Figure 13 indicates a ductility capacity for layup 3 of μ~4. Thus, if we compare the P-I diagrams of these three panels closer to their estimated limits of performance (see Figure 19), we see that layup 3 is expected to outperform both the 7-ply and 9-ply unreinforced CLT panels.

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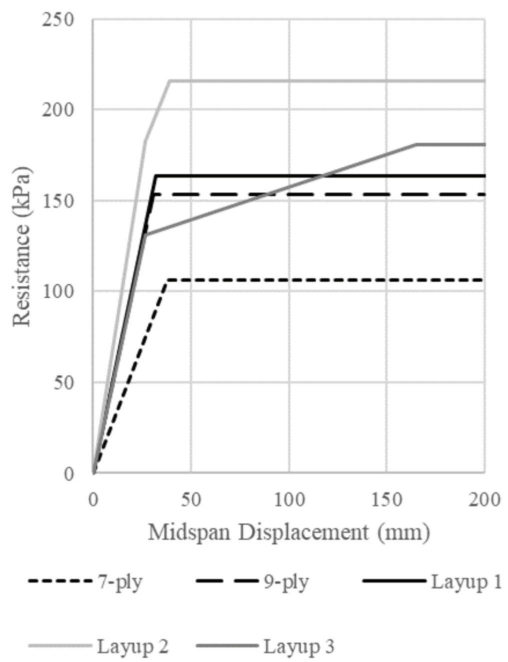

