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WOOD AND FIBER SCIENCE

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A multiplex qPCR assay for the identification of *Pinus palustris* and *Pinus elliottii*

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Abstract: Differentiating the wood of high-strength southern yellow pines (*Pinus palustris* and *P. elliottii*) from their lower-strength congeners (*P. taeda* and *P. echinata*) presents a substantial challenge once diagnostic external features, such as needles, cones, and bark are absent. This difficulty arises from their wood anatomical indistinguishability and low genetic resolution driven by highly conserved genome organization and extensive allele sharing. To address this, we developed and validated a multiplex qPCR assay utilizing SYBR Green chemistry and asymmetric primer concentrations, working with leaf and seed DNA. The assay targets short amplicons (<100 bp) to maximize the likelihood of amplification of the fragmented DNA typical of solid wood. The method achieved high linearity ($R^2 > 0.99$) and efficiency, successfully co-amplifying targets while maintaining clear discrimination through distinct mean melting temperatures (77.98°C [SD 0.11] and 69.25°C [SD 0.26], respectively). Sensitivity testing established limits of detection at 1 pg for *Pinus palustris* and 10 pg for *Pinus elliottii*. Validation across a panel of 84 individuals representing all four species yielded 100% accuracy, precision, and recall. This study establishes a rapid, gel-free molecular diagnostic tool that overcomes the limitations of wood anatomical identification. By validating a short-amplicon design on leaf and seed tissues, it provides a proof-of-concept for future applications on solid wood and offers a reproducible methodological framework for species verification.

Keywords: Multiplex qPCR; SYBR green; Southern yellow pine; Species identification.

Introduction

The commercial southern yellow pine (SYP) species group includes loblolly (*Pinus taeda*), shortleaf (*P. echinata*), longleaf (*P. palustris*), and slash (*P. elliottii*) pine. This group plays a central role in the United States' forest economy and is widely

distributed throughout the southeastern region (Koch 1972; Gaby 1985; Barnett 2019). Although these species are often marketed collectively, this practice masks species-specific variations in wood properties. For example, longleaf and slash pine typically have higher mechanical properties, such as modulus of elasticity (MOE) and modulus of rupture (MOR), and wood density compared to shortleaf and loblolly pine (Senalik and Farber 2021) (Table 1). These relative rankings can vary

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Table 1. Wood specific gravity and mechanical properties of southern yellow pine (SYP) species at 12% moisture content (Senalik and Farber 2021).

Species	SG	MOR (kPa)	MOE (MPa)
<i>P. elliotii</i> (Slash)	0.59	112,000	13,700
<i>P. palustris</i> (Longleaf)	0.59	100,000	13,700
<i>P. echinata</i> (Shortleaf)	0.51	90,000	12,100
<i>P. taeda</i> (Loblolly)	0.51	88,000	12,300

SG specific gravity, MOR modulus of rupture (kPa), and MOE modulus of elasticity (MPa)

among studies not only because different mechanical properties are being evaluated (e.g., SG, MOR, MOE), but also due to biological, environmental, and geographic factors. These differences are consistent with their phylogenetic relationships; recent analyses indicate that *P. elliotii* and *P. palustris* form a sister-species pair that is distinct from the pair formed by *P. taeda* and *P. echinata* (Jin et al. 2021). While species such as sand pine (*P. clausa*) and Virginia pine (*P. virginiana*) occur regionally, they are minor contributors to commercial lumber streams and were not included in this study (Gaby 1985; Senalik and Farber 2021).

Species-specific differences in mechanical properties, in conjunction with other interacting factors, are especially relevant in high-value structural markets and grading systems, such as Machine-Stress-Rated (MSR) lumber, where specific minimum strength and stiffness values are required (Galligan et al. 1977; Western Wood Products Association 2008; Ross 2015). SYP MSR lumber represents a substantial market sector: during the strong construction cycle of 2005, SYP contributed 347.1 million board feet of the more than 2 billion board feet of MSR lumber produced; by 2018, SYP MSR production increased to 568.3 million board feet, making it the second largest MSR species group after Spruce-Pine-Fir (Entsminger et al. 2020). Market trends reflect increasing adoption of machine grading systems, with MSR usage among mills in the US South rising from fewer than 5% in 2004 to nearly 30% by 2020 (Entsminger et al. 2020).

The economic importance of MSR lumber, combined with species-linked differences in mechanical performance, creates strong incentives for mills to verify the species composition of the logs they purchase. In cases where mills are willing to pay a premium for higher strength species such as slash or longleaf pine, unexpectedly low MSR yields might indicate that the delivered logs were perhaps not (or not entirely) the species ordered. Species verification provides a means for mills to assess whether the raw material aligns with purchase

agreements and expected product performance. A similar need arises in purchases of gatewood—logs not previously under supply contract purchased from independent logging firms upon delivery to the mill—where rapid screening of logs can help mills ensure they are procuring their desired species. These dynamics parallel broader trends in the forest products sector, where both landowners and mills increasingly engage in species- or quality-based valuation. Recent economic analyses show that landowners require price premiums to justify management intended to produce higher-quality pine sawtimber (Regmi et al. 2022) and that mills themselves are willing to pay premiums for higher-quality material (Regmi et al. 2022). In this context, a DNA-based identification tool offers a practical means of verifying species composition from suppliers, supporting adequate valuation, resolving disputes, and enhancing transparency in SYP procurement.

Identifying southern yellow pine species in standing trees relies on morphological characteristics such as needles, cones, and bark patterns, but most of these diagnostic features are lost once a tree is harvested. Species identification then depends on wood anatomical features, which can distinguish major sub-generic groups of *Pinus*, including the white, red, and yellow pine groups (Kukachka 1960; Panshin and Zeeuw 1980), but not individual species within the southern yellow pine group. The anatomical similarity among *P. elliotii*, *P. palustris*, *P. echinata*, and *P. taeda* has long been recognized (Kukachka 1960; Howard and Manwiller 1969; Eberhardt et al. 2022) (Figure 1) and reflects their close phylogenetic relationships (Gernandt et al. 2005; Jin et al. 2021; Xia et al. 2023).

The challenge of differentiating these genetically distinct but anatomically indistinguishable groups has prompted research into molecular tools that target subtle genetic differences. For instance, Olatinwo et al. (2020) developed a conventional polymerase chain reaction (PCR) method using a series of cpDNA markers to distinguish the four major SYP species specifically in the context of seed verification for reforestation efforts. Their method is unlikely to be suitable for our needs as conventional PCR depends on the successful amplification of relatively long DNA fragments, which is poorly suited for fragmented DNA typical of solid wood (Rachmayanti et al. 2009; Jiao et al. 2012, 2014; Höltnen et al. 2012), and we intend to have a field-deployable test not dependent on gel electrophoresis.

In contrast, quantitative polymerase chain reaction (qPCR) is particularly advantageous for analyzing fragmented DNA because it targets short amplicons and utilizes fluorescence-

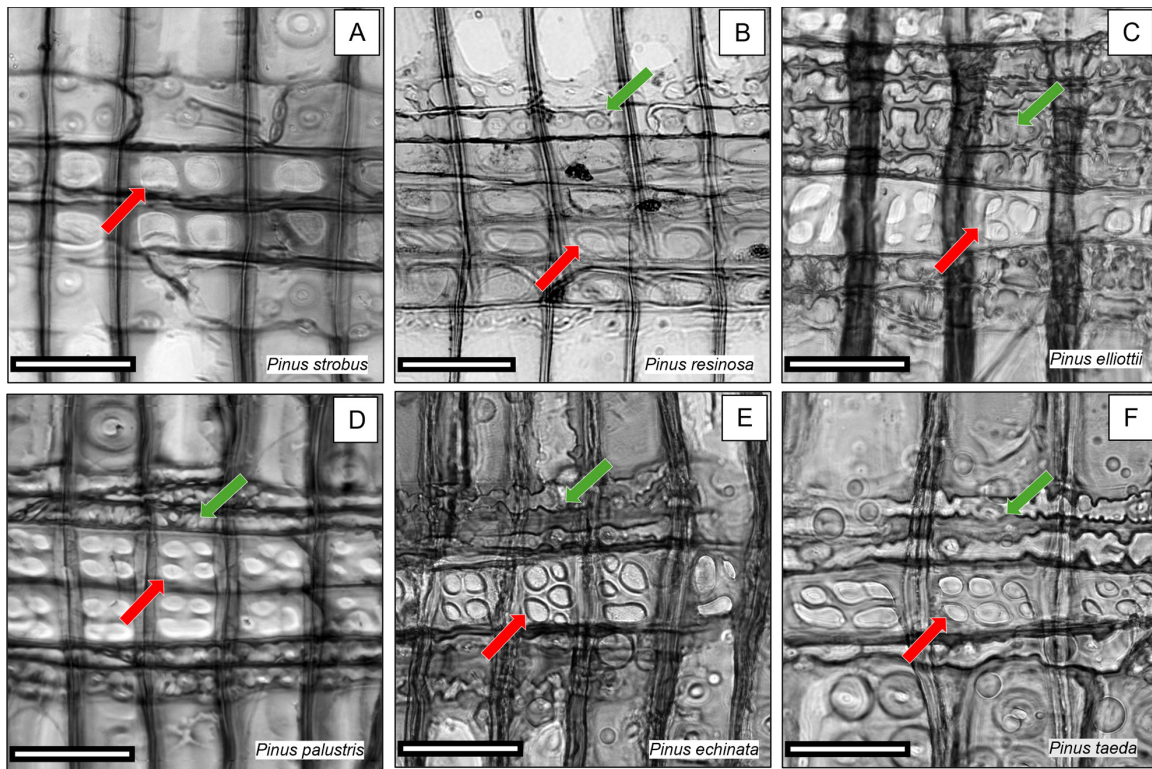


Figure 1. Radial wood anatomy comparison across six *Pinus* species. (A) White Pine Group (*P. strobus*): lacks dentate ray tracheids, with fenestriform cross-field pits; (B) Red Pine Group (*P. resinosa*): has dentate ray tracheids, with fenestriform cross-field pits; (C–F) Southern Yellow Pine species (*P. elliottii*, *P. palustris*, *P. echinata*, *P. taeda*): all show dentate ray tracheids and pinoid cross-field pits, illustrating the anatomical similarity within the group that precludes species-level identification. Green arrows point to ray tracheids; red arrows point to cross-field pitting; scale bar=50μm.

based detection to bypass the limitations of agarose gels. Fluorescence monitoring offers significantly higher sensitivity than gel electrophoresis; studies on processed wood have demonstrated that samples yielding no observable bands on a gel due to low copy number can still be reliably detected and identified through real-time fluorescence thresholds (Mohamed et al. 2012). Furthermore, this method allows for species authentication via melt curve rather than amplicon size, enabling accurate identification even with minimal starting material (Lee et al. 2016). qPCR has successfully been used to amplify DNA from ancient wood, which is a source of highly fragmented DNA (Watanabe et al. 2015; Gómez-Zeledón et al. 2017), and it is widely applied in forensic species identification and environmental DNA monitoring (Davy et al. 2015; Borland and Kading 2021). Given these advantages, qPCR represents a particularly suitable platform for developing species-differentiation assays in southern yellow pine wood. In this study, multiplex SYBR® Green chemistry (a proprietary nucleic acid stain; Thermo Fischer Scientific, Waltham, MA, USA) was selected for its cost-effectiveness compared to probe-based methods, efficiency, and compatibility with melt curve analysis, which

enables discrimination among closely related taxa without the need for post-PCR gel electrophoresis (Ponchel et al. 2003; Tajadini et al. 2014; Zhou et al. 2017; Naeem et al. 2024).

The objective of this study is to develop a qPCR-based multiplex assay capable of differentiating *Pinus elliottii* (slash pine) and *Pinus palustris* (longleaf pine) from one another and from the lower-strength species *Pinus echinata* (shortleaf pine) and *Pinus taeda* (loblolly pine). To demonstrate proof-of-concept, we validate this assay using DNA extracted from leaf and seed tissues, laying the groundwork for extension of this method to wood and wood products.

Materials and methods

Plant material and DNA extraction

Plant tissue was collected from four southern yellow pine species (*P. palustris*, *P. elliottii*, *P. echinata*, and *P. taeda*), including mature needles from field-grown trees, nursery seedlings, and commercially sourced seeds. Specimens were obtained from several regions within each species' native ranges, as detailed in Table 2. All leaves (needles) and seeds were stored with silica gel at −20°C until processing. To minimize the risk of cross-

Table 2. Sampling locations, tissue sources, and DNA parameters used for assay development.

Species	Sampling location	Developmental stage	Tissue source	Specimen (n)	Concentration (ng/ μ L)	A260/280	A260/230
<i>P. palustris</i> (Longleaf)	Mississippi	Trees	Needles	2	10.99	1.65	0.24
	Georgia	Seedlings	Needles	6	9.25	1.64	0.07
	Florida	Seedlings	Needles	10	9.14	1.70	0.06
	Louisiana	Seeds	Embryos	4	90.62	1.88	1.97
<i>P. elliotii</i> (Slash)	Mississippi	Trees	Needles	4	10.99	1.65	0.16
	Georgia	Seedlings	Needles	10	9.25	1.64	0.22
	Florida	Seedlings	Needles	8	9.14	1.70	0.32
<i>P. echinata</i> (Shortleaf)	Mississippi	Trees	Needles	5	45.59	1.62	0.48
	Missouri	Seedlings	Needles	9	16.70	1.94	0.38
	Arkansas	Seeds	Embryos	6	18.13	1.71	0.44
<i>P. taeda</i> (Loblolly)	Georgia	Seedlings	Needles	10	8.85	1.63	0.27
	Florida	Seedlings	Needles	10	13.26	1.55	0.25

contamination, work surfaces and tools were decontaminated with 10% bleach and 70% ethanol between samples. For DNA extraction, 50 mg of needle tissue was weighed per sample. For seeds, seed coats were removed, and a single embryo (mass not determined) was excised for each extraction using sterile tweezers and avoiding inclusion of endosperm tissue. Tissue disruption was performed in 2 mL tubes containing steel beads (DNeasy® Plant Pro Kit, QIAGEN, Germantown, MD, USA). Tubes were vortexed for 3 min, cooled on ice for 1 min, and the cycle was repeated (a total of 6 min vortexing). After addition of Solution CD1 (cell lysis buffer) and Solution PS (inhibitor removal reagent) from the kit, the vortex-cool cycle was repeated three additional times (9 min of vortexing with the solutions).

Total genomic DNA was isolated from the disrupted tissue following the DNeasy® Plant Pro Kit (Qiagen) manufacturer's protocol. DNA was eluted in 50 μ L of elution buffer. DNA concentration (ng/ μ L) and purity (A260/280 and A260/230 ratios) were measured using a NanoDrop 1000 spectrophotometer (Thermo Scientific, Wilmington, DE, USA). DNA extracts were diluted with nuclease-free water to a final working concentration of 5 ng/ μ L for qPCR assays. All DNA extracts were stored at -20°C until further use.

Primer design and *in silico* evaluation

Complete and partial chloroplast genome sequences for *Pinus palustris* (Longleaf pine, LLP; JN854176.1), *P. elliotii* (Slash pine, SLP; NC_042788.1, JN854202.1, MK135066.1), *P. echinata* (Shortleaf pine, SFP; NC_021440.1, JN854204.1, NC_065458.1, MZ424449.1), and *P. taeda* (Loblolly pine, LBP;

KC427273.1, KY964286.1) were retrieved from the National Center for Biotechnology Information (NCBI) GenBank database. The sequences were aligned in Geneious Prime® software (version 2024.0.5) by using the multiple alignment using fast fourier transform (MAFFT) plugin with default parameters (Kato and Standley 2013). Alignments were manually inspected to identify candidate diagnostic sequence polymorphisms located within potential primer binding sites that could distinguish between the four SYP species.

Primers targeting the diagnostic loci were initially designed using the Primer3 plugin in Geneious Prime®. Design parameters were set as follows: primer length of 18–30 bp, a product size of 75–150 bp, and an optimal melting temperature (T_m) of approximately 60°C with a maximum T_m difference of 2°C between primer pairs. This strict size constraint (<100 bp) was a deliberate methodological design choice to maximize the assay's future applicability to solid wood. Candidate primers were screened *in silico* to minimize secondary structures, such as hairpins and self-dimers. To improve primer performance and compatibility in multiplex reactions, sequences were manually refined beyond the initial Primer3 output. These refinements included optimizing G/C content at the 3' end (GC clamp), minimizing T_m differences between primer pairs, and reducing potential cross-dimerization. Final primer combinations were selected to generate amplicons with distinct melting temperatures, facilitating clear differentiation by melt curve analysis. Primer specificity was assessed *in silico* using NCBI Primer-BLAST against the nucleotide (nt) database restricted to *Pinus* (taxid:3318). Primer-BLAST results confirmed species-level specificity: LLP primers aligned only with *P. palustris*,

while SLP primers aligned with *P. elliottii* and *P. caribaea* (a non-regional species). Neither primer set aligned with *P. clausa*, *P. virginiana*, or any other species within the southern yellow pine group. This lack of complementarity indicates that the assay would fail to amplify these minor species, thereby preventing misidentification in mixed-species supply chains.

Final primer combinations were selected to generate amplicons with distinct melting temperatures, facilitating clear differentiation by melt curve analysis. The primer sequences, G/C content, and expected amplicon sizes are summarized in Table 3. To further evaluate the melting behavior of the amplicon products, *in silico* melt curve predictions were performed using uMELT v3 (<https://www.dna.utah.edu/umelt/>) using default parameters (Dwight et al. 2011). Each amplicon was assessed to ensure it produced a single, distinct melt peak, supporting its suitability for multiplexing. Predicted melting temperatures and complete amplicon sequences are shown in Table 4.

Simplex qPCR assay setup

Each primer pair was first validated individually in simplex qPCR assays under two distinct experimental conditions to assess performance and compatibility. First, assays were run under standard conditions following the master mix manufacturer's recommendations. These 10 µL reactions consisted of 5 µL PowerUp™ SYBR™ Green Master Mix (2X) (Thermo Fisher Scientific, Waltham, MA, USA), 5 ng of genomic DNA, and 0.5 µL each of forward and reverse primer (10 µM stock; Custom DNA Oligos Invitrogen), resulting in a final concen-

tration of 500 nM per primer (forward and reverse, 1000 nM in total primer concentration). Reactions were conducted in triplicate. Thermal cycling included Uracil-DNA Glycosylase (UDG) activation (50°C, 2 min) and denaturation (95°C, 2 min), followed by 35 cycles of 95°C for 30 s and 60°C for 1 min. Annealing temperature was optimized using gradient PCR from 55°C to 65°C to confirm single-amplicon specificity and determine characteristic melting temperatures (T_m). Second, assays were run under multiplex-simulation conditions to evaluate performance under the constraints of the final multiplex design. These reactions used reduced primer concentrations (200 nM for LLP; 300 nM for SLP) and the optimized multiplex thermal profile (denaturation at 95°C for 15 s; annealing/extension at 61°C for 20 s).

Multiplex qPCR assay optimization

Following simplex validation, a multiplex SYBR Green qPCR assay was optimized. An asymmetric primer concentration matrix was employed to compensate for the higher efficiency of the LLP primer pair (see results section for simplex assays). Optimization started with a total primer concentration of 1000 nM (matching the simplex assay), representing an initial 600 nM/400 nM (SLP/LLP) split. A matrix of combinations was then tested, using total SLP concentrations of 600, 400, and 200 nM and total LLP concentrations of 400 and 200 nM. Each concentration combination was tested in triplicate. Optimal concentrations (Table 5) were selected to minimize the Cq difference between LLP and SLP, but this condition produced primer-dimer artifacts and overlapping melt peaks (~69.0°C),

Table 3. Primer sequences and characteristics for SYBR Green qPCR assays targeting southern yellow pines.

Primer ID	Sequence (5' to 3')	G/C content (%)	T_m (°C)	Target gene	Product (bp)
LLP_F	TCTCATCATT[TTCCGA]TTCCG	42.9	62	ycf2	99
LLP_R	TCAATTCAGGATATGGCAAGAAC	39.1	60		
SLP_F	CTTGTGTCAACTAAAAAGAAGTAAAAA[TACC]	29.0	60	trnL-trnF IGS	95
SLP_R	TTTTTATCATATTAGTGACTTCCAGATCG	31.0	61		

Primer IDs indicate species and orientation: LLP for *Pinus palustris* (Longleaf pine), SLP for *P. elliottii* (Slash pine), with F and R denoting forward and reverse primers, respectively. Diagnostic sequences are bracketed within the primer binding sites.

Table 4. Predicted primer and amplicon sequences and melt temperatures for *Pinus palustris* and *Pinus elliottii*.

Primer	Primer + amplicon sequence	Melt peak (°C)
<i>P. palustris</i> (Longleaf)	TCTCATCATT[TTCCGA]TTCCGA ATGAGTTCGCTCGGTCCTTG ATTCTTCGAGATCATCTCATCCTTCTGTTCGTGTTCTTGGCAAT TCCTGAATTGA	84
<i>P. elliottii</i> (Slash)	CTTGTGTCAACTAAAAAGAAGTAAAAA[TACC] TTTTTACTAGT TTTTAGAATGATCTTTATTATTTTCGATCTGGAAGTCACTAATAT GATAAAAA	76

Primer binding sites are shown in bold. Species-specific sequences are bracketed. Melt peaks were calculated using uMELT.

Table 5. Reaction components for the optimized multiplex SYBR Green qPCR assay.

Component	Stock concentration	Volume per reaction (μL)	Final concentration
PowerUp™ SYBR™ Green Master Mix	2X	5.0	1X
SLP Forward Primer	10 μM	0.3	300 nM
SLP Reverse Primer	10 μM	0.3	300 nM
LLP Forward Primer	10 μM	0.2	200 nM
LLP Reverse Primer	10 μM	0.2	200 nM
DNA Template	5 ng/μL	1.0	5 ng
Nuclease-Free Water	N/A	3.0	N/A
Total volume	N/A	10.0	N/A

interfering with species differentiation (see results section, Figure 4). To address this, thermal cycling conditions were optimized by increasing the annealing temperature to 61°C and shortening the annealing/extension time to 20 s, and the denaturation time to 15 s. Final 10 μL multiplex qPCR reactions contained 5 μL PowerUp™ SYBR™ Green Master Mix (2X) (Thermo Fisher Scientific, Waltham, MA, USA), the optimal concentration of SLP and LLP primer pairs (Table 5), and 5 ng genomic DNA template. For extracts with a concentration below 5 ng/μL, 1 μL of the undiluted extract was used.

Thermal cycling and fluorescence data acquisition were performed on a BioRad iQ5 Real-Time PCR Detection System (Bio-Rad Laboratories, Inc., Hercules, CA, USA). The optimized protocol consisted of an initial Uracil-DNA Glycosylase (UDG) activation (50°C, 2 min), followed by denaturation and polymerase activation (95°C, 2 min). Amplification was carried out for 40 cycles of denaturation (95°C, 15 s) and annealing/extension (61°C, 20 s), with data acquisition at each annealing/extension step. Following amplification, a melt curve analysis was performed by increasing the temperature from 61°C to 95°C, with 0.5°C increments per 10 s, to confirm amplicon specificity.