Figure 14: *Resistance functions for dynamic analysis*

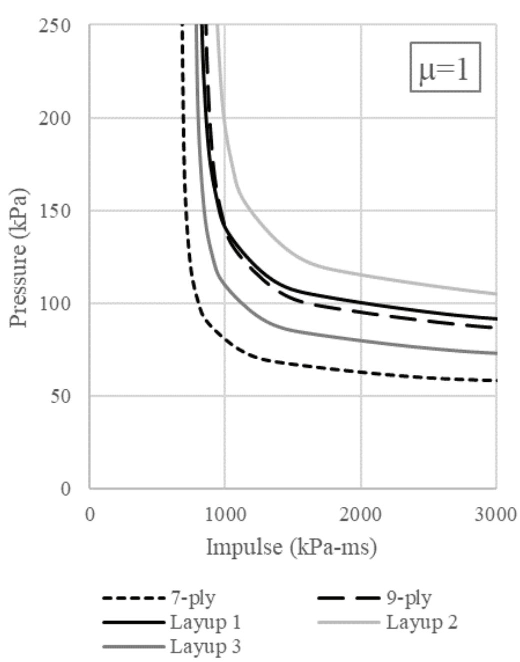

Figure 15: *Pressure-impulse diagram for μ = 1*

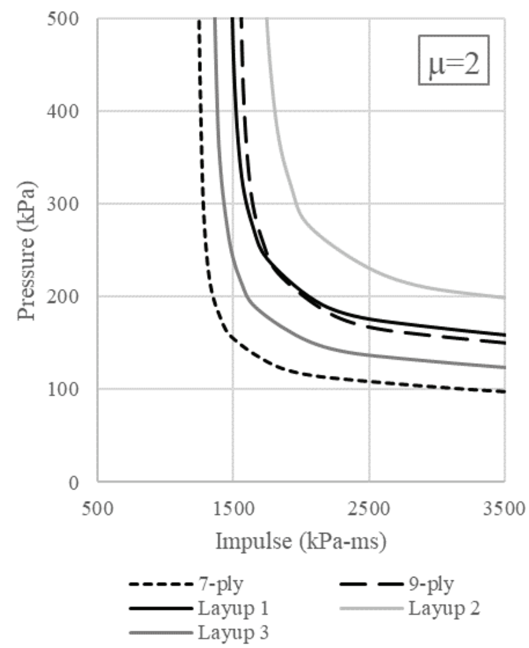

Figure 16: *Pressure-impulse diagram for μ = 2*

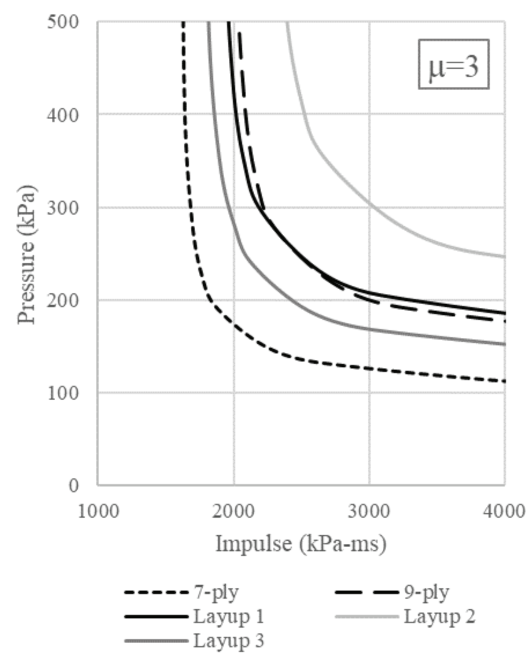

Figure 17: *Pressure-impulse diagram for μ = 3*

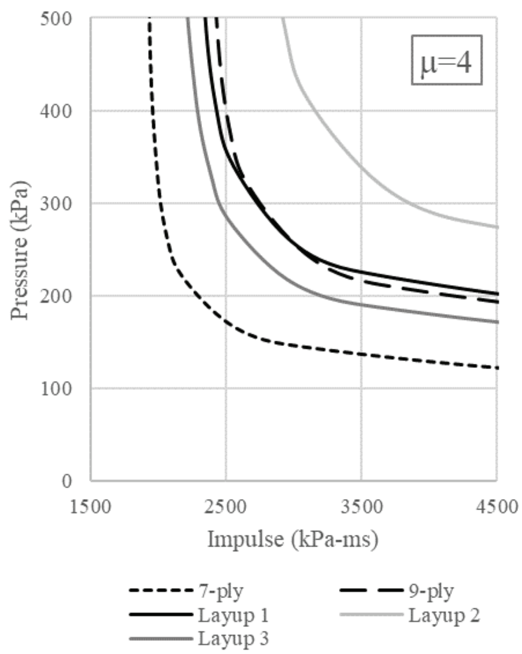

Figure 18: *Pressure-impulse diagram for μ = 4*

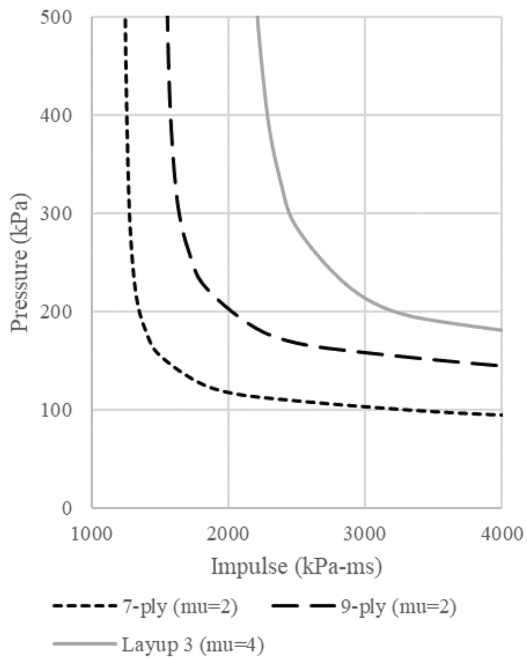

Figure 19: *Pressure-impulse diagram comparison for unreinforced CLT and layup 3 near limits of performance*

8 FUTURE EFFORTS

Several efforts are planned or ongoing to continue advancing the use of RCLT in protective design:

- Methodology Improvement (Ongoing). The methodology developed and utilized in this effort is being improved by implementing an incremental approach to calculating stiffness, flexural and shear capacities accounting for various damage mechanisms. The current version of the revised methodology provides better agreement with the test results presented herein (i.e., compare Figure 12 and Figure 13) and is planned to be included in a stand-alone software which calculates both CLT and RCLT resistance functions, as well as perform SDOF analyses.
- Weathering Study (Ongoing). Preliminary tests have shown the wood-to-steel adhesive bond not to be a limiting failure mechanism in RCLT panels. Both small-scale and large-scale weathering tests are ongoing to evaluate if temperature and moisture cycling degrades the adhesive bond and reduces the capacity of RCLT panels.
- Arena Blast Testing (Planned). Three RCLT layups will be subjected to arena blast testing. Two specimens will be fabricated for each layup, with one specimen subjected to six months of outdoor weathering prior to blast loading. Testing is scheduled for 2023.

9 CONCLUSIONS

This paper describes the development of a methodology to predict the performance of reinforced CLT (RCLT) panels, the manufacturing of full-scale RCLT panels, and quasi-static bending tests on those panels. The following general conclusions can be made from this work:

- The performance of RCLT panels can be well predicted. The methodology developed within this effort reasonably predicted the behavior of the RCLT panels selected. Future efforts include improving the methodology based on available test data.
- Full scale RCLT panels can be manufactured at an active CLT manufacturing facility. The cost to fabricate the RCLT panels is competitive with other protective solutions and would reasonably be expected to come down with wide-spread adoption of RCLT (e.g., automated vs. manual placement of the steel).
- Adhesive bond failure between the wood and steel layers was not typically observed, indicating that premature delamination failures at the embedded steel/lumber interface is not likely to compromise the out-of-plane resistance. Accelerated laboratory and short-term field weathering studies are currently ongoing to further evaluate the wood-to-steel adhesive bond.

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- Rolling shear or delamination are the likely ultimate limit states for RCLT, although delamination was not typically observed. Because of the continuity, ductility, and capacity of the embedded steel plates, wood flexural rupture does not appear to cause a catastrophic panel failure. Further, even after a rolling shear failure RCLT demonstrates significant residual load carrying capacity.

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