Assay validation

Validation of the qPCR assays followed the Minimum Information for Publication of Quantitative Real-Time PCR Experiments (MIQE) guidelines (Bustin et al. 2009). A MIQE compliance table is available as Table S1 in the supplement. The simplex assays were evaluated for specificity, efficiency and linearity, and sensitivity using a single representative specimen for each species. The multiplex assay was validated across all four validation steps, including screening performance:

- Specificity: Determined by testing the assay against genomic DNA from target and non-target *Pinus* species. The specificity of amplification products was further confirmed

by analysis of melt curves for each reaction (Ririe et al. 1997), ensuring a single, distinct melting peak for each target amplicon.

- Efficiency and linearity: Established using serial dilutions of target DNA (10-fold, across six orders of magnitude) to generate standard curves and calculate efficiency from the slope.
- Sensitivity: Defined as the lowest DNA concentration at which amplification was reliably detected in ≥ 95% of at least 20 replicates (Huber et al. 2013; Broeders et al. 2014).
- Screening performance: Evaluated on a panel of 84 individuals (Table 2), to determine accuracy, precision, and recall.

Specificity

To evaluate specificity, genomic DNA was extracted from one mature leaf from one individual for each of the two target species (*P. palustris* and *P. elliottii*) and the two non-target species (*P. taeda* and *P. echinata*), all collected in Mississippi. For each species, triplicate multiplex qPCR reactions were prepared using 5 ng of extracted DNA. Following amplification, melt curve analysis was performed to detect the presence of single, target-specific peaks, indicating specific amplification. For further confirmation, 4% agarose gel electrophoresis was used to separate qPCR products, with gels stained with SYBR Safe (Invitrogen, Carlsbad, CA, USA) and imaged under UV light with Bio-Rad Gel Doc XR+ System (Bio-Rad Laboratories, Inc., Hercules, CA, USA). Assay specificity was validated by the absence of off-target peaks in melt curves and non-specific bands on gels, and by verification of expected amplicon sizes for each target.

Efficiency and linearity

To assess quantitative performance, standard curves were generated separately for *P. palustris* and *P. elliottii*, using multiple

extractions of the same individuals used for specificity evaluation. For each species, a 6-point, 10-fold serial dilution series was prepared, ranging from 10 ng to 0.1 pg per reaction. Each dilution was tested in triplicate qPCR reactions.

Standard curves were generated by plotting quantification cycle (C_q) values against the logarithm (base 10) of the DNA concentration for each species. The coefficient of determination (R²) was calculated to assess the linearity of the assay over the dilution range. Efficiency (*E*) was determined from the slope of the standard curve using Equation (1). Assay performance was considered valid only when the standard curves exhibited R² values ≥ 0.98 and efficiencies were in the 80%–120% range (Taylor et al. 2010; Broeders et al. 2014).

$$E = 10^{\left(\frac{-1}{\text{slope}}\right)} - 1 \quad [1]$$

Sensitivity

Sensitivity was evaluated by determining the limit of detection (LOD) for each species. The lowest concentrations in the dilution series were analyzed using a minimum of 20 replicates from the same extraction (technical replicates) per concentration (Bustin et al. 2009, 2025; Bell 2016). The LOD was defined as the lowest DNA input at which ≥ 95% of technical replicates produced an amplification and a specific melt curve peak corresponding to the expected amplicon. All LOD testing included appropriate no-template controls to monitor contamination and false positives.

Screening performance

To assess the screening accuracy and specificity of the optimized multiplex qPCR assay, 84 individuals, as detailed in Table 2, were evaluated. Genomic DNA was extracted from leaf or seed tissue from each specimen using the previously described extraction protocol. Each DNA sample was analyzed with the optimized multiplex qPCR protocol. To minimize the risk of cross-contamination, individuals from different species were processed and amplified in separate qPCR runs. Negative (no-template) controls were included in each run to monitor contamination.

Scoring of assay results was performed as follows: A sample was classified as a true positive (TP) if the assay correctly identified it as either SLP or LLP based on the presence of the characteristic melt peak. A true negative (TN) was recorded when a non-target species (LBP or SFP), with template DNA at standardized template concentrations (5 ng), showed no

amplification signal. False positives (FP) were defined as any amplifications of non-target species, while false negatives (FN) were defined as any failure to detect a target species. For each metric, results were tabulated across all samples. The screening accuracy (Eq. 2), precision (Eq. 3), and recall (Eq. 4) were calculated using the following standard formulas:

$$\text{Accuracy} = \frac{TP + TN}{TP + TN + FP + FN} \quad [2]$$

$$\text{Precision} = \frac{TP}{TP + FP} \quad [3]$$

$$\text{Recall} = \frac{TP}{TP + FN} \quad [4]$$

Results

Simplex assay results

Specificity

Amplification curves for *P. palustris* and *P. elliottii* were compared between the two experimental setups (Figure 2, Panels A–D). Under standard conditions (1000 nM primer concentration; standard cycling), amplification signals were strong and consistent. However, under multiplex-simulation conditions (primer concentration of 200 nM for *P. palustris*, 300 nM for *P. elliottii*; rapid cycling), amplification signals were weaker, and C_q values were higher. Melt curve analysis (Figure 2, Panels E–H) showed single peaks for each species with mean melting temperatures (T_m) of 77.83 ± 0.29°C (*P. palustris*) and 69.5 ± 0.0°C (*P. elliottii*).

Efficiency and linearity

Standard curves for both species under the two conditions are shown in Figure 3. Under standard conditions (Panels A, C), amplification efficiencies were 99.1% for *P. palustris* and 103.1% for *P. elliottii*, with R² values exceeding 0.99. Under multiplex-simulation conditions (Panels B, D), efficiencies decreased to 77.0% for *P. palustris* and 73.3% for *P. elliottii*, with R² values still above 0.98.

Multiplex assay results

Assay optimization

The initial multiplex qPCR assay used a primer concentration of 300 nM for *Pinus elliottii* and 200 nM for *Pinus palustris* in one reaction, based on the starting point of a systematic asymmetric primer concentration matrix. Melt curve analysis of the no-template control (NTC) showed a low-amplitude peak at

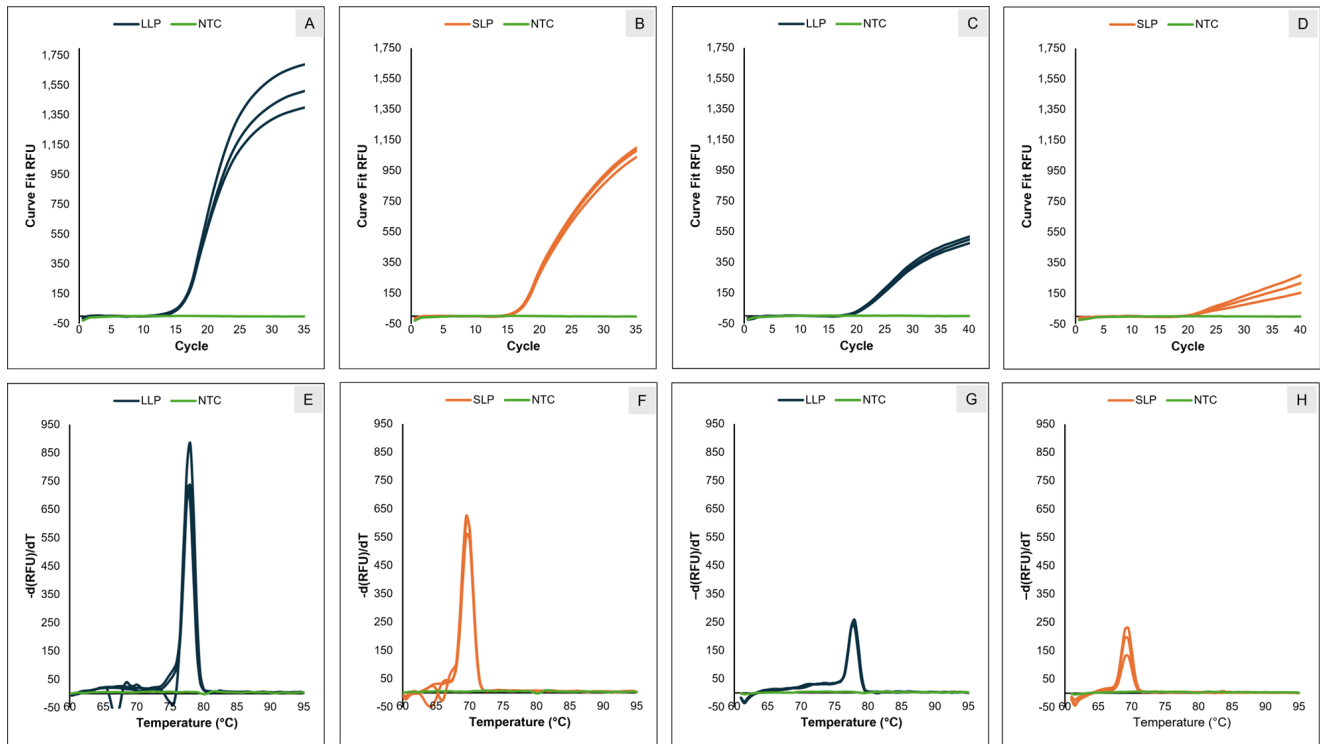


Figure 2. Amplification and melt curves for simplex qPCR assays under two conditions. (A–B) Amplification curves for *P. palustris* and *P. elliottii* at 1000 nM primer concentration (standard conditions); (C–D): Amplification curves for *P. palustris* (200 nM) and *P. elliottii* (300 nM) under multiplex-simulation conditions; (E–H) Melt peaks remained consistent (78.0 °C for *P. palustris*, 69.0 °C for *P. elliottii*) across both conditions.

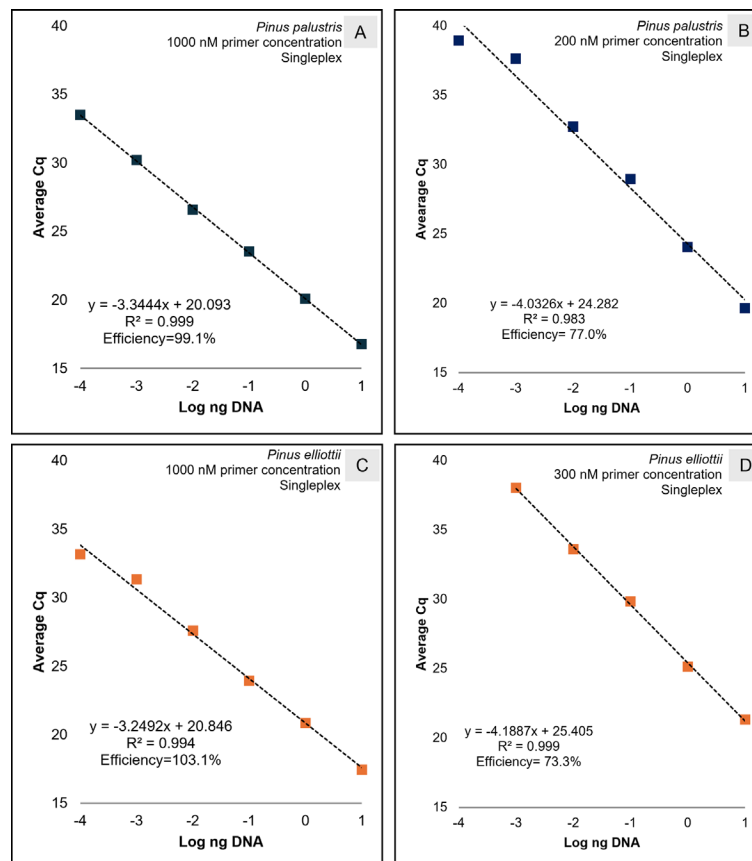


Figure 3. Standard curves for simplex qPCR assays under two conditions. (A, C): Standard conditions (1000 nM primers); (B, D): Multiplex-simulation conditions (200/300 nM primers). Amplification efficiencies were 99.1% and 103.1% under standard conditions, and 77.0% and 73.3% under multiplex-simulation conditions.

approximately 69.0°C. Agarose gel electrophoresis revealed low-molecular-weight bands in the NTC. These observations were recorded during initial testing with the above primer concentrations. Figure 4 presents the optimized multiplex qPCR assay results, including amplification curves (Figure 4A), melt curve profiles, and an inset gel electrophoresis for amplicon size confirmation (Figure 4B).

Specificity

The optimized multiplex qPCR assay used a primer concentration of 300 nM for *P. elliottii* and 200 nM for *P. palustris* in a single reaction, an annealing/extension temperature of 61°C, and shortened cycling time to 20 s. Under optimized multiplex conditions, amplification was observed only in the target species. Melt curve analysis showed distinct peaks with mean melting temperatures (T_m) of $77.67 \pm 0.29^\circ\text{C}$ for *P. palustris* and $69.0 \pm 0.0^\circ\text{C}$ for *P. elliottii*. No amplification was detected in non-target species (*P. taeda*, *P. echinata*) or in the no-template control (NTC). Agarose gel electrophoresis confirmed single bands of expected size for both target species and the absence of products in non-target species and the NTC (Figure 5).

Efficiency and linearity

Standard curves were generated for *P. palustris* and *P. elliottii* using 6-point, 10-fold serial dilutions of genomic DNA ranging from 10 ng to 0.1 pg of input DNA per reaction. The coefficients

of determination (R^2) exceeded 0.99 for both species, indicating high linearity across this range. Amplification efficiency (E) was calculated as 86.2% for the *P. palustris* assay and 81.6% for the *P. elliottii*, based on the slope of the standard curves (Figure 6).

Sensitivity

The limit of detection (LOD) was defined as the lowest DNA concentration at which $\geq 95\%$ of technical replicates produced a positive amplification and a specific melt curve peak. For *P. palustris*, the LOD was 1 pg (0.001 ng), and for *P. elliottii*, it was 10 pg (0.01 ng). At these concentrations, all 21 technical replicates amplified successfully (100%) and displayed a single melt peak at the expected melting temperature (T_m). The mean quantification cycle (C_q) values were 34.03 ± 0.90 for *P. palustris* and 33.20 ± 0.64 for *P. elliottii*, with all values below the MIQE-recommended threshold of 35 cycles.

Screening performance

The optimized multiplex qPCR assay was evaluated on a panel of 84 individuals representing both target and non-target species (Table 6). All 44 individuals representing target species (*P. palustris* and *P. elliottii*) amplified successfully and produced single melt peaks at the expected melting temperatures, whereas no amplification was observed in any of the 40 individuals of the non-target species (*P. echinata* and *P. taeda*) or in no-template controls (NTC). Based on these results, the

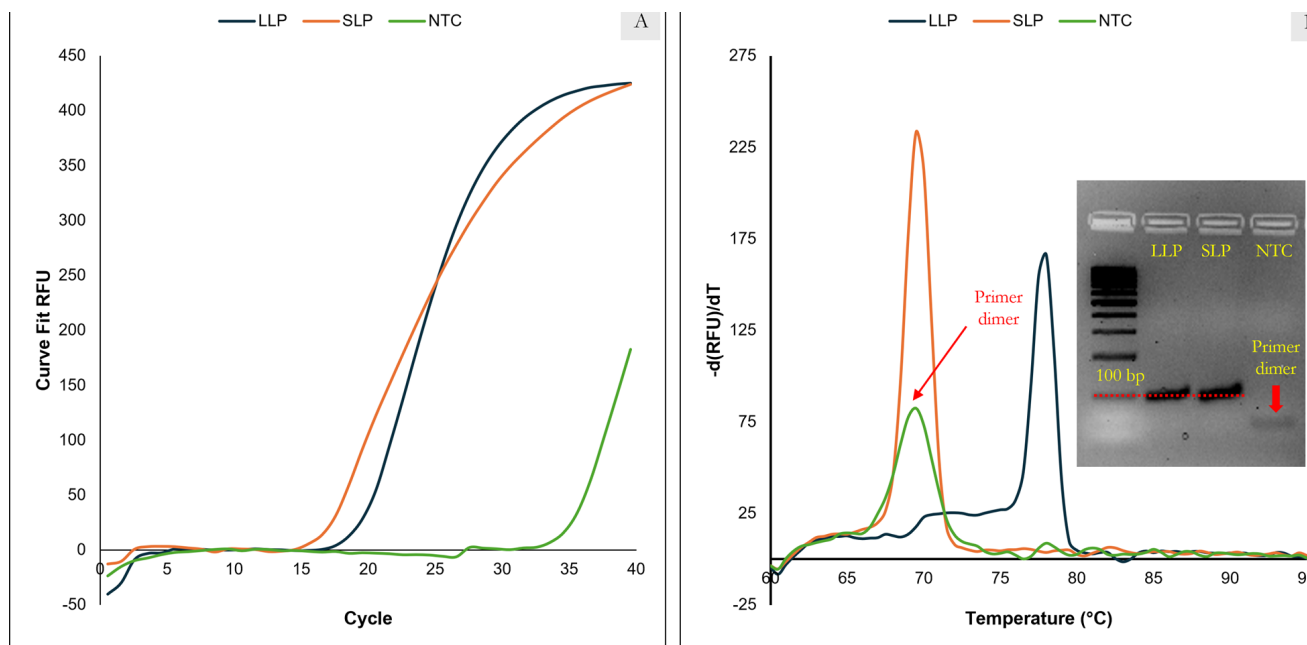


Figure 4. Multiplex qPCR assay results showing amplification and melt curve profiles with an inset gel electrophoresis. (A) Real-time amplification curves for *Pinus palustris* (LLP) and *P. elliottii* (SLP) under optimized multiplex conditions; (B) Melt curve analysis with distinct peaks at 78.0 °C (LLP) and 69.0 °C (SLP), confirming species differentiation. Inset: Agarose gel electrophoresis showing expected amplicon sizes for target species and absence of products for non-targets and NTC.

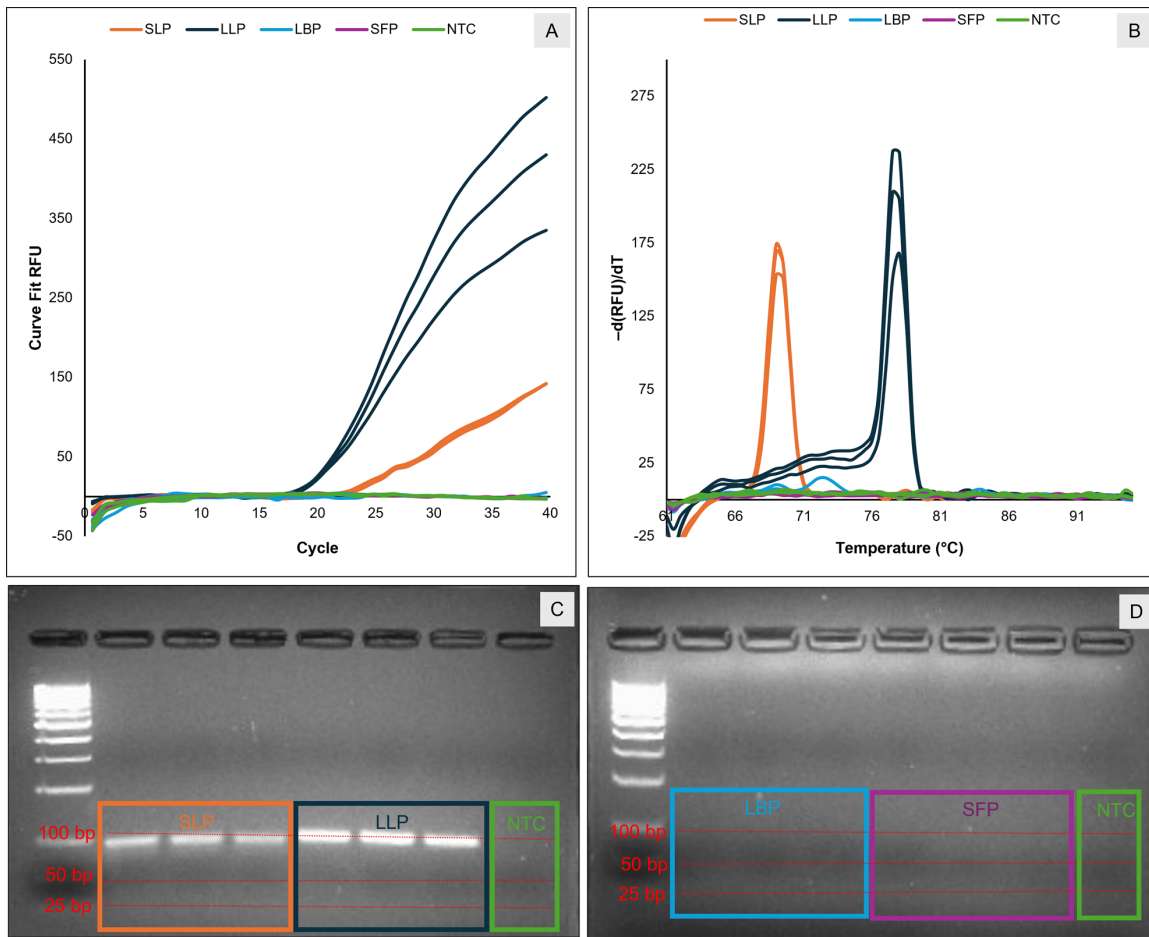


Figure 5. Analytical specificity of the optimized multiplex qPCR assay. (A) Real-time amplification curves for *Pinus eliottii* (SLP) and *P. palustris* (LLP) under optimized multiplex conditions. Non-target species (*P. taeda* (LBP), *P. echinata* (SFP)) and no-template control (NTC) showed no amplification; (B) Melt curve analysis showing distinct melting temperature (T_m) peaks at 69.0 $^{\circ}C$ for SLP and 78.0 $^{\circ}C$ for LLP, confirming species differentiation; (C) Agarose gel electrophoresis for target species showing expected amplicon sizes for SLP and LLP; (D) Agarose gel electrophoresis for non-target species and NTC showing absence of amplification products.

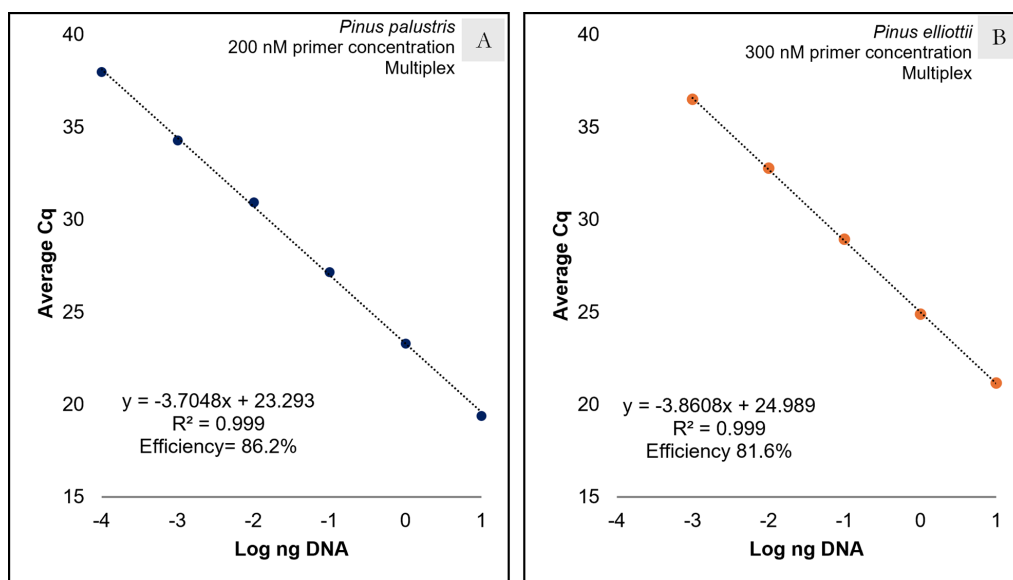


Figure 6. Standard curves for multiplex qPCR assays of *Pinus palustris* (A) and *P. eliottii* (B). Six-point, 10-fold serial dilutions (10 ng–0.1 pg) were amplified under optimized multiplex conditions. x-axis: $\log_{10}$ DNA concentration (ng); y-axis: quantification cycle (Cq). The linearity (R^2) exceeded 0.99 for both species; amplification efficiencies (E) were 81.6% (*P. eliottii*) and 86.2% (*P. palustris*).

Table 6. Quantification cycle (Cq) values for *Pinus palustris* and *P. elliottii* samples.

Species	Origin	Samples (n)	Cq (mean $\pm$ SD)
<i>P. palustris</i> (Longleaf)	Florida (FL)	10	21.6 $\pm$ 1.74
	Georgia (GA)	6	24.3 $\pm$ 0.50
	Mississippi (MS)	2	22.5 $\pm$ 0.70
	Louisiana (LA)	4	16.8 $\pm$ 0.59
<i>P. elliottii</i> (Slash)	Florida (FL)	9	23.2 $\pm$ 3.10
	Georgia (GA)	9	27.2 $\pm$ 4.29
	Mississippi (MS)	4	22.6 $\pm$ 2.07
	Mississippi (MS)	5	NA
<i>P. echinata</i> (Shortleaf)	Missouri (MO)	9	NA
	Arkansas (AR)	6	NA
	Florida (FL)	10	NA
<i>P. taeda</i> (Loblolly)	Georgia (GA)	10	NA

Values are presented as mean $\pm$ standard deviation for each origin. NA indicates no amplification in the multiplex qPCR assay.

screening performance metrics were calculated as follows: True Positives (TP) = 44, True Negatives (TN) = 40, False Positives (FP) = 0, False Negatives (FN) = 0, yielding 100% accuracy, precision, and recall.

Quantification cycle (Cq) values for the target samples (all at template concentration of 5 ng/ μ L) are summarized in Table 6, which shows mean Cq values ranged from 16.8 (Louisiana) to 24.3 (Georgia) for *P. palustris*, while *P. elliottii* ranged from 22.6 (Mississippi) to 27.2 (Georgia). All values remained below the MIQE-recommended threshold of 35 cycles. Figure 7 shows representative amplification curves and melt curve profiles for individuals from Florida, Georgia, Mississippi, and Louisiana, highlighting the distinct mean melting temperatures (T_m) (77.98 $\pm$ 0.11°C for *P. palustris* and 69.25 $\pm$ 0.11°C for *P. elliottii*) that enable clear species differentiation.

Discussion

Simplex assay performance

The simplex assays confirmed that high primer concentrations under standard conditions produced strong amplification and near-ideal efficiency, consistent with MIQE guidelines for qPCR performance (Bustin et al. 2009). When reactions were run under multiplex-simulation conditions with lower primer concentrations and shortened denaturation and annealing time, amplification efficiency decreased, but melt curve profiles remained unchanged. This stability indicates that specificity was maintained despite reduced amplification, supporting the reliability of melt curve analysis for differentiating the species (Ririe et al. 1997; Chabi et al. 2019). The reduced efficiency under multiplex-simulation conditions was expected because

lower primer concentrations and shorter extension steps limit primer-template interactions and amplification kinetics (Kralik and Ricchi 2017). This trade-off between efficiency and multiplex compatibility is widely reported in multiplex qPCR studies (Safdar and Junejo 2015; Sarkar et al. 2022).

Multiplex assay optimization

The initial multiplex reaction assay resulted in primer-dimer formation, evidenced by a low-amplitude melt peak near 69.0°C in the no-template control (NTC), and a low-molecular-weight band in agarose gels. To address this, the annealing temperature was increased to 61°C, and denaturation/annealing time was shortened to 15 s and 20 s, respectively. These adjustments are consistent with established qPCR optimization strategies, which show that higher annealing temperatures reduce non-specific primer binding and primer-dimer formation, while shorter annealing time prevents unintended amplification (Safdar and Junejo 2015; Kralik and Ricchi 2017).

Melt curve analysis was critical for detecting these artifacts because SYBR Green binds all double-stranded DNA, and primer dimers typically melt at a lower temperature than target amplicons (Ririe et al. 1997). Balancing amplification efficiency in multiplex reactions is a common technical hurdle. We demonstrate that optimizing primer asymmetry (300 nM vs. 200 nM) effectively mitigates competition between targets without sacrificing sensitivity (Sarkar et al. 2022).

Specificity

The Primer-BLAST search confirmed species-level specificity: (1) LLP matched only *P. palustris*, (2) SLP primers matched *P. elliottii* and *P. caribaea*, a non-regional species, minimiz-

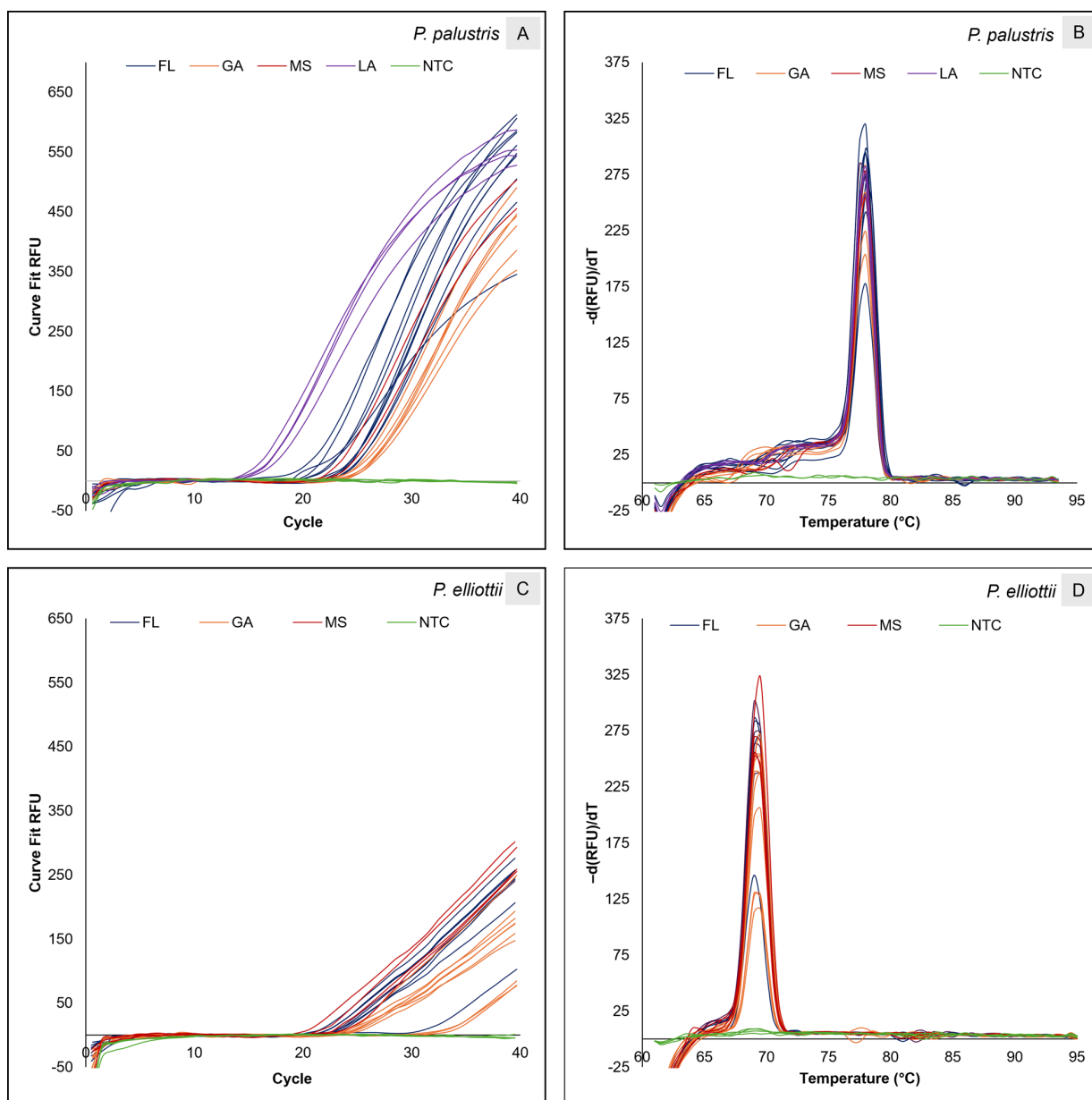


Figure 7. Screening performance of the multiplex qPCR assay for *Pinus palustris* and *P. elliotii*. (A) Amplification curves for *P. palustris* (Longleaf pine) from Florida (FL), Georgia (GA), Mississippi (MS), and Louisiana (LA); (B) Melt curve analysis for *P. palustris* showing a single peak at 78.0 °C; (C) Amplification curves for *P. elliotii* (Slash pine) from FL, GA, and MS; (D) Melt curve analysis for *P. elliotii* showing a single peak at 69.0 °C. No-template control (NTC) showed no amplification in all panels.

ing practical risk of misidentification. (3) No matches were found for other regional species not typically found in trade (*P. clausa* or *P. virginiana*). Predicted melt temperatures from uMELT supported the expectation of distinct peaks, although observed values deviated slightly, likely due to experimental factors such as salt concentration and ramp rate (Dwight et al. 2011). The ~9°C separation between peaks provided strong species discrimination (100% accuracy, precision, and recall)

under multiplex conditions, a strategy validated for SYBR Green assays (Zhou et al. 2017; Chabi et al. 2019).

Efficiency and linearity

The assay demonstrated high linearity under both simplex and multiplex conditions, with R^2 values exceeding 0.99 across six orders of magnitude of DNA concentration, meeting MIQE guidelines for qPCR performance (Bustin et al. 2009). Under standard conditions (1000 nM primers), amplification efficien-

cies were near ideal (99.1% for *P. palustris* and 103.1% for *P. elliottii*), indicating strong quantitative performance. When reactions were run under multiplex-simulation conditions with reduced primer concentrations and shortened cycling time, efficiencies decreased to 77.0% and 73.3%, respectively.

In the optimized multiplex conditions, efficiencies improved to 86.2% for *P. palustris* and 81.6% for *P. elliottii*, confirming that primer concentration balancing and thermal adjustments mitigated some of the trade-off between multiplex compatibility and amplification kinetics (Kralik and Ricchi 2017; Sarkar et al. 2022). These values fall within the 80%–120% range recommended for diagnostic screening, supporting the assay's reliability for species identification, rather than precise quantification (Broeders et al. 2014). The lower efficiency for the *P. elliottii* target may be affected by the lower G/C content of its primers and amplicon (Kralik and Ricchi 2017). This issue could not be solved by redesigning the primers in our case, due to the location of the target sequences, inherent interspecific DNA sequence similarities of cpDNA in *Pinus*, and sequence availability in NCBI.

Sensitivity and DNA Quality

The multiplex qPCR assay demonstrated high sensitivity, detecting as little as 1 pg of DNA for *P. palustris* and 10 pg for *P. elliottii*, with all technical replicates producing specific melt peaks and C_q values below the MIQE threshold of 35 cycles (Bustin et al. 2009; Forootan et al. 2017). These limits of detection indicate the assay's suitability for low-template scenarios, which is critical for future application to solid wood, where DNA quantity and quality are often compromised (Rachmayanti et al. 2009; Costa et al. 2022).

DNA extracted from leaf and seed tissues showed low A260/280 and A260/230 ratios, suggesting the presence of proteins, polysaccharides, and phenolic compounds—known PCR inhibitors in plant tissues (Demeke and Jenkins 2010). Despite these challenges, sensitivity testing confirmed reliable amplification at picogram-level DNA inputs, supporting the robustness of short amplicons (<100 bp) for material with low quantity and low-quality DNA, a strategy validated for forensic wood identification (Dormontt et al. 2015; Debode et al. 2017).

Screening Performance

The optimized multiplex qPCR assay achieved 100% accuracy, precision, and recall across 84 individuals, with no false positives or false negatives observed. All samples of target species exhibited single melt peaks at the expected temperatures (78.0°C and 69.0°C), while samples of non-

target species (*P. taeda* and *P. echinata*) and no-template controls showed no amplification. This performance aligns with MIQE-based validation principles emphasizing accuracy, reproducibility, and specificity for quantitative real-time PCR assays (Broeders et al. 2014; Bustin et al. 2025). The absence of misclassification (i.e., false positives and false negatives) demonstrates that melt curve analysis provided sufficient resolution for species differentiation, even in specimens with variable DNA purity—a known challenge in plant molecular diagnostics (Demeke and Jenkins 2010; Schrader et al. 2012). Compared to conventional PCR methods, such as the one reported by Olatinwo et al. (2020), which requires post-PCR gel electrophoresis, this multiplex qPCR approach offers a faster workflow by detecting and separating two species in a single reaction (Sarkar et al. 2022). Furthermore, the use of SYBR Green chemistry provides a more cost-effective alternative to hydrolysis probe-based assays (Tajadini et al. 2014; Naeem et al. 2024). Most critically, our assay's short amplicons (<100 bp) are designed for low-quality and low-quantity DNA, which is expected to be a key feature for future work with solid wood (Debode et al. 2017).

Although all target samples were standardized to 5 ng/μL DNA input, Table 6 shows considerable variation in C_q values across geographic origins, ranging from 16.8 (Louisiana) to 24.3 (Georgia) for *P. palustris* and from 22.6 (Mississippi) to 27.2 (Georgia) for *P. elliottii*. These differences likely reflect biological and/or chemical factors. First, variation in parent material quality, such as the number of viable cells per unit tissue, can influence the effective copy number of chloroplast genomes, which likely affects amplification kinetics (Rowan et al. 2009). Second, residual PCR inhibitors (e.g., polysaccharides, phenolic compounds) commonly present in pine tissues may not have been fully removed by the commercial extraction kit, reducing amplification efficiency despite equalized DNA concentrations (Demeke and Jenkins 2010). Both factors are plausible explanations for the observed C_q range and have been widely reported in plant qPCR studies (Brunner et al. 2004; Kontanis and Reed 2006; Rowan et al. 2009; Demeke and Jenkins 2010; Die et al. 2016). While these differences did not compromise species identification, they underscore the importance of considering DNA purity and inhibitor removal in future applications.

In this study, true negatives were defined by the lack of amplification in non-target species assayed at standard template concentrations of 5 ng/μL DNA. However, in practical applications involving an unknown template DNA concentration (as

would be true in the field with solid wood), the absence of an amplification signal cannot necessarily be interpreted as the absence of the target species. As noted in studies involving solid wood (Mohamed et al. 2012; Lee et al. 2016), reaction failure can be a result of a failed extraction protocol, degradation of target DNA in the specimen, or an extracted DNA concentration below the detection threshold. One approach to overcome these concerns would be to incorporate an internal amplification control (IAC) (Hoorfar et al. 2004). If the IAC fails to amplify, it suggests that there are one or more problems with the PCR conditions, indicating that the results of the test are inconclusive. Future work on solid wood should develop empirical diagnostic criteria for interpreting results based on rates of false negatives, false positives, and amplification failures when using wood as the DNA source.

Target plant genomes, marker development for *Pinus* Identification

The development of molecular screening tools for southern yellow pines is constrained by the close phylogenetic relationship among the species, which limits the number of differentiating genetic markers (Jin et al. 2021), especially in the chloroplast genome (plastome), where few sequence polymorphisms suitable for species discrimination have been identified. The plastome of *Pinus* is moderately A/T-rich (about 60% A + T overall) and contains simple sequence repeats, many of which are mononucleotide tracts composed of A or T bases (Powell et al. 1995). Such low-complexity, A/T-rich, or repetitive regions reduce the number of suitable primer-binding sites, requiring careful design to avoid non-specific amplification. The limitations of chloroplast-based markers highlight the importance of considering the distinct characteristics of the three plant genomes—chloroplast, mitochondrial, and nuclear—for species identification (Hollingsworth et al. 2011). The chloroplast genome in *Pinus*, a small, circular chromosome, is typically inherited paternally (Neale and Sederoff 1989; Chen et al. 2002) and is present in high copy number per cell (Sakamoto and Takami 2018), but its low rate of nucleotide substitution limits its utility for distinguishing closely related species (Wolfe et al. 1987), especially when considering the limited number of NCBI sequences available to assess sequence variability within and between taxa.

The mitochondrial genome in conifers is maternally inherited (Neale and Sederoff 1989) and present in high copy number per cell (Wolfe et al. 1987). Although it evolves at a slower nucleotide substitution rate compared to nuclear and chloroplast genomes (Drouin et al. 2008), this does not imply a lack of

informative variation. In fact, studies have shown that conifers, due to their ancient lineage and long generation times, can accumulate substantial mitochondrial diversity over evolutionary timescales (Wu et al. 1998). Mitochondrial haplotypes have been successfully used to distinguish populations in *Pinus* (Potter et al. 2013), although their utility for fine-scale species discrimination may be limited compared to chloroplast markers (Fang et al. 2009). The mitochondrial genome is also structurally complex, often consisting of multiple large, recombining circular molecules (Gualberto and Newton 2017; Jackman et al. 2020). In contrast, the nuclear genome is organized into large, linear, biparentally inherited chromosomes and, despite its single copy number per cell, its enormous size in *Pinus* (often exceeding 20 Gb) (Neale et al. 2014; Zimin et al. 2014) provides a vast reservoir of comparatively rapidly mutating non-coding regions expected to contain an abundance of targets suitable for high-resolution assays (Eckert et al. 2010). The selection of a genomic target for species identification thus requires balancing genetic resolution with expected DNA source quantity and quality. For plant parts or products with high-quality and quantity DNA, the nuclear genome inevitably provides the most promise for high-resolution diagnostics (Eckert et al. 2010; Neale et al. 2014). For plant parts or products with inherently low quantity, and probably also low-quality DNA, such as wood, recovering the single-copy nuclear DNA can be challenging, even for small amplicons, in part due to the low copy number, and in part as a result of the comparatively high environmental lability of DNA in a linear chromosome (Watanabe et al. 2015). In these cases, high-copy-number organellar DNA, often with circular chromosomes, is a more practical target, as its abundance increases the likelihood of successful amplification in low-yield and/or low-quality material (Dumolin-Lapegue et al. 1997).

This genomic comparison highlights a direction for future research. Expanding marker discovery efforts beyond the highly constrained plastome (Liu et al. 2021) and into the nuclear genome is an alternative approach to enhance screening capabilities (Särkinen and George 2013). Exploring these larger and more diverse genomic regions offers the potential to uncover more diagnostically informative sequences, which would provide much-needed flexibility for assay design and could enable the development of higher-capacity multiplex assays for more comprehensive species identification (Hollingsworth et al. 2011). Such an approach would call for more reliable DNA extraction protocols—regardless of the targeted plant genome—that can overcome the low-yield DNA nature of wood and wood products (Costa et al. 2022).

Conclusions

The successful validation of this assay on leaf and seed tissue serves as a critical proof of concept for its application to solid wood. When coupled with a rapid, field-deployable DNA extraction protocol for wood and comparably field-deployable hardware, with lyophilized reagents, this assay could serve as a highly specific screening tool for the Southern yellow pine industry, where distinguishing anatomically similar species is a known, significant challenge. Ultimately, this work establishes a robust molecular workflow that integrates short-amplicon design with high-resolution melt curve analysis. While validated here for *Pinus*, this method offers an exemplar for developing high-throughput diagnostic tools for other plant taxa where morpho-anatomical identification is insufficient. Future validation using DNA extracted from solid wood will require systematic assessment, particularly as DNA yield and integrity are expected to be substantially lower than from needles or seeds. Wood-derived DNA often exhibits extensive degradation and co-extracted PCR inhibitors; furthermore, processing methods—such as drying from green to kiln-dried—introduce significant variability. Future research should quantify detection performance across these material types, specifically measuring amplification success rates to evaluate whether the assay maintains required sensitivity under these constraints.

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Appendix - Supplemental tables

Table S1. MIQE checklist compliance.

Item to check	Importance	Checklist
EXPERIMENTAL DESIGN		
Definition of experimental and control groups	E	Materials and Methods
Number within each group	E	Table 2
Assay carried out by core lab or investigator's lab?	D	NA
Acknowledgement of authors' contributions	D	NA
SAMPLE		
Description	E	Materials and Methods
Volume/mass of sample processed	D	Materials and Methods
Microdissection or macrodissection	E	Materials and Methods
Processing procedure	E	Materials and Methods
If frozen - how and how quickly?	E	Materials and Methods
If fixed - with what, how quickly?	E	NA
Sample storage conditions and duration (especially for FFPE samples)	E	Materials and Methods
NUCLEIC ACID EXTRACTION		
Procedure and/or instrumentation	E	Materials and Methods
Name of kit and details of any modifications	E	Materials and Methods
Source of additional reagents used	D	NA
Details of DNase or RNase treatment	E	NA
Contamination assessment (DNA or RNA)	E	NA
Nucleic acid quantification	E	Materials and Methods
Instrument and method	E	Materials and Methods
Purity (A260/A280)	D	Table 2
Yield	D	Table 2
RNA integrity method/instrument	E	NA
RIN/RQI or Cq of 3' and 5' transcripts	E	NA
Electrophoresis traces	D	NA
Inhibition testing (Cq dilutions, spike or other)	E	ND
REVERSE TRANSCRIPTION		
Complete reaction conditions	E	NA
Amount of RNA and reaction volume	E	NA
Priming oligonucleotide (if using GSP) and concentration	E	NA
Reverse transcriptase and concentration	E	NA
Temperature and time	E	NA
Manufacturer of reagents and catalogue numbers	D	NA
Cqs with and without RT	D*	NA
Storage conditions of cDNA	D	NA

NA = Not applicable; ND = Not determined; E = Essential; D = Desired

* Testing for DNA contamination with a no-reverse-transcription assay is essential when RNA is first extracted. After DNA-free status has been verified, the control is recommended but no longer required.

** Reporting probe sequences is strongly encouraged to support transparency and reproducibility, but it cannot be required because some commercial predesigned assays do not provide this information.

Table S1. MIQE checklist compliance (*continued*).

Item to check	Importance	Checklist
qPCR TARGET INFORMATION		
If multiplex, efficiency and LOD of each assay	E	Results
Sequence accession number	E	Materials and Methods
Location of amplicon	D	Materials and Methods
Amplicon length	E	Table 4
<i>In silico</i> specificity screen (BLAST, etc.)	E	Results
Pseudogenes, retropseudogenes or other homologs?	D	NA
Sequence alignment	D	NA
Secondary structure analysis of amplicon	D	Results, Figure 4
Location of each primer by exon or intron (if applicable)	E	NA
What splice variants are targeted?	E	NA
qPCR OLIGONUCLEOTIDES		
Primer sequences	E	Table 3
RTPrimerDB Identification Number	D	NA
Probe sequences	D**	NA
Location and identity of any modifications	E	Table 3
Manufacturer of oligonucleotides	D	Materials and Methods
Purification method	D	NA
qPCR PROTOCOL		
Complete reaction conditions	E	Table 5
Reaction volume and amount of cDNA/DNA	E	Table 5
Primer, (probe), Mg ⁺⁺ and dNTP concentrations	E	Table 5
Polymerase identity and concentration	E	Table 5
Buffer/kit identity and manufacturer	E	NA
Exact chemical constitution of the buffer	D	NA
Additives (SYBR Green I, DMSO, etc.)	E	Materials and Methods
Manufacturer of plates/tubes and catalog number	D	NA
Complete thermocycling parameters	E	Materials and Methods
Reaction setup (manual/robotic)	D	NA
Manufacturer of qPCR instrument	E	Materials and Methods
qPCR VALIDATION		
Evidence of optimization (from gradients)	D	Figure 2
Specificity (gel, sequence, melt, or digest)	E	Figure 2, 4, 5, 7
For SYBR Green I, Cq of the NTC	E	Figure 2, 4, 5, 7, Tables S2, S3, S4
Standard curves with slope and y-intercept	E	Figure 3, 6
PCR efficiency calculated from slope	E	Figure 3, 6
Confidence interval for PCR efficiency or standard error	D	ND
r ² of standard curve	E	Figure 3, 6
Linear dynamic range	E	Figure 3, 6
Cq variation at lower limit	E	Table S5
Confidence intervals throughout range	D	ND
Evidence for limit of detection	E	Table S5
If multiplex, efficiency and LOD of each assay	E	Results

NA = Not applicable; ND = Not determined; E = Essential; D = Desired

* Testing for DNA contamination with a no-reverse-transcription assay is essential when RNA is first extracted. After DNA-free status has been verified, the control is recommended but no longer required.

** Reporting probe sequences is strongly encouraged to support transparency and reproducibility, but it cannot be required because some commercial predesigned assays do not provide this information.

Table S1. MIQE checklist compliance (*continued*).

Item to check	Importance	Checklist
DATA ANALYSIS		
qPCR analysis program (source, version)	E	Materials and Methods
Cq method determination	E	Materials and Methods
Outlier identification and disposition	E	ND
Results of NTCs	E	Figure 2, 4, 5, 7
Justification of number and choice of reference genes	E	NA
Description of normalization method	E	NA
Number and concordance of biological replicates	D	Table 6, Table S4
Number and stage (RT or qPCR) of technical replicates	E	Tables S2 and S3
Repeatability (intra-assay variation)	E	Results
Reproducibility (inter-assay variation, %CV)	D	ND
Power analysis	D	ND
Statistical methods for result significance	E	ND
Software (source, version)	E	Materials and Methods
Cq or raw data submission using RDML	D	NA

NA = Not applicable; ND = Not determined; E = Essential; D = Desired

* Testing for DNA contamination with a no-reverse-transcription assay is essential when RNA is first extracted. After DNA-free status has been verified, the control is recommended but no longer required.

** Reporting probe sequences is strongly encouraged to support transparency and reproducibility, but it cannot be required because some commercial predesigned assays do not provide this information.

Table S2. Cq values and melting temperatures (T_m) for technical replicates of the species-specific simplex assay.

Simplex assay 1000nM					
Fluor	Type	Identifier	Replicate #	Threshold Cycle (Cq)	Melting temperatures (T_m)
SYBR	Pos Ctrl	LLP	1	16.15	77.50
SYBR	Pos Ctrl	LLP	2	16.36	78.00
SYBR	Pos Ctrl	LLP	3	16.22	78.00
SYBR	Pos Ctrl	SLP	1	17.88	69.50
SYBR	Pos Ctrl	SLP	2	17.82	69.50
SYBR	Pos Ctrl	SLP	3	18.06	69.50
SYBR	NTC	WTR	1	N/A	N/A
SYBR	NTC	WTR	2	N/A	N/A

Table S3. Cq values and melting temperatures (T_m) for technical replicates of the species-specific multiplex assay.

Multiplex assay					
Fluor	Type	Identifier	Replicate #	Threshold cycle (Cq)	Melting temperatures (T_m)
SYBR	Pos Ctrl	LLP	1	21.25	77.50
SYBR	Pos Ctrl	LLP	2	20.97	77.50
SYBR	Pos Ctrl	LLP	3	21.48	78.00
SYBR	Pos Ctrl	SLP	1	28.56	69.00
SYBR	Pos Ctrl	SLP	2	28.13	69.00
SYBR	Pos Ctrl	SLP	3	28.03	69.00
SYBR	Neg Ctrl	LBP	1	N/A	N/A
SYBR	Neg Ctrl	LBP	2	N/A	N/A
SYBR	Neg Ctrl	LBP	3	N/A	N/A
SYBR	Neg Ctrl	SFP	1	N/A	N/A
SYBR	Neg Ctrl	SFP	2	N/A	N/A
SYBR	Neg Ctrl	SFP	3	N/A	N/A
SYBR	NTC	WTR	1	N/A	N/A
SYBR	NTC	WTR	2	N/A	N/A
SYBR	NTC	WTR	3	N/A	N/A

Table S4. Cq values and the melting temperatures for the biological replicates in screening performance.

<i>P. elliottii</i> (SLP)				<i>P. palustris</i> (LLP)				<i>P. echinata</i> (SFP)				<i>P. taeda</i> (LBP)			
Location	Samples	Cq	T_m	Location	Samples	Cq	T_m	Location	Samples	Cq	T_m	Location	Samples	Cq	T_m
Florida	1	20.74	69.5	Florida	1	19.52	78.0	Arkansas	1	N/A	N/A	Florida	1	N/A	N/A
Florida	2	22.70	69.0	Florida	2	22.78	78.0	Arkansas	2	N/A	N/A	Florida	2	N/A	N/A
Florida	3	22.92	69.0	Florida	3	18.55	77.5	Arkansas	3	N/A	N/A	Florida	3	N/A	N/A
Florida	4	22.77	69.5	Florida	4	20.34	78.0	Arkansas	4	N/A	N/A	Florida	4	N/A	N/A
Florida	5	22.48	69.5	Florida	5	22.37	78.0	Arkansas	5	N/A	N/A	Florida	5	N/A	N/A
Florida	6	21.69	69.5	Florida	6	23.41	78.0	Arkansas	6	N/A	N/A	Florida	6	N/A	N/A
Florida	7	30.65	69.0	Florida	7	23.59	78.0	Mississippi	7	N/A	N/A	Florida	7	N/A	N/A
Florida	8	21.71	69.0	Florida	8	23.08	78.0	Mississippi	8	N/A	N/A	Florida	8	N/A	N/A
Georgia	9	24.23	69.0	Florida	9	20.97	78.0	Mississippi	9	N/A	N/A	Florida	9	N/A	N/A
Georgia	10	24.40	69.5	Florida	10	21.23	78.0	Mississippi	10	N/A	N/A	Florida	10	N/A	N/A
Georgia	11	23.07	69.5	Georgia	11	24.86	78.0	Mississippi	11	N/A	N/A	Georgia	11	N/A	N/A
Georgia	12	23.82	69.5	Georgia	12	23.71	78.0	Missouri	12	N/A	N/A	Georgia	12	N/A	N/A
Georgia	13	32.86	69.0	Georgia	13	23.75	78.0	Missouri	13	N/A	N/A	Georgia	13	N/A	N/A
Georgia	14	23.35	69.0	Georgia	14	24.52	78.0	Missouri	14	N/A	N/A	Georgia	14	N/A	N/A
Georgia	15	24.76	69.0	Georgia	15	24.37	78.0	Missouri	15	N/A	N/A	Georgia	15	N/A	N/A
Georgia	16	31.78	69.5	Georgia	16	24.78	78.0	Missouri	16	N/A	N/A	Georgia	16	N/A	N/A
Georgia	17	32.33	69.5	Mississippi	17	22.00	78.0	Missouri	17	N/A	N/A	Georgia	17	N/A	N/A
Georgia	18	31.71	69.5	Mississippi	18	22.99	78.0	Missouri	18	N/A	N/A	Georgia	18	N/A	N/A
Mississippi	19	19.78	69.5	Louisiana	19	16.78	78.0	Missouri	19	N/A	N/A	Georgia	19	N/A	N/A
Mississippi	20	22.53	69.0	Louisiana	20	16.85	78.0	Missouri	20	N/A	N/A	Georgia	20	N/A	N/A
Mississippi	21	24.66	69.0	Louisiana	21	17.46	78.0								
Mississippi	22	23.41	69.0	Louisiana	22	16.03	78.0								
Mean		24.93	69.25	Mean		21.54	77.98								
Standard deviation (SD)		4.05	0.26	Standard deviation (SD)		1.83	0.11								

Table S5. Cq values for technical replicates in the limit of detection (LOD) analysis.

Replication	<i>P. elliottii</i> (SLP)	<i>P. palustris</i> (LLP)
1	32.50	33.05
2	32.46	32.85
3	33.57	33.46
4	33.82	35.16
5	33.75	32.96
6	32.73	36.52
7	34.14	34.95
8	33.83	33.89
9	34.13	34.40
10	33.71	33.82
11	33.85	33.67
12	33.81	33.90
13	33.01	33.89
14	33.09	35.20
15	33.17	33.18
16	32.23	33.14
17	32.22	33.61
18	32.79	34.01
19	33.11	34.68
20	32.75	34.12
21	32.47	34.09
Mean	33.20	34.03
Standard deviation	0.64	0.90

Performance of light-frame shear wall framed with Japanese lumber

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Abstract: Recently, Hinoki and Sugi, two predominant wood species in Japan, had approval for their design values to be listed in the National Design Specification for wood-frame construction in the United States. However, their ability to meet the demands of a light-framed shear wall has not been studied. The objective of this study was to characterize the behavior of Hinoki and Sugi framed shear walls with conventional sheathing material, and compare the performance to Douglas-fir framed shear walls. Twelve light-frame shear walls were constructed, and one monotonic and three cyclic tests were performed per wall type consisting of Douglas-fir, Hinoki, and Sugi. Hinoki showed similar properties to that of Douglas-fir, while Sugi observed a lower lateral resistance and stiffness compared to the Douglas-fir. All walls in this study met seismic equivalent parameters, which allow the wall configurations used in this study to be compared to other wood-frame sheathed wall types.

Keywords: Shear wall; Light-frame; Seismic performance; Hysteresis; EEEP.

Introduction

Wood light-frame construction, referred to as wood-framed construction from here on, represents approximately 94% of residential houses constructed in the United States in 2021 (Fu 2025). An important aspect of wood-frame construction is the design and performance of a shear wall, which is an integral component of the lateral force resisting system (LFRS). Shear walls are typically framed with dimension lumber with structural sheathing (such as oriented strand board and plywood) fastened to the frame using nails and other mechanical fasteners. To regulate the performance of these buildings, the International

Residential Code (IRC) (International Code Council 2020) provides prescriptive building designs to ensure that minimum performance standards are met. However, to obtain unit shear capacities and design values of these shear walls, the “Special Design Provisions for Wind and Seismic” (SDPWS) standard, published by American Wood Council, is used. Because Sugi (*Cryptomeria japonica*) and Hinoki (*Chamaecyparis obtusa*) are widely used in Japanese construction and originate from a region with seismic demands comparable to those in the US, evaluating their performance within US light-frame design standards provides important context for their potential use in domestic construction.

Planted forests account for approximately 40% of Japan’s total forest area (Ministry of Agriculture, Forestry and Fisheries 2023). Among these, Sugi is the most extensively planted

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species, followed by Hinoki, reflecting their long-standing importance in Japan's timber supply. Following World War II, Japan faced a critical need for construction materials, which led to large-scale harvesting, the rapid establishment of reforestation programs, and widespread planting of Sugi and Hinoki to support the nation's rebuilding efforts. Today, forestry remains a significant industry, with a total output of 580.7 billion yen in 2022 (approximately 3.464 billion USD). About 80% of low-rise residential buildings in Japan are constructed from wood, underscoring the continued dominant use of these species in the built environment. Japan, like the Pacific Coast of the United States, is located in a high seismic risk zone, further emphasizing the relevance of understanding the structural performance of Sugi and Hinoki for modern construction practices.

Given the extensive use of Sugi and Hinoki in Japan's planted forests and their prominence in Japanese residential construction, it is important to understand how these species perform when used in structural systems common in the United States. In typical shear wall framing, any species listed in the American Wood Council (AWC) National Design Specification (NDS) Supplement (AWC 2024) can be used. With the recent introduction of Sugi and Hinoki into the US construction market through their acceptance into the NDS (American Wood Council 2024), there is a need to characterize their performance as framing materials in shear wall applications. Furthermore, comparing their behavior with that of Douglas-fir (*Pseudotsuga menziesii*), a commonly used US framing species, can provide essential information that may support broader adoption of Sugi (*Cryptomeria japonica*) and Hinoki (*Chamaecyparis obtusa*) within the US construction sector.

During an earthquake, ground shaking and ground failure are the primary contributors to loss of life and property. This was evident during the Northridge earthquake on January 17, 1994. The Los Angeles metropolitan area experienced significant damage to wood-frame construction, which was categorized into three main areas: casualties, property loss, and loss of functionality (Krawinkler et al. 2001). In response, the CUREE-Caltech Woodframe Project was launched to combine research and practical implementation to enhance the seismic resistance of wood-frame buildings. Through the Consortium of Universities for Research in Earthquake Engineering (CUREE), a standard testing protocol was created for component tests. Representative loading histories were developed based on the seismic demands placed on wood buildings during California earthquakes. This included the development of a protocol for displacement-controlled quasi-static reverse cyclic testing.

In low-rise wood structures, shear walls act as the primary LFRS. These consist of dimensional lumber framing combined with plywood, oriented strand board (OSB), or other sheathing materials capable of resisting shear forces acting on the assembly. Shear walls are primarily connected using nails; the sheathing resists lateral forces, while the framing supports vertical gravity loads. This combination contributes to overall rigidity and limits deflection. Van de Lindt (2004) highlights two characteristics that enable light-frame wood structures to perform well during moderate and severe ground shaking: their high strength-to-weight ratio and their ability to dissipate energy through the accumulation of individual fastener deformations.

Standard racking tests have been used since the 1940s to determine the shear strength of wood shear walls. These tests have undergone considerable evolution, including protocols developed under the CUREE project. Research has revealed key insights, for example, the critical influence of shear wall-to-framing connections on structural behavior (Cheung et al. 1988), and the comparable shear capacities of plywood and OSB as sheathing materials (Serrette et al. 1997). Van de Lindt (2004) provides a comprehensive review of shear wall testing and its evolution over time. Since 2004, several other studies have characterized shear wall systems. Sinha and Gupta (2009) investigated the contribution of gypsum wallboard to the overall performance of shear walls. Seaders et al. (2009) examined the performance of wood-frame shear walls under monotonic and cyclic loading, including comparisons of partially and fully anchored configurations. Way et al. (2019) investigated the performance of light-frame shear walls with weathered sheathing. Bagheri and Doudak (2020) evaluated the characteristics of shear walls with various construction details, identifying the effects of different wall types on structural performance. Key research advances on characterizing the performance of shear walls constructed with Sugi has been reported in Japan. Shear wall systems constructed with Sugi have demonstrated favorable lateral performance, including adequate strength, deformability, and energy dissipation capacity, in both traditional and contemporary timber construction in Japan (Okazaki and Watanabe 2005). However, much of the foundational experimental and analytical work on Sugi-based shear walls remains published in Japanese-language journals, technical reports, and conference proceedings (Mori et al. 2005; Inayama 2003) limiting accessibility and broader dissemination within the international research community. As a result, despite a body of knowledge on wall system performance, these findings are often underrepresented in global design frameworks

and comparative studies. Greater synthesis and translation of this literature are needed to enable more rigorous benchmarking and to facilitate the integration of Sugi-based systems into international timber engineering practice. ASTM E564, Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings (ASTM International 2018), evaluates the capacity of a typical framed wall section, using a rigid foundation and applying a static load parallel to the base at the opposite edge. Later, ASTM developed method E2126, Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings (2019), which provides a standardized cyclic loading test to assess the shear resistance of vertical components in LFRS assemblies. To demonstrate equivalency between various wood-frame shear walls, ASTM introduced D7989, Standard Practice for Demonstrating Equivalent In-Plane Lateral Seismic Performance to Wood-Frame Shear Walls Sheathed with Wood Structural Panels (2021), which includes seismic equivalency parameters (SEPs). The SEPs were developed by an International Code Council (ICC) Evaluation Service task group and are used to demonstrate seismic equivalence

for new structural systems. These are calculated using allowable stress design (ASD) values derived from SDPWS.

The objectives of this study are to

1. Characterize the performance of shear walls built with Hinoki and Sugi framing.
2. Investigate the difference in behavior between Hinoki, Sugi, and Douglas-fir framing with full-scale monotonic and cyclic lateral testing.
3. Compare the performance of each shear wall type to other wood-frame sheathed wall types.

Materials and methods

Wall specimen

Wall specimens were designed based on the 2021 International Residential Code (IRC) prescribed brace panel construction. Walls were designed at 2440×2440 mm (8 ft $\times$ 8 ft) sections, as shown in Figure 1. Fastener details for all three wall types are provided in Table 1.

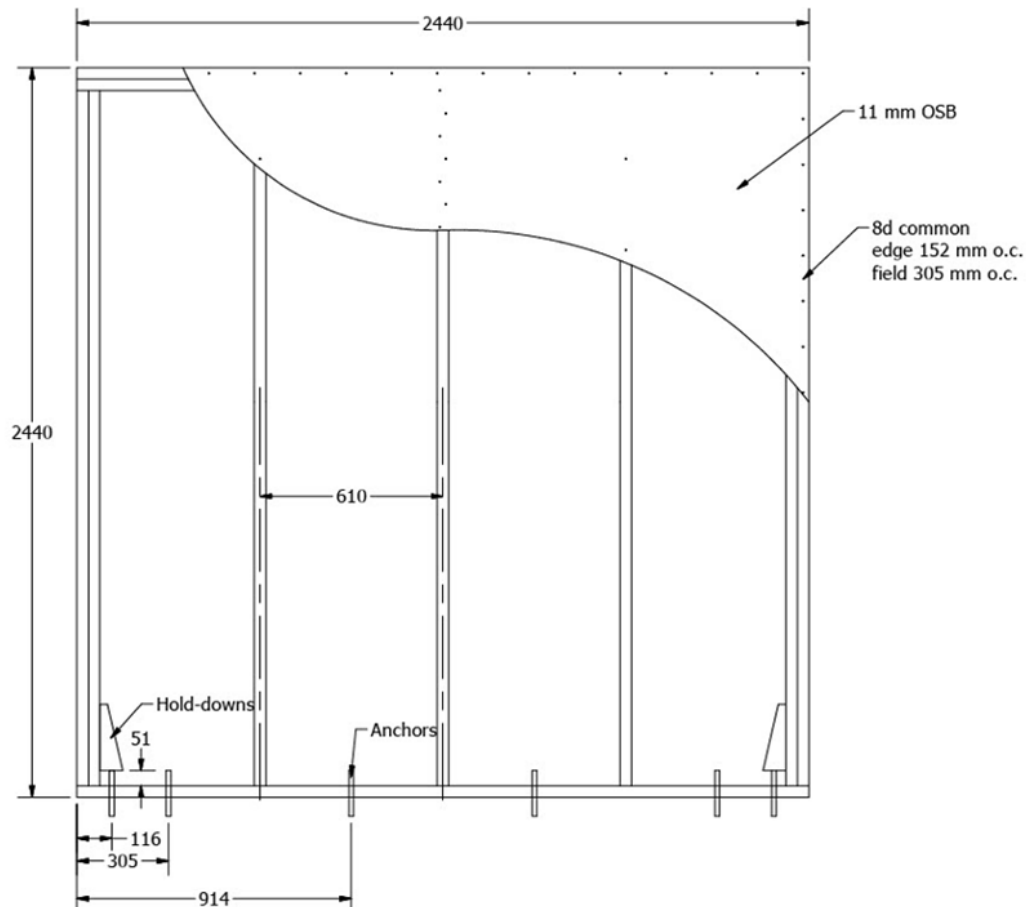


Figure 1. Schematic of wood-frame shear wall.

Table 1. Fastener details and schedule for walls tested.

Details	Type	Description
Framing material	No. 2 graded, 38 × 140 mm	Douglas-fir, Sugi, Hinoki
Sheathing material	OSB	11.1 mm thickness, 24/16 rated
Fasteners	8d common – 3.32 mm dia; 63.5 mm length	Sheathing
	10d box – 3.32 mm dia; 76 mm length	Stud to Stud
	16d common – 3.33 mm dia; 82.5 mm length	Framing
Fastener spacing	152 mm	Edges
	304 mm	Field

For each species used, the board size and grade was 38 × 140mm (2×6) graded No. 2 boards. Studs were spaced at 610 mm (24 in.) on center; these were fastened to the top and bottom plates with 2-16d common end nails (3.33 mm dia × 82.55 mm length; 0.131 in dia × 3.25 in length). Double studs and double plates were fastened together using 2-10d box nails (3.32 mm dia × 76 mm length; 0.131 in dia × 3 in length) every 406 mm (16 in.). Two structural OSB boards (1220 mm width × 2440 mm length × 11.1 mm thickness; and 48 in width × 96 in length × 7/16 in thick) were used with a span rating of 24/16. Sheathing was fastened using 8d common nails at 152 mm (6 in.) on center around the edges of the panel and at 305 mm (12 in.) in the panel field. All nails were smooth shank and were fastened using a Bostitch pneumatic nail gun. Four 19 mm (0.75 in.) anchor bolts (grade 5) were placed along the sill plate with Simpson bearing plates BP 5/8-2 (51 mm width × 51 mm length x 4.76 mm thick; 2 in width × 2 in width × 0.1875 in thick). Two Simpson HDU hold-downs (221 mm; 8.6875 in.) were attached 2 inches above the sill plate on the inside face of the end studs and fastened using Strong-Drive SDS Heavy Duty Connector screws (6.35 mm dia × 63.5 mm length; 0.25 in dia × 2.5 in length) with the use of a Hilti impact driver.

Load frame and test equipment

All tests were conducted at the Gene D. Knudson Wood Engineering Laboratory at Oregon State University. The loading frame used for monotonic and cyclic testing is shown in Figure 2.

The bases of the walls were bolted to a steel beam that was attached to a strong floor to simulate a fixed foundation. Attached to the strong wall was a 490 kN (110 kip) servo controlled hydraulic actuator with 254 mm (10 in.) total of stroke used to apply force to the top of the wall. To allow vertical movement of the actuator while relieving its gravity load on the wall, it was supported by a vertical 102 mm (4 in.) hydraulic cylinder.



Figure 2. Test set up.

The supporting piston utilizes a 111.2 kN (25 kip) load cell for force measurements. The steel C-channel was attached to the top of the wall with four 22 mm (0.87 in.) short-threaded rods (ASTM F1554 grade 36) across the top of the wall. To prevent out-of-plane movement, two steel poles were laterally braced from the strong wall to the C-channel.

Testing protocol

Monotonic tests

Monotonic tests were based on the ASTM E564-06 (ASTM International 2018) test protocol, which was used to determine static load capacity and deflection of framed wall sections. This protocol required the ultimate load to be reached in no less than 5 minutes. Loading rate of monotonic test was 0.21 mm/sec (0.008 in./sec). Using this testing method, a monotonic deformation capacity (Δ_m) was derived; this deformation capacity was equal to the distance that a wall deflects for the first time below 80% maximum load.

Cyclic tests

Cyclic tests followed ASTM E2126-19 (ASTM International 2019) Test Method C, also known as the CUREE Basic Loading Protocol as shown in Figure 3. This protocol is designed to simulate the deformation demands that vertical elements of a lateral force resisting system (LFRS) may experience under lateral loads (such as wind or seismic forces). The CUREE protocol required a reference displacement (Δ_{ref}), which is 60% of monotonic deformation capacity ($\Delta_{ref} = 0.6 \cdot \Delta_m$) to build a series of increasing deformation cycles.

This loading protocol consisted of three types of cycles: initial cycles, primary cycles, and trailing cycles. Loading began with six initial cycles at small amplitudes equal to 5% of the reference displacement. These were followed by the primary cycles, with the first primary cycle applied at 7.5% of the reference displacement. Subsequent primary cycles increased in amplitude, progressing up to 2 times the reference displacement. After each primary cycle, a trailing cycle was applied at an amplitude equal to 75% of the displacement of the preceding primary cycle. In the full protocol, primary cycles continued to increase in increments of 0.5 times the reference displacement, beyond 2 times the reference displacement; however, in this study, loading was only required up to 2 times the reference displacement.

Test matrix

Testing consisted of three different species (Douglas-fir, Hinoki, and Sugi) under two different loading conditions (monotonic and cyclic). Three walls were tested in monotonic, and nine walls were tested in cyclic loading. One monotonic test was required to gain the reference displacement for each wall type, while the number of tests for cyclic was in accordance with ASTM E2126 (ASTM International 2019). In accordance with the standard, only two cyclic tests are required for a given wall configuration; however, the standard also specifies that a third test must be conducted when the difference in peak load between the first two tests exceeds 10%. For this reason, a third wall was tested in this study. Table 2 shows the complete listing of the testing and the labeling used for each wall type.

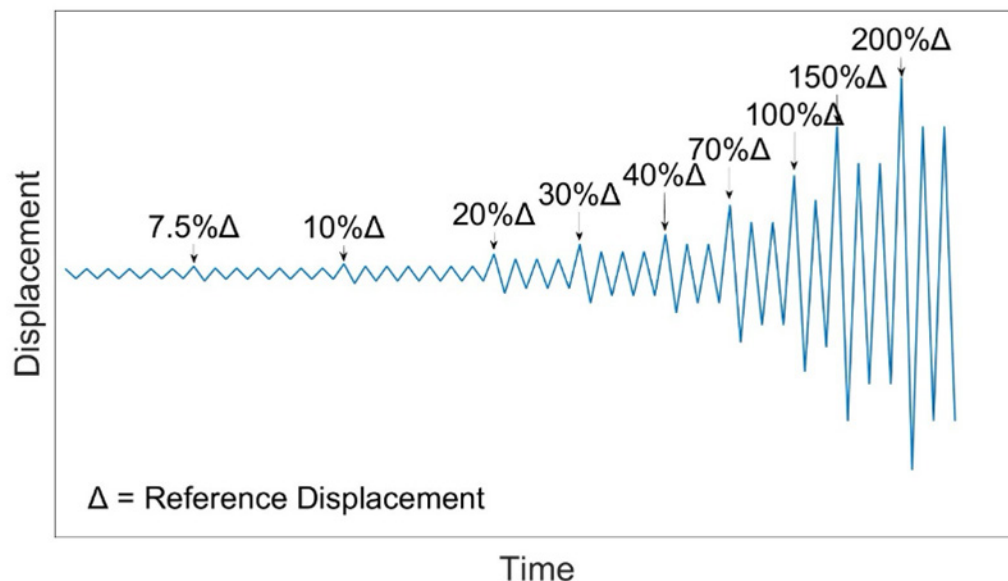


Figure 3. Cyclic loading protocol.

Table 2. Test matrix.

Wall type	Monotonic	Cyclic
Douglas-fir	1	3
Hinoki	1	3
Sugi	1	3

Data Acquisition

Multiple measurements were made during testing to help characterize the performance of the walls. These measurements included force applied along the top-plate, displacement of the top-plate, uplift that occurred at each end stud, and slip that occurred at the sill plate. Force applied was measured using an Interface High-Capacity Fatigue-Rated Universal Low-profile load cell. The load cell's capacity is 450 kN (100 Kip); in use of hysteresis, its accuracy was $\pm 0.06\%$ full scale. Displacement of the top of the wall was measured by an internal displacement sensor in the actuator, as well as a string potentiometer attached to the top plate and the opposite end of the wall. For calculations, the string potentiometer displacement was used to reduce error in displacement that could have been caused by slipping between the top-plate and the attached steel beam. The string potentiometer was a Micro-epsilon wire sensor, WDS-P60, with a measuring range of 300 mm (11.8 in.), with a linear accuracy of $\pm 0.1\%$ full scale output. To measure the uplift of the wall, one linear variable differential transformer (LVDT) was placed at each end of the wall approximately 127 mm (5 in.) from the ground. The LVDT had a range of ± 12.7 mm (0.5 in.) with an accuracy of 0.25% of its full stroke. To measure the slip of the sill plate, a LVDT was used with a range of ± 7.62 mm (0.3 in.), with an accuracy of 0.25% of its full stroke.

Load-displacement curves

To present the structural behavior of the shear walls, a load displacement curve or load envelope is commonly employed. For monotonic tests, the acquired load was directly plotted from the observed load-displacement data. From the cyclic tests, hysteresis was acquired. To gain a more representative figure and one that could be compared to that of monotonic testing, an envelope curve was derived. The envelope curve was made up of maximum forces reached at every primary cycle in both directions. From these, we gained a positive and negative envelope curve that could be averaged to show a representative load-displacement curve that can be compared with monotonic test. From the average envelope, an equivalent energy-plastic (EEEP) curve can be calculated.

Table 3. EEEP parameters.

Parameters	Units	Description
P_{yield}	kN	Yield load
E	kN•mm	Energy under back bone curve up to failure
P_{peak}	kN	Maximum load in a given envelope
Δ_e	mm	Displacement of specimen at $0.4 \cdot P_{peak}$
K_e	kN/mm	$0.4 \cdot P_{peak} / \Delta_e$; Initial stiffness

EEEP curve

All parameters of the EEEP curve were derived from the envelope curve and are listed in Table 3, and were calculated as per ASTM D2126 (2019)

The initial portion of the EEEP curve is the initial stiffness of the wall. This initial stiffness (K_0) is equal to 40% of the maximum load over the displacement that it took to reach said load. Due to the non-linear behavior of lateral loading in wood shear walls, an alternative to traditional ductility was determined. The alternative was the absorbed energy (E), calculated as the area enclosed under the monotonic and envelope curves, up to the deformation capacity.

Yield load is calculated as per Eq. 1,

$$P_{yield} = \left(\Delta_u^2 - \sqrt{\Delta_u^2 - \frac{2E}{K_e}} \right) K_e \quad [1]$$

Where, Δ_u is the ultimate displacement, E is the energy under the curve up to failure; K_e is the initial stiffness.

Post-testing

After testing, moisture content for each wall was measured using two different methods, selected based on the species of wood. For the Douglas-fir walls, a pin-type moisture meter was used, with four readings taken at different locations on each wall and then averaged. For the Hinoki and Sugi walls, moisture content was determined using the oven-dry method. Small sections of framing were removed from each wall and placed in a dry oven set to $103^\circ\text{C} \pm 2^\circ\text{C}$ for 24 hours and weighed before and after drying.

Results and Discussion

Monotonic testing

Load-displacement curves from monotonic testing for the three wall types are shown in Figure 4. Peak load of the three wall

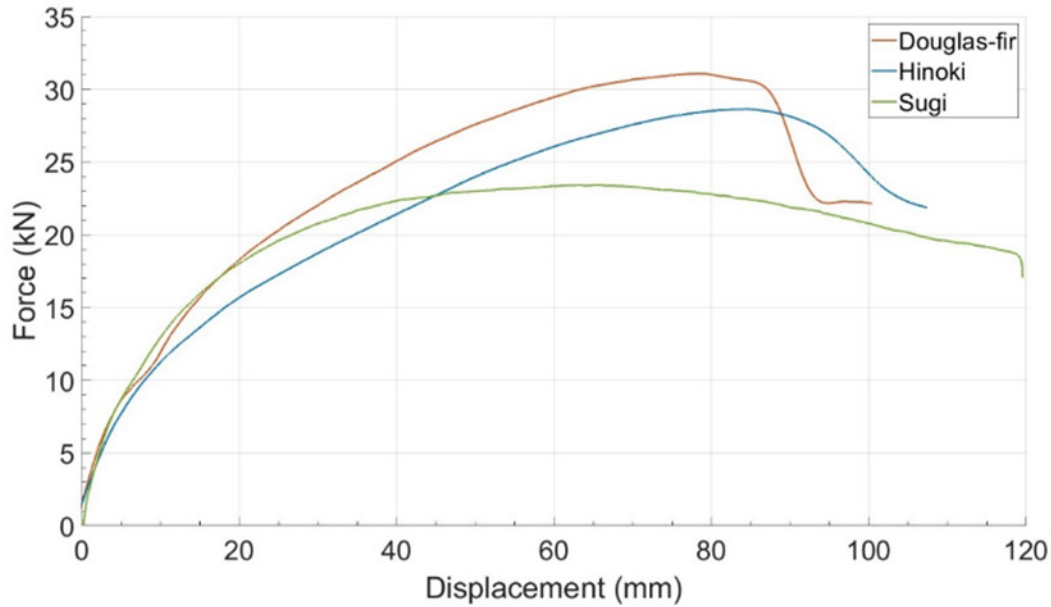


Figure 4. Load -displacement response from monotonic tests.

types were as follows: 31.85 kN (7160 lbf) for the Douglas-fir wall; 29.51 kN (6634 lbf) for the Hinoki wall; and 23.00 kN (5170 lbf) for the Sugi wall. The main contributor to peak force is the sheathing type and amount of panel to stud nails. However, a 28% decrease in strength in comparison with the Sugi to Douglas-fir [Specific Gravity (SG) = 0.50] was observed and can be attributed to the lower density of Sugi (SG = 0.36). The SG of Sugi is 28% lower than that of Douglas-fir. This also can account for its lower stiffness in comparison to both Douglas-fir and Hinoki (SG = 0.45).

The ultimate displacement is the point of the linear curve where the force reaches 80% of the peak force for the first time after peak load had been reached. Douglas-fir reached its ultimate displacement at 82.7 mm (3.25 in.), while Hinoki reached it at 101 mm (3.98 in.), and Sugi at 110 mm (4.33 in.). These values were used to calculate the Δ_{ref} . Results from the Douglas-fir and Hinoki monotonic test were averaged to determine Δ_{ref} . The reference displacement values from each test were 49.5 mm (1.95 in.) for Douglas-fir and 60.5 mm (2.38 in.) for Hinoki. For cyclic testing, the two were averaged and rounded to 56 mm (2.2 in.). During the monotonic test for Sugi, the measured Δ_{ref} was 66.14 mm (2.60 in.). However, to account for the larger ultimate displacement exhibited by Sugi and to remain within the actuator's stroke limit of ± 5 in, a reference displacement of 61 mm (2.4 in.) was selected.

The purpose of this change was to allow the Sugi specimens to have a greater displacement at the final cycles of the test and reach a failure point.

Cyclic testing

Cyclic tests were done in accordance with ASTM E2126-19 (ASTM International 2019). The hysteresis loops for all tests are shown in Figure 5. It is noted that the positive direction showed higher strength and stiffness than the negative cycle. The higher resistance observed under positive loading compared with negative loading is likely associated with the cumulative effects of reversed cyclic loading. Under repeated load reversals, localized damage mechanisms such as embedment, slip, gap evolution, and progressive inelastic deformation can accumulate, resulting in degradation of both stiffness and peak resistance. Because the response in one loading direction is influenced by damage sustained during prior excursions in the opposite direction, the lower negative-direction resistance likely reflects a more degraded structural state of the specimen rather than an intrinsic directional strength difference alone. Thus, the asymmetry in resistance is better interpreted because of cyclic damage accumulation and load-history-dependent degradation of the force-transfer mechanism.

Douglas-fir (Figure 5a) and Hinoki (Figure 5b) had similar behavior, while Sugi (Figure 5c) had a lower stiffness. The differences in stiffness between the Sugi and the other two

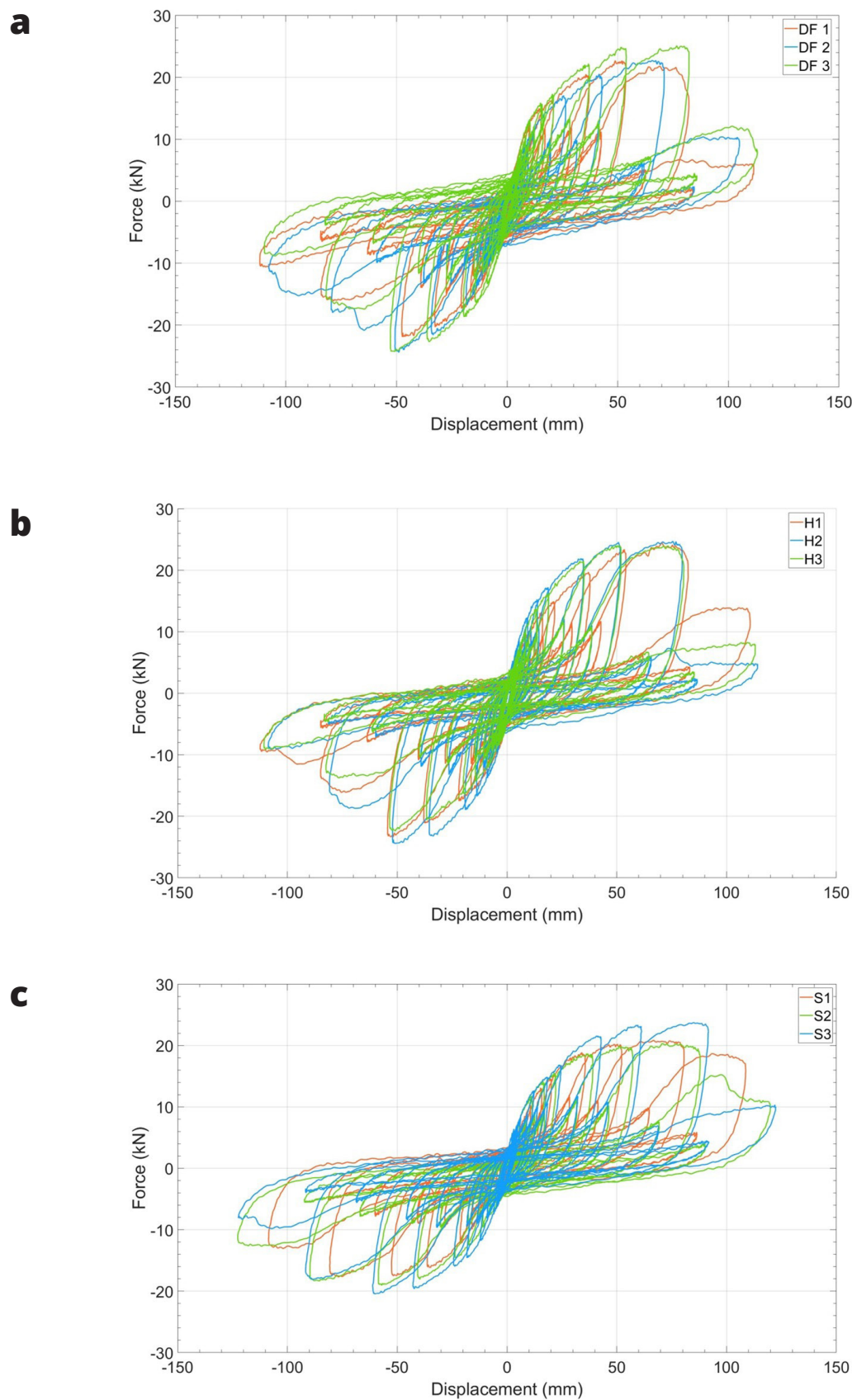


Figure 5. Typical hysteresis of wall tested (a) Douglas-fir; (b) Hinoki; and (c) Sugi.

species can be attributed to the differences in specific gravity of the framing material.

While Sugi demonstrated a larger displacement capacity in monotonic testing, an initial attempt was made to use the same reference displacement as Douglas-fir and Hinoki for the first Sugi cyclic test. However, failure, defined in ASTM E2126 (ASTM International 2019) as the point where the lateral resistance drops to 80% of the peak load, was not reached in this test. As a result, a larger reference displacement was adopted for the subsequent two cyclic tests to ensure that failure was achieved. The initial Sugi cyclic test was still included in the species comparison, and for this specimen the ultimate displacement was taken as the last point on the envelope curve rather than an interpolated point corresponding to 80% of peak load, as permitted by ASTM E2126. The average envelope curve, shown in Figure 6, represents the mean backbone response derived from both the positive and negative loading segments of the hysteresis loops for each species. Further presented in Figure 6 are the monotonic curves for direct comparison of each wall species.

Based on average envelopes from across tested species, the shear capacity of the Douglas-fir wall type was 9.56 kN/m (655 lb/ft), while the Hinoki wall type was 9.76 kN/m (669 lb/ft), and the Sugi wall type was 8.69 kN/mm (595 lb/ft). Douglas-fir

and Hinoki walls reached their maximum resistance at lower drift levels, while the Sugi wall exhibited its peak force at a noticeably larger drift. This behavior is reflected in the envelope curves shown in Figure 6. In general, direct comparison between monotonic and cyclic envelope responses reveals that the monotonic tests exhibited higher peak resistance and greater initial stiffness. This difference arises from the absence of load reversals in monotonic loading, which limits the accumulation of inelastic damage. Under cyclic loading, repeated reversals induced progressive degradation mechanisms, including localized crushing, interfacial slip, and gap evolution, leading to stiffness reduction, pinching behavior, and diminished peak resistance. As a result, the cyclic envelope curve reflects a degraded response that incorporates cumulative damage effects, whereas the monotonic response represents an upper-bound capacity under a single loading path.

Results from monotonic tests can be compared to the Seaders et al. (2009) fully anchored monotonic and cyclic results, since they used a construction geometry exactly similar to this study, with the only difference being Douglas-fir framing and 11-mm thick OSB as sheathing. The average peak force of the monotonic walls for this study was 15% greater than that of Seaders et al. (2009), while the cyclic average response was 2.5% greater. The EEEP presented in Figure 7 were derived

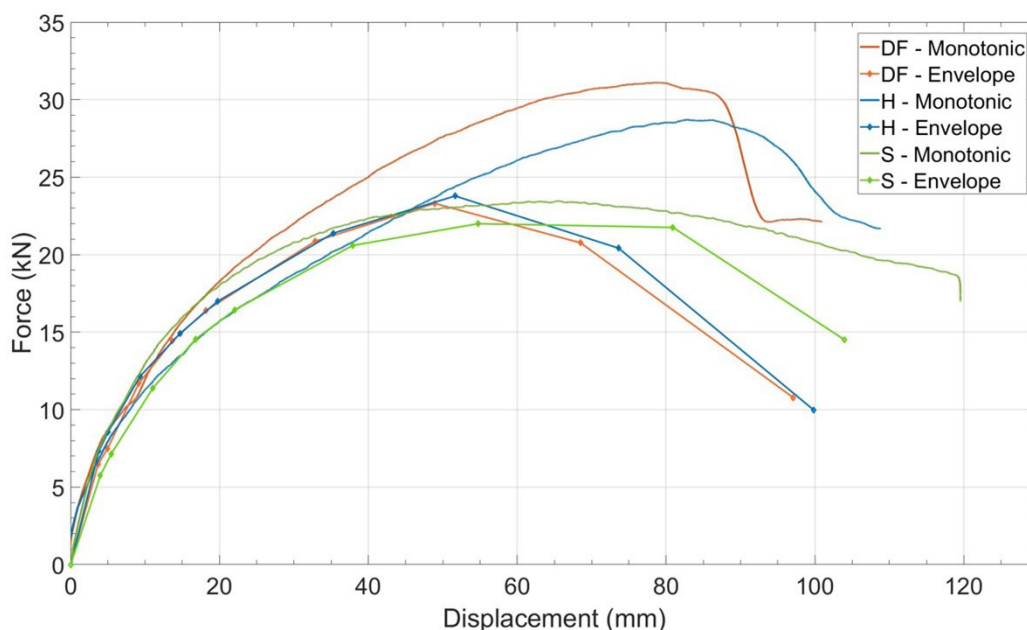


Figure 6. Average envelope curves for the tested wall types.

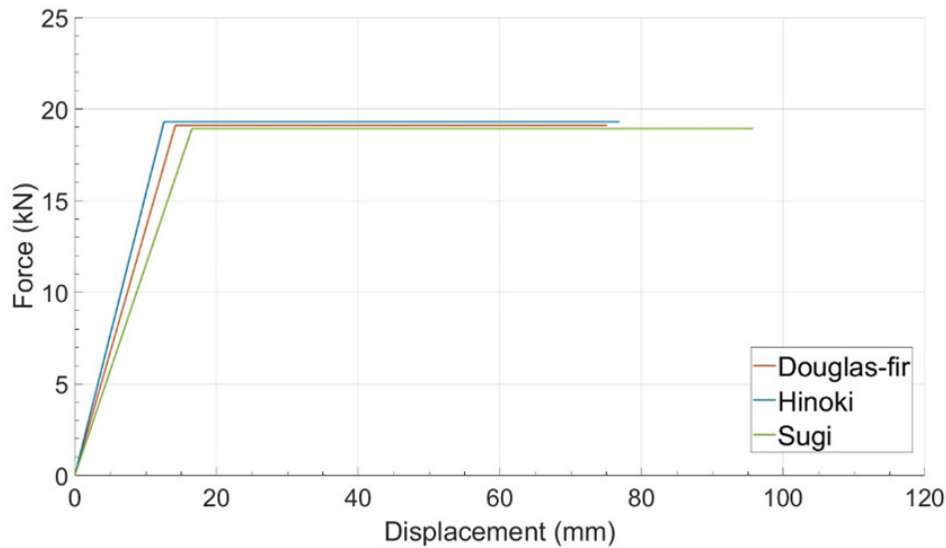


Figure 7. Average equivalent energy elastic plastic (EEEE) curves for the wall types tested.

from the average EEEP parameter calculations that were made for the average envelope of each cyclic test. All the EEEP parameters, along with their associated variability, are presented in Table 4.

Using Douglas-fir as the reference, Hinoki had a 14% increase in initial stiffness, while Sugi had a 14% decrease in initial stiffness. Douglas-fir average ultimate displacement was 73.6 mm (29 in.). The ultimate displacement of Hinoki was 4.5% higher than Douglas-fir, while for Sugi, it was 30% higher than Douglas-fir (Table 4).

Seismic equivalence parameters (SEP)

To provide a method for evaluating alternative shear wall systems, ASTM D7989-21 establishes criteria for determining equivalency to nailed, wood-frame, wood structural panel shear walls (ASTM International 2021). The average envelope

curve of each specimen was used to calculate the SEPs. A shear wall configuration is considered seismically equivalent when each SEP is satisfied and when vertical load-supporting elements remain intact and capable of supporting gravity loads. The SEPs consist of component overstrength, drift capacity, and ductility. These parameters were calculated for each test and then averaged by wall type, and they require an allowable stress design (ASD) allowable shear capacity for each wall configuration. The ASD allowable shear capacity was calculated by dividing the nominal shear capacity of the wall configuration by 2.8 (American Wood Council 2020).

The nominal shear capacity was derived from Table 4.3A of the SDPWS and adjusted for specific gravity. The nominal unit shear capacity of the Douglas-fir walls was 9.78 kN/m (670 plf). For the Hinoki and Sugi walls, it was 9.30 kN/m (637 plf) and 8.41 kN/m (576 plf), respectively. The reductions in nominal

Table 4. Average EEEP parameters, along with their coefficient of variation (COV) for the wall types tested.

EEEE parameters	Douglas-fir	COV (%)	Hinoki	COV (%)	Sugi	COV (%)
P_{yield}	19.10	6.00	19.26	4.76	18.93	5.40
Δ_u (mm)	75.11	2.98	76.91	2.68	95.79	1.91
Energy (kN*mm)	1299	6.80	1360	5.17	1655	4.08
K_e (kN/mm)	1.36	14.92	1.55	14.77	1.17	20.19
$0.4*P_{peak}$ (kN)	9.32	5.54	9.52	3.05	8.87	7.28
Δ_e (mm)	6.93	9.53	6.20	11.41	7.75	18.97
Ductility ratio (m)	10.72	11.1	12.40	8.33	12.6	10.42

shear capacity for Hinoki and Sugi resulted from the specific-gravity adjustment factor noted in Table 4.3A of the SDPWS. These values were used to calculate component overstrength and ductility. The two values required for these calculations are the allowable design load (P_{ASD}) and the displacement corresponding to the allowable design load (Δ_{ASD}). Overstrength is calculated as the ratio of observed average peak load to allowable design load (P_{ASD}); drift capacity is the measure of the ultimate displacement reached of the wall; and ductility is the ratio of average ultimate displacement observed to Δ_{ASD} .

Table 5 results were derived from the average of the three cyclic wall tests per wall type. Each of the three SEPs were above prescribed target values outlined in ASTM 7989-21. Of the three SEPs, overstrength contains a lower and upper limit of 2.5 and 5.0, respectively. The upper limit exists to prevent any given component from overloading the surrounding structural elements in an actual earthquake. Each wall type met the requirements for overstrength. Furthermore, each wall type met the minimum requirement for ductility being greater than or equal to 11 and the ultimate drift being greater than or equal to 2.8% of the wall height. All wall types in this study met SEP requirements to be able to compare the results to other wall wood-based sheathed wall types in future studies.

Observed Failure Modes

Common failure of the walls was primarily fastener connection failures to the sheathing.

Fasteners were found to be either bent and pulled out of the sheathing, which occurred along the center stud of the wall or along the top and bottom of the sheathing. At the outer edges, fasteners were bent or the nail head was pulling out. Along the center stud where two sheathing panels met, nails frequently tore out the side of the OSB. This occurred because the nails were placed only 9.5 mm from the panel edge at the panel joint, which reduced the available bearing area and increased the likelihood of edge breakout. This type of behavior is a common observation in shear wall tests constructed with a single center stud (Sinha and Gupta 2009). The inner studs located at the center of the sheathing did not experience fastener failure, as the studs moved together with the panels during cyclic loading. A few walls, however, failed at the stud-to-stud connection, as shown in Figure 8.

Conclusions

Performance of shear walls framed with Sugi and Hinoki were characterized through a series of monotonic and reverse cyclic quasi-static tests. Furthermore, the performance was compared

Table 5. Seismic equivalence parameters for the wall type tested.

Species	Seismic equivalence parameters					
	Overstrength	COV (%)	Drift capacity	COV (%)	Ductility	COV (%)
DF	2.7	5.5	3.0	1.7	12.3	20.5
Hinoki	2.9	3.0	3.2	2.7	16.6	12.8
Sugi	3.0	7.2	3.9	1.9	17.4	24.5



Figure 8. Observed failure types: (a) nail pull-out; (b) nail withdrawal; (c) stud-to-stud failure.

to a standard Douglas-fir shear-wall system. Peak load of the three wall types was 31.85 kN for the Douglas-fir wall; 29.51 kN for the Hinoki wall; and 23.00 kN for the Sugi wall. The initial stiffness for Hinoki was 14% higher than that of Douglas-fir framed walls, while Sugi frame walls exhibited a 30% lower initial stiffness than Douglas-fir framed walls. Within the experimental program, Hinoki walls performed similarly to Douglas-fir in terms of peak load, while Sugi walls exhibited lower peak load, initial displacement, and yield strength than Douglas-fir and Hinoki.

All walls in this study met seismic equivalence parameters defined within ASTM D7989 (ASTM International 2021). The main failure across all walls were tested occurred between the sheathing and the center stud. The distance between the nail and the edge of the sheathing at the center created a weak point in testing, exhibiting both nail head pull-through and a significant amount of nails withdrawal.

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Biological durability of preservative treated cross-laminated timber (CLT) made with southern pine lumber

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Abstract: Cross-laminated timber (CLT) is an engineered wood product that is being increasingly utilized in structural applications. However, its susceptibility to fungal decay and termite attack raises concerns about long-term durability, particularly in high-moisture and insect-prone environments. This study evaluates the biological durability of CLT fabricated with southern pine and treated post-layup with two copper-based preservatives: copper azole (CA-C) and micronized copper azole (MCA) under laboratory conditions. Fungal decay resistance was assessed using brown-rot (*Serpula lacrymans*) and white-rot (*Irpex lacteus*) fungi, while termite resistance was evaluated through exposure to *Reticulitermes* spp. Results showed that preservative treatments significantly reduced fungal decay and termite damage compared to untreated controls, although treatment effectiveness varied with CLT configuration (3-ply or 5-ply). While both preservatives effectively mitigated decay in 3-ply panels, treated 5-ply panels exhibited comparable mass loss to untreated counterparts. Termite resistance tests demonstrated that MCA and CA-C treated CLT samples exhibited significantly lower mass loss and higher termite mortality compared to the untreated controls, with both preservatives providing comparable protection.

Keywords: Copper-based preservatives; Mass timber; Termites; Wood decay.

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Introduction

Decay fungi and termite activity present severe hazards that can cause structural weakness in wood materials and ultimately jeopardize the integrity of wooden structures (DeGroot 1976; Garbacz et al. 2020). In response to these issues, many wood preservation technologies have been developed to increase durability against biological deterioration and the service life of timber products (Loferski 1999; Archer and Lebow 2006; Calovi et al. 2024; Ong et al. 2025). Chemical treatment with preservatives like copper azole (CA-C) and micronized copper azole (MCA) are widely utilized for wood used in ground contact due to their effectiveness in preventing fungal degradation and termite attack (Verma et al. 2009; Schultz and Nicholas 2010). Copper-based preservatives protect wood by disrupting cellular processes (i.e., enzyme inhibition, nutrient transport interruption, protein denaturation, etc.) in addition to acting as a toxic deterrent to termite feeding (Civardi et al. 2015; Ohno et al. 2020).

While the efficacy of CA-C and MCA treatments has been widely researched within the context of traditional lumber, their effectiveness when applied to engineered wood products, such as cross-laminated timber (CLT), is in early stages of development and requires more investigation (Lim et al. 2020; Taylor et al. 2023; Ayanleye et al. 2023; Quin et al. 2024, 2025). This is in part because CLT presents added difficulties for applying available wood preservation strategies due to its large size, laminated structure, and adhesive bonds (Lim et al. 2020). Additionally, the penetration depths required to adequately protect CLT from biological agents are not well defined (Wang et al. 2018). The timing of preservative treatment—whether applied before or after panel assembly—is a critical factor influencing the durability, adhesive performance, and treatment penetration of CLT. Pre-treating individual laminates before bonding allows for deeper preservative penetration and more uniform distribution, enhancing resistance to decay fungi and termites (VanAcker et al. 2023). This approach aligns with traditional wood preservation practices, where pressure treatment effectively protects solid wood products. However, pretreatment can interfere with adhesive bonding by introducing chemical residues or altering surface wettability (Lorenz and Frihart 2006). Pre-treatment also generates more preservative-treated waste, as panels typically require additional sawing and sanding post layup. Post-assembly treatment ensures optimal bonding conditions before preservative application, reducing adhesive compatibility issues. However, this method often results in limited preservative penetration, especially in

thicker panels, such as 5-ply vs. 3-ply (Ayanleye et al. 2023). Additionally, preservative retention may be inconsistent, with outer layers receiving greater protection than interior sections (Walker et al. 2006).

Biological durability is a key factor influencing the performance, longevity, and useful service-life of wood products, including engineered wood systems like CLT. Cross-laminated timber has gained widespread adoption due to its strength, versatility, and sustainability (Syahirah et al., 2025). However, its vulnerability to degradation by fungi and termites can limit its application in environments with high moisture or pest exposure. This study evaluates the biological durability of southern pine CLT treated with CA-C and MCA post-layup against the white-rot fungus *Irpex lacteus* (Fr.) and the brown-rot fungus *Serpula lacrymans* (Wulf. ex Fr.). *Irpex lacteus*, a white-rot fungus, is known for its ability to degrade both lignin and cellulose, causing significant structural damage, while *S. lacrymans* is a highly destructive brown-rot fungus with documented tolerance to copper-based preservatives and is commonly found in indoor environments (Capelari and Zadrazil 1997; Hastrup et al. 2005; Novotný et al. 2009; Watkinson and Eastwood 2012; Akgul and Akgul 2018). In addition to evaluating durability of treated CLT to fungal colonization, this study also investigates the resistance of treated CLT to subterranean termites (*Reticulitermes* spp.), which are among the most economically significant wood-destroying insects in North America (Shelton and Grace 2003). Research has demonstrated that copper-based preservatives are effective against *Reticulitermes* (Freeman and McIntyre 2008; Lin et al. 2009; Kim et al. 2010; Khademibami and Bobadilha 2022), but their performance in multi-layered engineered wood products like CLT is less understood.

Materials and methods

Cross-laminated timber fabrication

Cross-laminated timber panels were assembled using visually graded Select Structural (SS) southern pine (*Pinus* spp.) lumber ($38.1 \times 139.7 \times 2438.4$ mm) which was supplied by a local sawmill located in Mississippi, USA. The average specific gravity of the laminates for the panels was 0.49, with an average moisture content (MC) of 12%. The laminates were conditioned to the desired MC indoors under a controlled environment. A single-component polyurethane adhesive (LOCTITE PUR HB X602) was supplied by Henkel Corporation (Düsseldorf, Germany) and used in this study. Ten minutes prior to applying the adhesive, a wood primer (LOCTITE PR 3105) was sprayed

onto the surface of the wood at a spreading rate of 20 g/m². Once the primer was cured (after 10 min), the adhesive was applied at a spreading rate of 180 g/m² according to adhesive manufacturer specifications, at ambient temperature (21°C) and approximately 65% relative humidity (RH), with 1 h for pressing and curing.

Given the prevalence of 3-ply, 5-ply, and 7-ply CLT configurations in North America, we selected only 3-ply and 5-ply panels for this study due to manufacturing constraints associated with 7-ply panels. The final dimensions of the panels were 404 × 676 × 104 mm and 404 × 676 × 175 mm for the 3-ply and 5-ply panels, respectively. A computerized Dieffenbacher laboratory hydraulic hot press equipped with PressMan industrial programmable logic controllers (Dieffenbacher North America Inc., Windsor, Canada) was used to manufacture the panels (Figure 1). The press featured 86.36 × 86.36 cm platens, capable of applying a maximum pressure of 5.02 MPa. Panels were pressed at ambient temperature (21°C), and the applied pressing pressure was 0.689 MPa (100 psi), with press time of 3 h. After fabrication, the panels were stored indoors at an average temperature and relative humidity (RH) of 21°C and

65%, respectively. For more details on CLT fabrication see Quin (2023).

After gluing, CLT panels were separated into three groups: untreated controls, pressure-treated with copper azole (CA-C) or pressure-treated with micronized copper azole (MCA).

Commercially available formulations of CA-C and MCA were used as preservative treatments in this study due to their proven effectiveness against termites and decay fungi and their widespread application in the wood protection industry. The panels that received CA-C treatment were treated by Deforest Wood Preserving (Bolton, Mississippi, USA). The composition of the CA-C treating solution on a 100% active basis was as follows: 96.1% copper, 1.95% propiconazole and 1.95% tebuconazole. The panels that were impregnated with MCA were treated at Koppers Performance Chemicals (Griffin, Georgia, USA). The composition of the MCA treating solution on a 100% active basis consists of 96.1% copper and 3.9% azoles as tebuconazole (AWPA 2023a). Both sets of panels were treated with a target retention of 2.4 kg/m³, which is the retention level specified by AWPA U1-23 for UC4A



Figure 1. Panels in Dieffenbacher laboratory hydraulic press.

(ground contact or freshwater) applications (AWPA 2023b). Both facilities confirmed that the target retention was achieved. After treatment, CLT panels were cut into test specimen sizes of 5.08×2.54 cm (W $\times$ T) samples for both 3-ply and 5-ply configurations. The test specimens were then conditioned (4 weeks) to equilibrium moisture content ($\sim 17\%$) before testing.

Fungal Decay Resistance

Modifications to the AWPA E10-22 (AWPA 2023c) standard soil block test method were made to accommodate the larger sample sizes in this study. The modifications on the soil block test included larger container size, greater amounts of water and soil added to each container and longer exposure time to accommodate the larger specimen size utilized in this test. Soil used in this study was collected from John W. Starr Memorial Forest (Starkville, Mississippi, USA) prior to being sifted. The 2-L containers were filled with 480 g of sieved soil and 275 mL of deionized (DI) water based on a water-holding capacity (WHC) test (AWPA 2023c). The 8-L containers were filled with 1.44 kg of sieved soil and 600 mL of water based on the WHC. The results of this test indicated that the soil had a water hold capacity of 38.9% and a pH of 5.14. Ten feeder strips (6.35×1.27 cm) were then added to each 2-L container and 12 feeder strips of the same dimensions were added to each 8-L container. Southern pine (*Pinus* spp.) feeder strips were used for the containers to which the brown-rot fungus was introduced, and quaking aspen (*Populus tremuloides* Michx.) feeder strips were used for the containers to which the white-rot fungus was introduced. The containers and lids were covered with aluminum foil then autoclaved at 20.7 kPa and 150°C for 45 min.



Figure 2. An 8-L soil block container with 12 aspen feeder strips inoculated with mycelial plugs of the white-rot fungus, *Irpex lacteus*.

Mycelial plugs of *Serpula lacrymans*, Wulf ex Fr., ATCC 36335 and *Irpex lacteus* (Fr.) Fr., ATCC 11245 were inoculated into their respective containers as illustrated in Figure 2 and left to grow until full colonization of the feeder strips (~ 2 weeks). *Serpula lacrymans* was selected for this study because of its demonstrated tolerance to copper-based preservatives, such as those evaluated in the present study. Test samples were autoclaved at 20.7 kPa and 121°C for 45 min, and two samples of the same treatment group were added to each of their respective containers. There were 16 replicates for each fungus, treatment, and layout combination, totaling 192 samples, and exposed in order of treatment. Normally, the standard is based on 10 mm or 19 mm wood cubes, which are deliberately exposed to a test fungus for 8 or 12 weeks, respectively (Figure 4). In this study, 2-L and 8-L polycarbonate containers were used for the 3-ply and 5-ply panels, respectively. Due to the larger size of the samples, the test was conducted for 24 weeks in a Luanaire environmental chamber (Thermal Products Solutions, White Deer, Pennsylvania, USA) maintained at 28°C. During this period, the containers were monitored daily and weighed weekly to ensure that they contained adequate moisture to promote fungal growth. If containers lost weight, sterile water was added to ensure proper conditions for fungal growth. At the end of the test, the samples were removed and oven dried at 35°C until constant weight was obtained to calculate mass losses.

Termite Resistance

Reticulitermes spp. were collected from John W. Starr Memorial Forest (Starkville, Mississippi, USA) and separated from debris as described in AWPA E1-23 (AWPA 2023d). The termites were then counted to determine the weight of three replicate sets of 1000 individuals. This number of termites used for these large-scale tests was based on the findings of França et al. (2022). In addition to the number of termites used, Table 1 summarizes the other parameters of the AWPA E1 standard (AWPA 2023d) that were also modified in this study. Samples were exposed for a total of 28 days. At the end of the test, samples were removed from the containers, cleaned gently with a small brush, and termite mortality (%) was estimated. Termite mortality was assessed visually as prescribed in Clausen et al. (2011). The clean samples were then reconditioned to a constant weight in an oven at 35°C and 65% RH for approximately 2 weeks, before being weighed and subsequently visually assessed, based on Table 2.

Statistical analyses

The data was analyzed as a three-way cross-classification of effects because this analysis allows for the examination of

Table 1. Modifications to the AWP A E1 termite testing standard for evaluation of cross-laminated timber.

Testing conditions	AWPA standard	Modification for CLT samples
Oven drying temp	40°C ± 2 or 103°C ± 2 until constant weight is achieved	60 °C ± 2 until constant weight is achieved
Wood sample size(s)	2.5 × 2.5 × 0.6 cm	10 × 5 × 2.5 cm and 17.5 × 5 × 2.5 cm
Test container(s)	80 × 100 mm glass screw-top jars	2L and 8L
Sand and water	150 g/27 mL	400 g/ 117 mL and 1.4 kg/ 400 mL
Number of termites	400	1000

interactions among multiple factors—configuration (3-ply vs. 5-ply), preservative type (untreated, CA-C, or MCA), and fungus (*S. lacrymans* or *I. lacteus*). This method is important, as the biological durability of timber is influenced not only by individual factors but also by how they interact with one another.

The data analysis was conducted using SAS® software, Version 9.4 of the SAS System for Windows. Prior to analysis, the data was evaluated for conformity with the assumptions of homogeneity of variance (HOV) and normality at the 0.01 significance level using Levene's and Shapiro-Wilk tests, respectively. The models for the fungal decay and termite resistance test are presented in Tables 3 and 4, respectively. If the data met the assumptions, an ANOVA was performed, followed by post-hoc comparisons using LSMEANS in PROC GLM. If the assumptions were not met (p-value < 0.01), a non-parametric approach was taken. Specifically, the dependent variable was rank-transformed, and a rank-based non-parametric ANOVA was performed. If the effect(s) being analyzed proved to be significant at $\alpha = 0.01$, then mean rank separation was done using a Wilcoxon rank pairwise test adjusted by the Bonferroni correction. Termite mortality percentages were arcsine square root transformed to stabilize variance and improve normality prior to statistical analysis. The transformed data were analyzed using the PROC GLIMMIX procedure in SAS, incorporating a binomial distribution to appropriately model the response variable. Multiple comparisons were conducted using Tukey's Honestly Significant Difference (HSD) test at a significance level of $\alpha = 0.01$ to determine statistically significant differences between treatments.

The statistical model in Table 3 represents a three-way cross classification model in which α , β , and γ represent fungus type

Table 2. AWP A Visual rating system (AWPA 2023d).

Visual Rating	Description *
10	Sound
9.5	Trace
9	Slight attack (up to 3% of CSA affected)
8	Moderate attack (3-10% CSA affected)
7	Moderate/Severe attack (10-30% CSA affected)
6	Severe attack (30-50% CSA affected)
4	Very severe attack (50-75% CSA affected)
0	Failure

* CSA= Cross sectional area.

Table 3. Statistical model and assumptions for termite resistance test.

Model	$Y_{ijk} = \mu + \alpha_i + \beta_j + (\alpha\beta)_{ij} + e_{ijk}$	$i = 1, 2, 3$ $j = 1, 2$ $k = 1, \dots, 12$
Assumptions	$\sum_{i=1}^3 \alpha_i = 0 ; \sum_{j=1}^2 \beta_j = 0 ; \sum_{i=1}^3 (\alpha\beta)_{ij} = \sum_{j=1}^2 (\alpha\beta)_{ij} = 0$ $e_{ijk} \sim N(0, \sigma^2), iid$	

Table 4. Statistical model and assumptions for termite resistance test.

Model	$Y_{ijk} = \mu + \alpha_i + \beta_j + (\alpha\beta)_{ij} + e_{ijk}$	$i = 1, 2, 3$ $j = 1, 2$ $k = 1, \dots, 12$
Assumptions	$\sum_{i=1}^3 \alpha_i = 0 ; \sum_{j=1}^2 \beta_j = 0 ; \sum_{i=1}^3 (\alpha\beta)_{ij} = \sum_{j=1}^2 (\alpha\beta)_{ij} = 0$ $e_{ijk} \sim N(0, \sigma^2), iid$	

(*S. lacrymans* or *I. lacteus*), ply configuration (3-ply or 5-ply), and treatment type (CA-C, MCA, untreated control), respectively. The statistical model in Table 4 represents a two-way cross-classification model in which α and β represent treatment type and ply configuration, respectively.

Results and discussion

Fungal decay tests

Average mass loss in CLT specimens exposed to fungal decay are shown in Figure 3.

These data show that untreated southern pine CLT samples did not meet the 40% mass loss threshold typically seen in solid wood after 24 weeks. This is likely due to the larger sample size, which limits fungal access, compared to smaller wood blocks (Figure 4; AWP A 2023c). To account for this, a 24-week exposure period was chosen instead of 12 weeks to increase time for decay, while minimizing contamination risk. Thus, these results suggest that even longer periods of testing may be needed when evaluating fungal resistance of mass timber

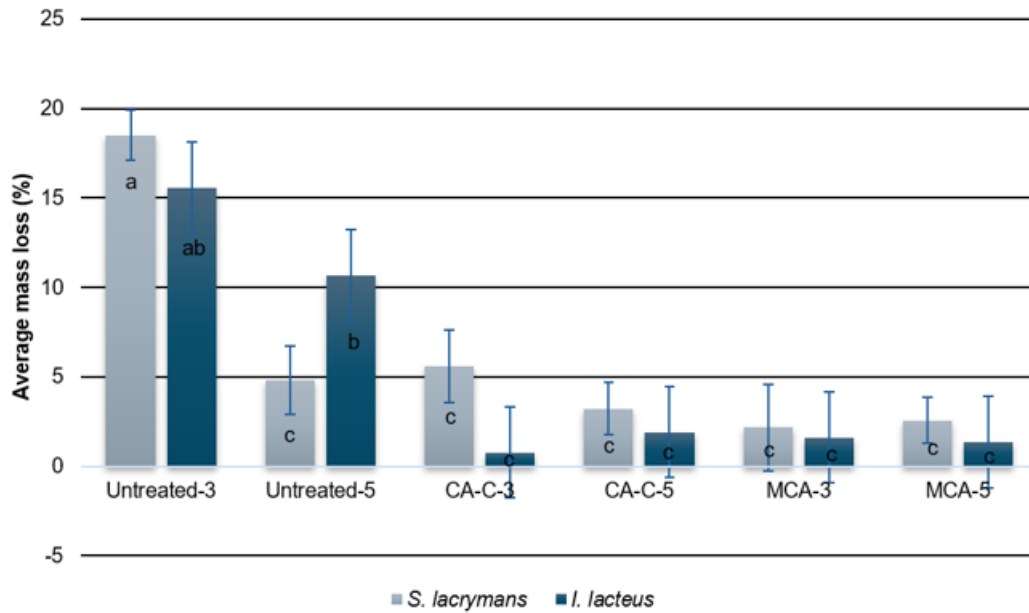


Figure 3. Average mass loss of preservative treated cross-laminated timber samples after 24-week decay tests (3 and 5 represent sample configuration). Bars with the same letter are not significantly different at a 0.01 significance level.

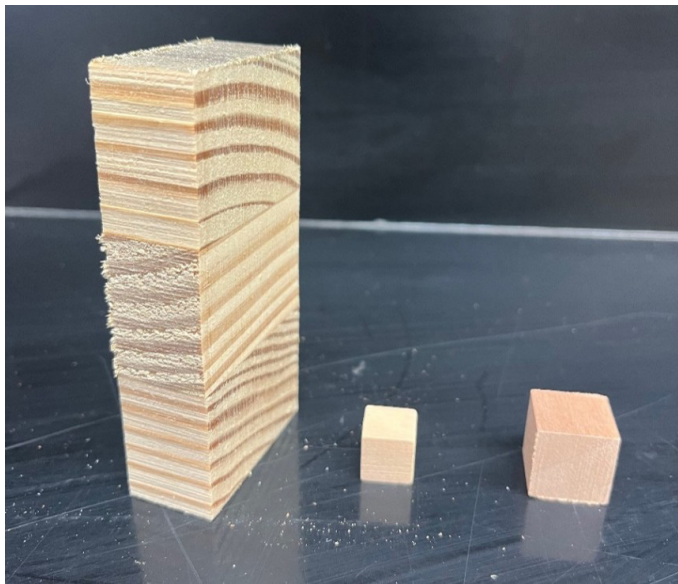


Figure 4. Size comparison of the 3-ply sample used in this study (left) compared to the AWP E10 standard-recommended sample sizes.

materials. Additionally, the test specimens used in this study were southern pine, which is a softwood, and as a result may have contributed to the lower mass loss observed for *I. lacteus*, as this fungus preferentially attacks hardwood species.

The data was analyzed with a three-way cross-classification to examine the effects interactions among multiple factors—configuration (3-ply vs. 5-ply), preservative type (untreated,

CA-C, or MCA), and fungus (*S. lacrymans* or *I. lacteus*). This analysis captures the full complexity of the system, helping to identify specific combinations that may lead to higher mass loss or greater durability. Statistical analysis revealed a statistically significant three-factor interaction ($p = 0.0014$, $\alpha = 0.01$) between number of laminates, treatment, and fungal species, indicating a complex relationship among these factors affecting the mass loss of wood samples.

Untreated CLT samples exposed to *I. lacteus* exhibited significantly greater mass loss after 24 weeks than samples treated with either MCA or CA-C. These findings align with previous research on the effectiveness of copper-based preservatives against *I. lacteus*, although variations in wood species, preservative formulations, and exposure durations can influence decay performance (Freeman and McIntyre 2008). Decay from *Serpula lacrymans* was significantly greater on untreated 3-ply CLT than on 5-ply CLT and treated CLT. Additionally, copper-tolerance by *S. lacrymans* was not observed in this study, as both MCA and CA-C seemed to effectively deter fungal growth for all ply configurations. In general, *S. lacrymans* is a dry-rot fungus and has a lower optimal growth range temperature (22°C) than traditional brown-rot fungi (28°C) (Sienkiewicz et al. 1997; Schmidt 2006), which could explain why tolerance was not observed in this study.

Representative images of CLT samples after the 24-week exposure period (Figure 5) visually illustrate the extent of

decay in both untreated and treated samples. Degradation was observed across all laminates in untreated samples, suggesting uniform fungal colonization. In contrast, decay of preservative-treated samples was primarily localized to middle laminates, despite equal exposure on the feeder strips. This pattern may be attributed, in part, to challenges in preservative penetration. As chemical treatment occurred after CLT fabrication, it is possible that the middle laminates were not as well treated as the outer laminates. In CLT, the adhesive layers can obstruct preservative diffusion (Tascioglu et al. 2003), resulting in an “envelope protection” where the outer laminates receive higher copper penetration and retention compared to the inner layers. Ayanleye et al. (2023) showed that samples taken from the center laminate exhibited reduced copper retention, likely due to limited exposure to the preservative and restricted diffusion across adhesive layers.

It is worth noting that during this study, we did observe occasional negative mass loss (0.01%–2.6%) in some preservative-treated samples, particularly those exposed to *I. lacteus*. This phenomenon is likely attributed to moisture uptake during fungal exposure, where water absorption outweighed degradation-induced mass loss. This is supported by several published studies. For example, Palanti et al. (2012) observed that *Pinus sylvestris* L. treated with a copper complex grafted to amino-functionalized silica gel showed negative mass loss when exposed to *Coniophora puteana* (Fr.) Karst., likely due to hydrolyzed water absorption. Likewise, Meyer and Brischke (2015) reported negative mass loss in Douglas fir (*Pseudotsuga menziesii* (Mirbel) Franco) samples during a pile test with *C. puteana*, which they attributed to internal mycelium growth with minimal wood metabolization. In the present study, preservative treatments may have altered the hygroscopic properties of the wood, enhancing moisture absorption during fungal exposure, as noted by Brischke and Alfredsen (2020). In addition to the effects of preservative treatments, the presence of several adhesive layers comprising the CLT panel may have also contributed to moisture retention by hindering drying, particularly within inner plies. These types of moisture dynamics should be evaluated further in future studies.

Overall, results from the fungal durability tests highlight the effectiveness of a chemical treatment utilized to protect the wood against decay fungi. It is clearly shown that both MCA and CA-C are hindering growth of both *I. lacteus* and *S. lacrymans* for 3-ply and 5-ply CLT layouts. Future research should aim to evaluate additional chemical treatments against common decay fungi. While this study was able to ascertain

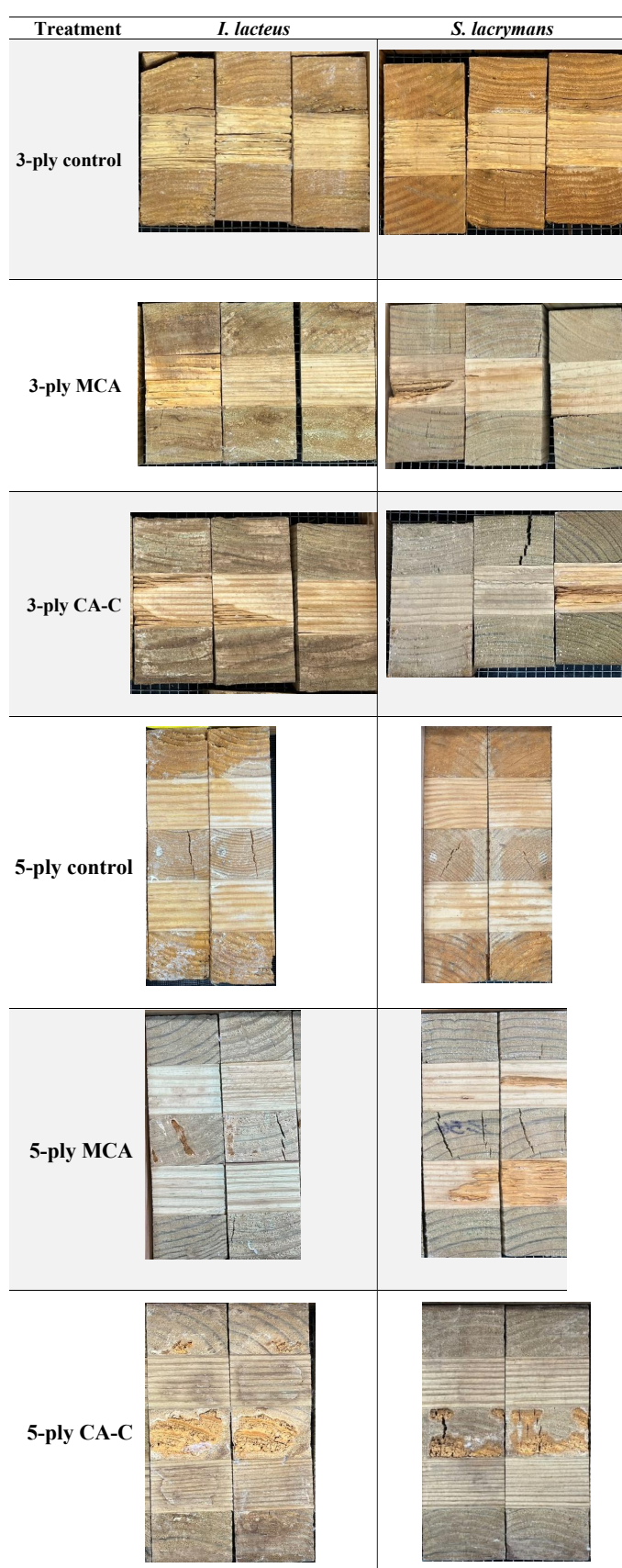


Figure 5. Cross-laminated timber 3-ply and 5-ply samples after 24 weeks of fungal exposure to *I. lacteus* and *S. lacrymans* for control, MCA-treated, and CA-C-treated samples.

useful information on the durability of treated 3-ply and 5-ply southern pine CLT, additional research is needed to understand the treatability and durability of softwood and hardwood mass timber products. Additionally, exploring interactions between panel configuration, environmental variables, and fungal physiology could lead to more accurate assessments of mass timber durability.

Termite tests

Results from laboratory termite tests are presented in Table 5, and images of the samples at the end of the test are shown in Figures 6 and 7. Although the mass loss data are presented as percentages for clarity and ease of comparison, all statistical analyses were conducted on the raw mass loss values. This approach ensured that our analyses reflected the actual consumption rates observed in the samples. The analysis revealed that the interaction between treatment and configuration was not significant ($p = 0.7132$), indicating that the combined effects of these factors did not influence termite consumption. Additionally, the main effect of configuration was not significant at the 0.01 significance level ($p = 0.0141$), suggesting that panel thickness (3-ply versus 5-ply) did not markedly affect termite consumption. In contrast, the main effect of treatment was highly significant ($p < 0.0001$). Post hoc comparisons using Tukey’s HSD ($\alpha = 0.01$) demonstrated that untreated controls had significantly greater mass loss compared to both CA-C and MCA treated panels, with no significant difference observed between the two preservative treatments.

For termite mortality, least squares means comparisons indicated that both CA-C and MCA treatments resulted in significantly higher termite mortality rates, compared to the untreated controls. Analysis of the termite mortality data confirmed that the effect of treatment was highly significant ($p < 0.0001$). It is important to note that termite mortality was done in a subjective manner in this study, instead of performing direct counts,

due to the large numbers of termites. However, these mortality results generally support the conclusion that both treatments were toxic to the termites, which agrees with published literature on similarly treated wood materials.

Similar to decay results, the untreated controls in this study exhibited lower mass loss (3.3% in 3-ply and 1.3% in 5-ply) (Table 5) when compared to results from other published studies. For example, França et al. (2018) reported an average mass loss of 6%, a visual rating of 7.5, and termite mortality ranging from 0–33% in untreated CLT (25.4 mm thick, 101.6 or 50.8 mm wide, 101.6 mm long). In solid untreated southern yellow pine (4–10 mm thick, 40 mm long, and 27 mm wide) Arango et al. (2006) recorded mean mass loss values of 41.3%. However, differences in sample dimensions likely contributed to these discrepancies, as smaller specimens have a higher surface-area-to-volume ratio compared to larger specimens, which can result in higher percent mass losses in the smaller specimens even under identical exposure conditions. Some of the treated specimens even showed slight increases in mass at the end of the test. This can be attributed to several factors, including the absorption of moisture and the accumulation of sand and frass pasted in by the termites. The visual ratings (Table 5, Figure 6 and 7) further reinforce these findings, especially in the 3-ply samples, where there was clear evidence of termite attack, even though percent mass loss values were low.

In aboveground field tests, Mankowski et al. (2022) evaluated soil insecticidal drenches and spray-on insecticide/fungicide treatments for mass timber protection, finding that untreated southern pine CLT had an average termite rating of 8.9 after 2 years. This rating is slightly higher than the untreated control samples in the present study, possibly due to differences in exposure duration and test conditions. Additionally, visual ratings of termite damage are inherently subjective, which may contribute to variations in reported resistance levels across

Table 5. Mass loss, visual ratings and termite mortality data for untreated, CA-C, and MCA treated 3-ply and 5-ply samples.

Configuration	Treatment	Average mass loss (Std. Dev.) (%)	Average visual rating*	Average termite mortality (%)
3-ply	Untreated	3.33 (1.0)	7.0	16.6
	CA-C	0.94 (1.61)	8.1	68.1
	MCA	0.59 (0.94)	8.4	77
5-ply	Untreated	1.28 (0.27)	7.6	22.4
	CA-C	-0.06 (0.40)	8.6	100
	MCA	0.004 (0.32)	8.7	100

* Visual rating based on AWP A E1 (AWPA 2020b).



Figure 6. Cross-laminated timber 3-ply: (a) untreated, (b) MCA-treated, and (c) CA-C-treated samples following exposure to *Reticulitermes* spp. for 4 weeks in a no-choice termite test.

both studies. It is important to note that the Mankowski et al. (2022) study was performed as a field study rather than a controlled laboratory study, therefore termite pressure likely varied compared to that of the present study. However, the consistently high ratings for preservative-treated samples across studies reinforce the effectiveness of copper-based treatments in enhancing termite resistance.

These results demonstrate some of the challenges associated with laboratory termite testing of mass timber products, as well as the importance of wood preservatives in enhancing resistance of southern pine CLT against termite attack. However, laboratory tests cannot account for many of the variables that are likely to affect mass timber materials in use. For example, concerns raised by Kirker et al. (2021) regarding moisture absorption in mass timber elements warrants further investigation, particularly to assess the combined effects of termite exposure and moisture on CLT performance over time. This is in part due to the fact that subterranean termites can increase the moisture content of wood if they have contact with soil or some type of building water source (e.g., leaky pipes).

Conclusions

Results from this study highlight the effectiveness of copper azole and micronized copper azole preservative treatments in enhancing the termite and decay resistance of both 3-ply and

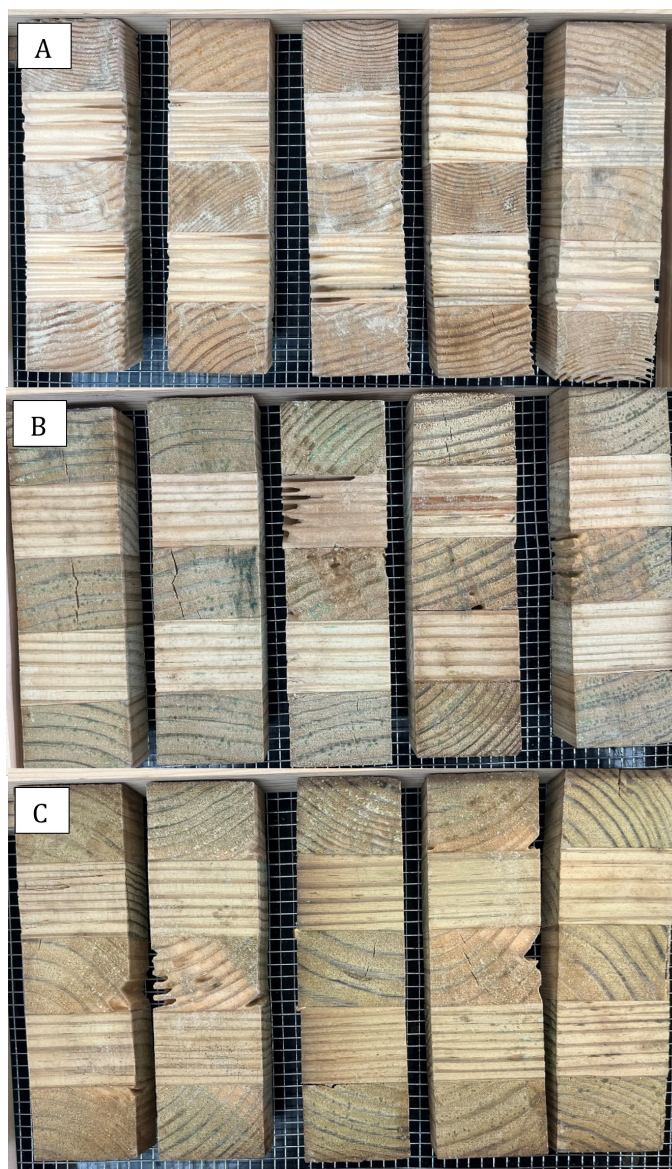


Figure 7. Cross-laminated timber 5-ply samples of (a) untreated, (b) MCA-treated, and (c) CA-C-treated samples following 4-week exposure to *Reticulitermes* spp. in a no-choice termite test.

5-ply cross-laminated timber panels under laboratory conditions, although these findings should be evaluated further in field studies. Fungal decay and termite feeding both lead to mass loss, which directly correlates to reductions in mechanical properties (Curling et al. 2002). Maeda et al. (2015) demonstrated that strength properties of decayed wood can be estimated based on mass loss, reinforcing the relationship between fungal degradation and structural performance. In the current study, preservative-treated samples exhibited lower mass loss from termites or decay, suggesting improved long-term mechanical integrity. However, further research on mass timber should incorporate mechanical testing, such as shear

and compression assessments before and after fungal and termite exposure, to quantify strength performance over time. Future research should also explore long-term performance under varying environmental conditions, optimize treatment formulations, and assess potential impacts on structural properties and sustainability.

One important thing to note is that cross-laminated timber samples evaluated here were produced from post-layup treated materials (i.e., panels were fabricated prior to chemical pressure treatment). While the issue of mass timber treatability was not directly examined in this study, treatment timing undoubtedly impacted durability results. Visual observations of treated samples from both decay and termite tests show that damage was primarily to the middle laminates. This suggests that preservatives may not have penetrated past the outer layer during pressure treatment, which may present a substantial challenge for protection of mass timber panels that are treated post-layup. A better understanding of preservative penetration in mass timber materials fabricated pre- and post-layup—particularly across different wood species—is still needed to guide treatment strategies.

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Interface shear capacity of veneer-based cross-laminated timber glued-in rod splice connections

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Abstract: North America's engineered mass timber sector has expanded rapidly in the past decade, with growing adoption of mass timber products as structural components in tall buildings. Among these, mass timber panels have emerged as viable shear wall elements for lateral force-resisting systems. In taller balloon-type or self-centering rocking wall constructions, panels are stacked vertically, necessitating reliable splice connections between adjacent panels. This paper introduces a novel glued-in rod (GiR) splice connection for vertical joints between stacked panels, featuring a two-stage adhesive installation process and a filler/weep hole system designed to promote full adhesive penetration and accommodate construction tolerances. An experimental program was conducted to evaluate the interface shear strength of GiRs embedded in veneer-based cross-laminated timber under both monotonic and reversed-cyclic loading. Unlike previous studies that focused on tensile strength or monotonic loading only, this work investigates shear behavior relevant to seismic and wind demands. Experimental results are compared against conservative yield strength predictions from the U.S. National Design Specification for Wood Construction (NDS) dowel-type equations. Findings offer new insights into the shear performance of GiR splices in veneer-based cross-laminated timber, informing design assumptions for vertical panel joints in balloon-type mass timber wall systems and providing experimentally validated allowable loads for seismic design.

Keywords: Glued-in rods; Interface shear; Mass ply panel; Shear wall; Splice connection; Veneer-based cross-laminated timber

Introduction

The continued advancement of mass timber engineered wood products provides architects and engineers with a wide range of design solutions. Among these, the use of mass timber panel elements in floor diaphragms and shear walls has emerged as a promising strategy for achieving both sustainability and structural resilience (Tannert et al. 2018; Izzi et al. 2018; Sun

et al. 2020; Granello et al. 2020; Di Cesare et al. 2020; Abed et al. 2022). These developments have enabled mass timber to be considered as a primary structural material for mid- and high-rise buildings. However, challenges remain, particularly in implementing mass timber panels in tall structures (Pei et al. 2016). A key limitation stems from manufacturing and transportation constraints, which restrict panel lengths. In tall balloon-frame wall systems, for instance, the required wall height may exceed the available panel length, necessitating vertical stacking and the use of splice connections at panel joints

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to ensure structural continuity. Therefore, the performance and reliability of panel splice connections have become a critical focus of research and design. Figure 1 illustrates this need, showing a rendering of a 10-story mass timber shake-table test building (Pei et al. 2024), where splice connections are required to form continuous balloon-type shear walls.

Several design alternatives have been proposed for splice connections in balloon-type mass timber shear walls. Two conceptual examples are illustrated in Figure 2. In the first configuration (Figure 2a), six steel plates are fastened externally across the panel joint. In this design, the end plates are typically assumed to resist overturning moments, while the central plates carry shear forces (Zimmerman et al. 2020). The second configuration (Figure 2b) utilizes glued-in rods (GiRs) to transfer forces across the splice in a similar manner. In design, the GiRs are grouped according to their function: longer rods embedded near the panel ends can be assumed to resist overturning moments, while shorter, centrally located rods are assumed to resist shear.

Both splice types shown in Figure 2 are viable. The steel plate connection offers the advantage of rapid on-site assembly but

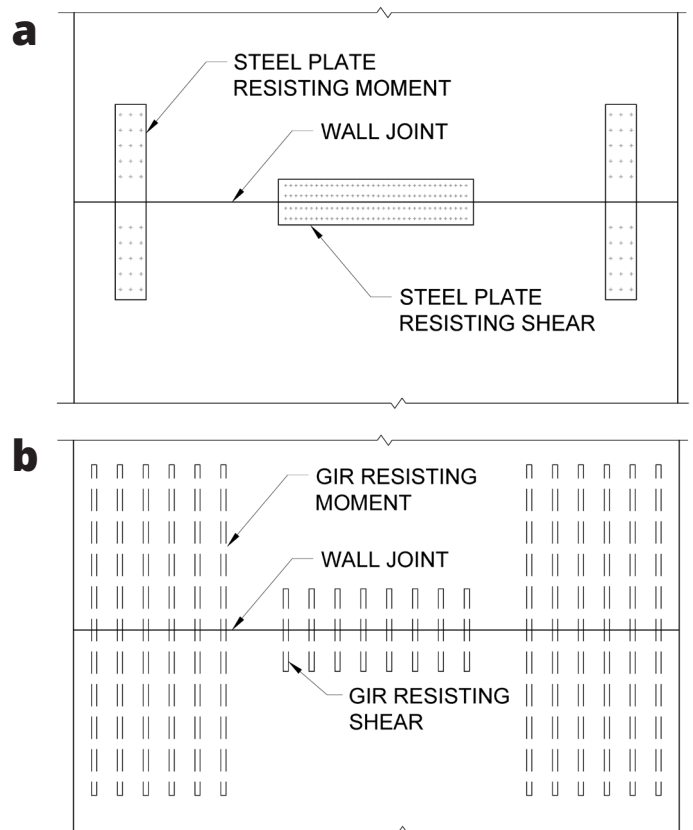


Figure 2. Examples of mass timber wall splice connections: (a) external steel plates; (b) internal GiRs.

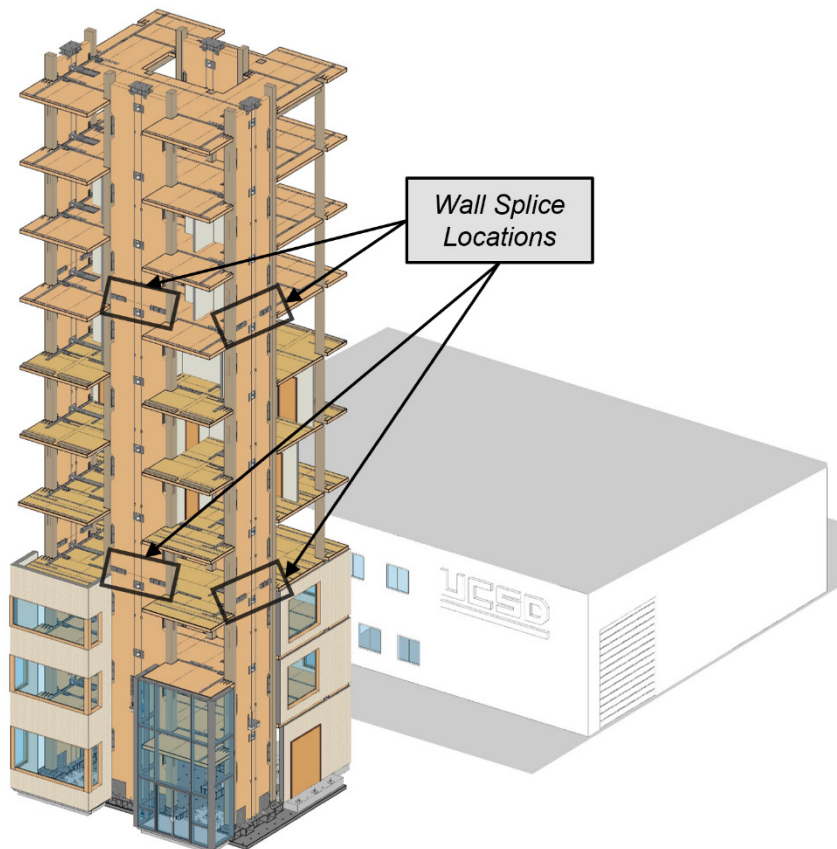


Figure 1. Mass timber wall splice locations along the building height (adapted from Pei et al. 2024).

requires additional measures for corrosion and fire protection. In contrast, the GiR connection provides benefits such as improved corrosion and fire resistance, enhanced aesthetics, and favorable ductile behavior under seismic loading (Townsend et al. 1990), making it an attractive option for designers. However, GiR connections demand greater prefabrication effort, as precise hole alignment is necessary for proper rod placement—especially challenging for longer embedded lengths. Moreover, successful installation requires controlled environmental conditions and skilled application to ensure proper adhesive bonding between the rods and the mass timber substrate. Based on the design premise presented in the previous paragraph, when designing the GiR splice connection to resist lateral loads from both wind and seismic actions, the main design considerations are that the moment-resisting outer rods produce an internal force couple, which places individual rods in tension (and potentially some compression), while the central rods resist shear forces and provide lateral stiffness.

Glued-in rod connections were originally developed for glue-laminated timber, where their axial (pull-out) capacity and, to a lesser extent, their lateral (shear) and combined load-carrying behavior have been experimentally characterized (Deng 1997; Bengtsson and Johansson 2002; Jensen and Gustafsson 2004; Steiger et al. 2007; Jensen and Quenneville 2009). More recently, the axial and bond performance of GiRs has been investigated in cross-laminated timber (Azinović et al. 2018; Sofi et al. 2021; Ayansola et al. 2022), and the tension pull-out capacity of single GiRs has been quantified in mass ply panel (MPP) (Field et al. 2023). GiR splice connections are attractive for vertical panel-to-panel joints because the rods are fully embedded within the panels, providing improved corrosion and fire resistance, enhanced aesthetics, and ductile behavior relative to externally fastened steel-plate splices (Townsend et al. 1990; Zimmerman et al. 2020). In contrast, the behavior of GiR connections loaded in interface shear, especially under reversed-cyclic demands and in the panel-to-panel splice configuration studied here, has received comparatively little attention. The present study addresses this gap by characterizing the interface shear capacity, stiffness, and cyclic response of GiR splice connections in MPP and by comparing the observed response with U.S. National Design Specification for Wood Construction (NDS) yield predictions (AWC 2018).

Previous research has studied the behavior of GiRs in tension in a variety of timber substrates. Bengtsson and Johansson (2002), Deng (1997), and Steiger et al. (2007) investigated the structural performance in a glue-laminated timber substrate;

Ayansola et al. (2022), Azinović et al. (2018), and Sofi et al. (2021) in a cross-laminated timber (CLT) substrate; and Field et al. (2023) in an MPP (ICC-ES 2022) substrate. The experimental program presented in Field et al. (2023) characterized the pull-out tensile strength and stiffness of a single GiR at different levels of embedment. Test results were used to propose a design equation with a format that is similar to the ones available in the New Zealand design code (NZS 3603 1993), which is based on work developed in Deng (1997), among others.

With respect to the behavior of GiRs in shear in timber substrates, several experimental and analytical modeling efforts are available in the literature. Bengtsson and Johansson (2002) evaluated steel-threaded GiRs embedded into glulam specimens as part of the Glued-in Rods for Timber Structures (GIROD) project used to promote code adoption of this connection type in Europe. The objective of the study was to investigate the effect of rod spacing and edge distance on the axial and lateral load-carrying capacity. For the lateral loading tests, rods were glued-in at a length of 320 mm parallel to the grain of glulam specimens with varied edge distances. For some specimens, low-strength rods were installed to investigate the influence of steel quality on lateral load-carrying capacity. For low-strength rods particularly, two plastic hinges were observed to form in the rods during testing. When the edge distance was decreased, larger embedding displacements in the timber were observed. Jensen and Gustafsson (2004) and Jensen and Quenneville (2009) validated this conclusion through experimental analysis and modeling of a GiR shear connection of dowels embedded in glulam. In both works, different joint parameters were evaluated with varying glulam geometries, as well as dowel quantities and placement, ultimately confirming that increasing edge distance increased observed shear strength. From these test programs, the minimum edge distance was 2 rod diameters (d), from which an initial testing plan could conceivably be developed based on this minimum distance to provide a lower bound for expected capacity of GiRs in shear.

This research for both tension and shear loading has helped to establish parameters for detailing GiR connections. In state-of-the-art reviews, Steiger et al. (2015) and Tlustochowicz et al. (2011) summarized several important recommendations as follows: target edge distance of $2.5d$ with a lower bound of $2d$, rod spacing minimum of $5d$, and an upper bound glue-line thickness of 2 mm. Three adhesives were also listed that have been utilized most prominently for GiR connections: polyurethane (PUR), phenol resorcinol (PRF), and epoxy resins (EPX),

where it was noted that EPX developed an especially strong bond with the steel and wood. A study by Xu et al. (2020) pointed out that some uncertainty can arise in the strength of these connections due to panel manufacturing defects that lead to panel voids, underscoring the necessity for the careful selection and application of adhesive in order to maximize the strength of the bonded connection.

In shear design, the European Yield Model (EYM) developed by Johansen (1949) serves as the basis for calculating the lateral load-resisting capacity of the embedded dowels in timber. This EYM established several parameters that are needed for characterizing the embedment strength, including bearing resistance, dowel size, and loading angle. The model has been used extensively to predict the shear strength of embedded dowels in a variety of timber substrates, including glue-laminated timber (Stamatopoulos et al. 2022), CLT (Uibel and Blaß 2006), and laminated-veneer lumber (Bader et al. 2016).

Mass ply panel is a veneer-based cross-laminated timber panel fabricated from nominal 25.4 mm (1 in.) Douglas-fir veneers assembled in cross-laminated layups in accordance with ANSI/APA PRG 320 (APA 2025) and ESR-4760 (ICC-ES 2022). This veneer-based assembly produces a homogeneous layup with lower variability than sawn-lumber-based CLT. Relative to lumber-based CLT wall panels, MPP provides greater dimensional stability and higher in-plane shear stiffness for an equivalent panel thickness, owing to the absence of the gaps and lumber-layer discontinuities present in lumber-based CLT and the elimination of strength-reducing characteristics such as knots and slope of grain that can govern shear capacity. Relative to structural insulated panels (SIP) and plywood-sheathed light-frame walls, for example, MPP offers a solid, mass timber panel capable of carrying significant in-plane shear without reliance on fasteners between a frame and sheathing. These attributes make MPP well suited to wall applications requiring high in-plane stiffness and strength and serve as motivation for the development of high-capacity, internally embedded splice connections such as the GiR connections studied here. Recently, experiments have been performed to determine the embedment strength in MPP using the half-hole method prescribed in ASTM D5764 (ASTM International 2018). Miyamoto et al. (2020) evaluated the embedment strength of MPP with screws, nails, and bolts in different orientations. While bolts were not tested along the ply (loading parallel to the plane of the veneer, and perpendicular to the strong axis), screws and nails were tested in this orientation, and an average strength of 18 MPa was recorded.

Generalized yield limit equations were developed based on the EYM (AWC 2014, 2018) to predict yield strengths for dowel-based connections in single and double shear. Table 1 shows a schematic of the six different yield modes and the corresponding equations for single shear, whereby the predicted yield strength for each dowel was determined by taking the lowest result from the six yield mode equations. Note that to determine the reference lateral design values, the predicted yield strength equations shown in Table 1 were divided by a reduction term, R_d , that is listed in the NDS Table 12.3.1B (AWC 2018). This term converts the theoretical yield load predicted by each EYM equation into a reference lateral design value consistent with the NDS allowable stress design (ASD). R_d is mode-specific and accounts for the ratio between the load at first yield and the load corresponding to the reference deformation limit of 0.381 mm (0.015 in.), as well as the format conversion from the Load and Resistance Factor Design (LRFD) basis of the yield model. Applying R_d ensures the resulting reference design value reflects appropriate deformation and safety levels for dowel-type connections. For Mode III_s, III_m, and IV, and for dowels with diameter between 6.35 mm (0.25 in.) and 25.4 mm (1 in.), the R_d is given by:

$$R_d = 3.2 K_\theta \quad [1]$$

where $K_\theta = 1 + 0.25(\theta/90)$, and θ is equal to the maximum angle between the direction of load and the direction of grain for any member in the connection ($0^\circ \leq \theta \leq 90^\circ$).

Prior work has extensively characterized tension capacity of GiRs in timber substrates and, to a lesser extent, monotonic shear behavior. However, reversed-cyclic shear response of GiR splices in MPP for vertical joints remains unexplored. Thus, the main objectives of this work are to:

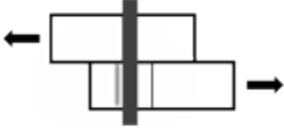
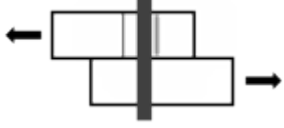
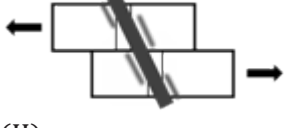
1. quantify interface shear capacity and stiffness under monotonic and cyclic loading;
2. assess the influence of joint interface adhesion (unintended adhesive at the panel interface) on strength and stiffness; and
3. compare observed yield to NDS predictions (AWC 2018) and report design values following ICC-AC526 (ICC-ES 2021).

Materials and methods

Specimen design

Eleven MPP specimens were fabricated for the experimental program. The panels used in this study were grade F16, fabri-

Table 1. Yield modes and predicted yield strength per NDS (AWC 2018).

Yield Mode	Equation	Eq. No.
 (I _m)	$P = D l_m F_{em}$	[2]
 (I _s)	$P = D l_s F_{es}$	[3]
 (II)	$P = k_1 D l_s F_{es}$ where: $k_1 = \frac{\sqrt{R_e + 2 R_e^2 (1 + R_t + R_t^2) + R_t^2 R_e^3 - R_e (1 + R_t)}}{(1 + R_e)}$	[4]
 (III _m)	$P = k_2 D l_m F_{em}$ where: $k_2 = -1 + \sqrt{2 (1 + R_e) + \frac{2 F_{yb} (1 + 2 R_e) D^2}{3 F_{em} l_m^2}}$	[5]
 (III _s)	$P = \frac{k_3 D l_s F_{em}}{(2 + R_e)}$ where: $k_3 = -1 + \sqrt{\frac{2 (1 + R_e)}{R_e} + \frac{2 F_{yb} (2 + R_e) D^2}{3 F_{em} l_s^2}}$	[6]
 (IV)	$P = D^2 \sqrt{\frac{2 F_{em} F_{yb}}{3 (1 + R_e)}}$	[7]
where:	P = predicted yield strength D = dowel diameter (in.) F _{yb} = dowel bending yield strength (psi) R _e = F _{em} / F _{es} R _t = l _m / l _s l _m = main member dowel bearing length (in.) l _s = side member dowel bearing length (in.) F _{em} = main member dowel bearing strength (psi) F _{es} = side member dowel bearing strength (psi)	

cated from 1.6E Douglas-fir veneers with an equivalent specific gravity of approximately 0.42 (ESR-4760). As shown in Figure 3, each specimen consisted of two 102-mm-thick grade F16-4 MPP parts, including a custom-cut top section (Part A) and a rectangular bottom section (Part B). The shape of the specimens allowed for the line of action of the actuator-applied load to

act collinearly with the interface between the two parts (i.e., no imposed moment at the panel-to-panel interface). Parts A and B were connected using three 25.4 mm (1 in.) diameter × 406 mm (16 in.) long ASTM F1554 Grade 36 (248 MPa [36 ksi]) rods centered in the panel thickness and embedded 203 mm (8 in.) into each part. The rods were partially threaded on

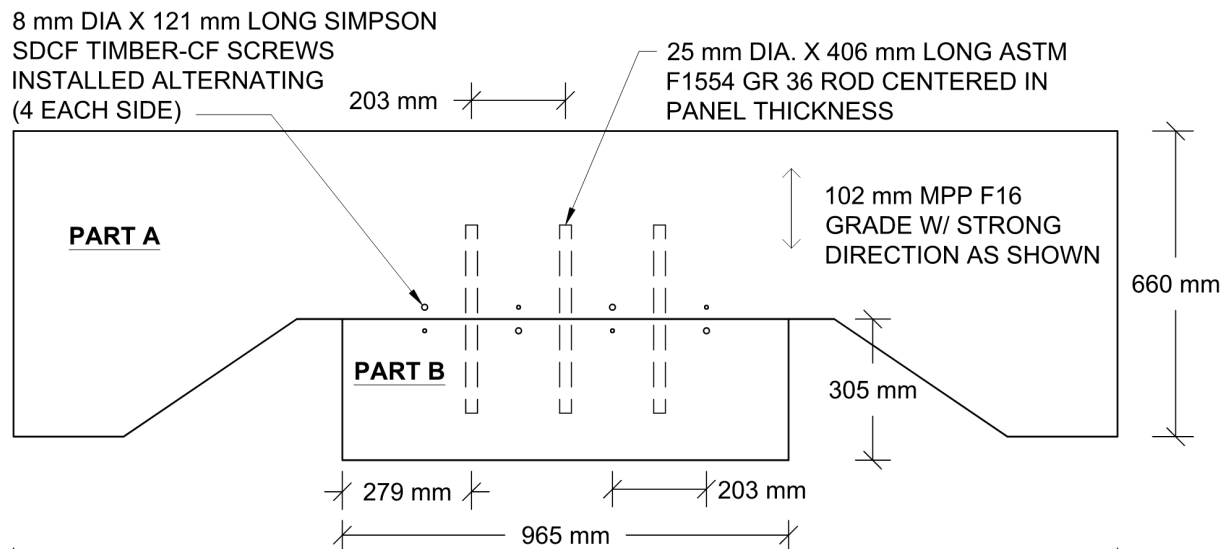


Figure 3. Specimen geometry.

the ends over a length of 102 mm (4 in.), leaving a 203 mm (8 in.) unthreaded (smooth) length in the middle of the rod. The outer rods were nominally spaced 279 mm (11 in.) from the edge of Part B and nominally spaced between rods 203 mm (8 in.) on-center.

The position of the rods created an edge distance of $2d$ (51 mm; 2 in.), the minimum edge distance tested in the literature (Steiger et al. 2015). This allowed characterization of a lower-bound capacity. Rods were installed in 32 mm (1-1/4 in.) diameter by 216 mm (8-1/2 in.) long holes in each panel and bonded to the MPP with structural epoxy in the vertical position. The holes were oversized by 6.35 mm (0.25 in.) relative to the diameter of the rods, resulting in a glue-line thickness of 3.175 mm, exceeding the largest recommended glue-line thickness of 2 mm in the prior literature (Steiger et al. 2015). This larger glue-line thickness represented an upper-bound condition expected in practice, which may be needed, such as in the case of construction tolerances of mass timber wall splice connections to accommodate the simultaneous installation of dozens of long threaded rods into both panels. Four 8 mm (5/16 in.) diameter by 121 mm (4-3/4 in.) long Simpson SDCF Timber-CF (fully threaded) screws were installed in an alternating pattern on each side of the specimen to restrain out-of-plane panel cracking and dilation.

Each rod was embedded 203 mm (8 in.) into each panel, corresponding to an embedment-to-diameter ratio of approximately 8 for the 25.4 mm (1 in.) diameter rods. This embedment was selected so that the rods would be sufficiently anchored to develop their full lateral (dowel) capacity and form plastic hinges

under interface shear, rather than failing by rod pull-out or loss of adhesive anchorage. The selected length was consistent with guidance that the bonded length be sufficient to mobilize the stiffness and strength of the surrounding timber (Steiger et al. 2015) and with embedment-to-capacity relationships established for axially loaded GiRs in MPP (Field et al. 2023). It was also compatible with the available panel thickness and with the rod sizes adopted for the GiR splice connections in the companion six-story shake-table study (Kontra et al. 2026).

Specimen fabrication

The grade F16-4 MPP were manufactured by Freres Engineered Wood, Inc. (Lyons, OR USA). Parts were milled and machined to size at A.A. “Red” Emmerson Laboratory at Oregon State University (OSU, Corvallis, OR), then shipped to Timberlab in Portland, OR for GiR installation and specimen assembly. Specimens were then delivered back to OSU for testing in the Structural Engineering Research Laboratory at the O.H. Hinsdale Wave Research Laboratory.

The GiR installation procedure was the same for all shear specimens. Figure 4 illustrates a multi-step process that was developed for administering adhesive into the GiR connection to facilitate uniform and complete bonding of the rod and MPP. First, before applying any adhesive, holes were drilled and then cleaned using a round wire brush and compressed air to remove any free particles in both Part A and Part B. Second, a small amount of high viscosity adhesive was applied to the surface of the holes to fill any voids in the MPP so that the injected low-viscosity epoxy was confined to the area around the rod and did not disperse into the interior voids in the panel.

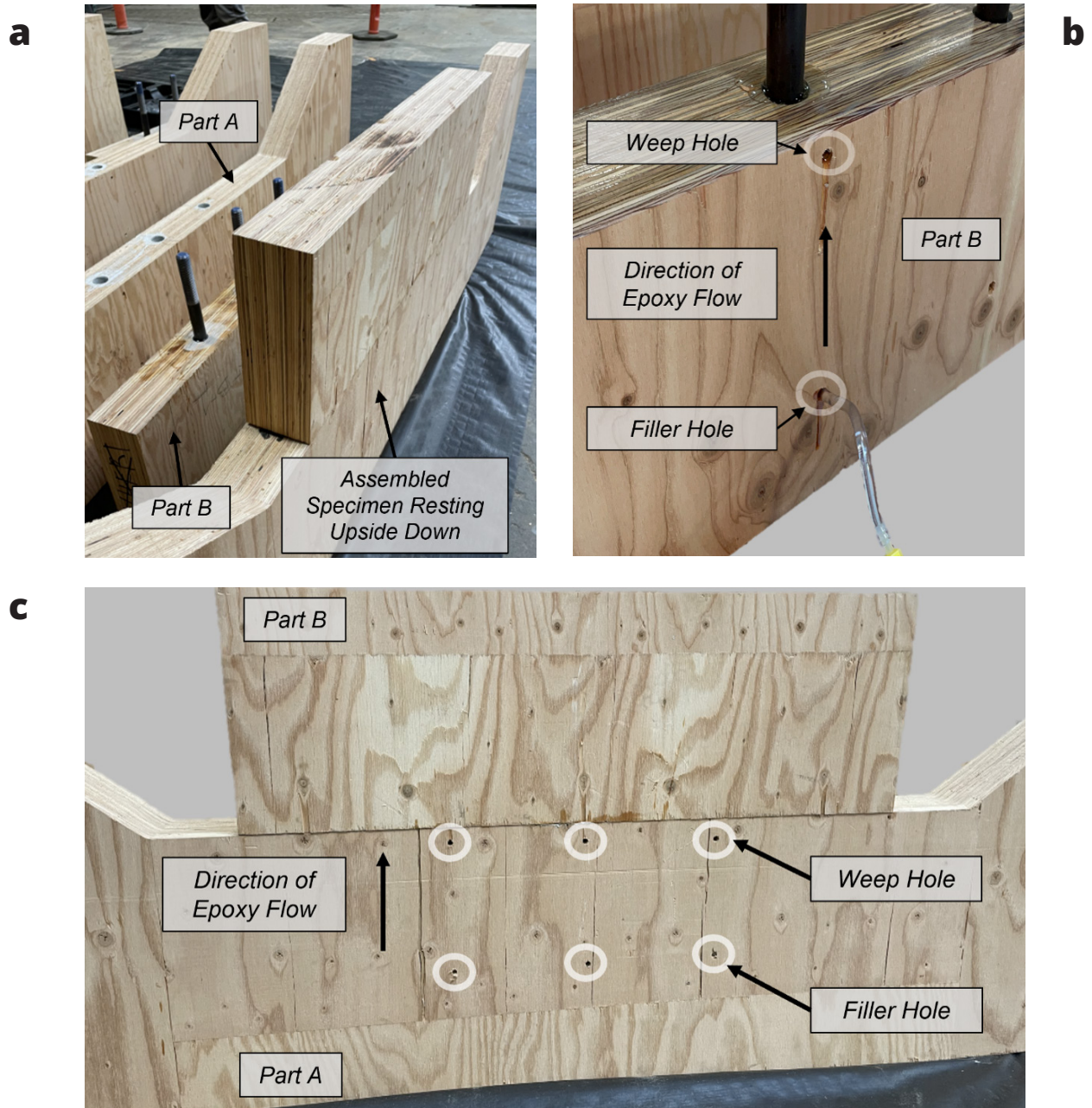


Figure 4. Specimen fabrication process: (a) Part A with first coat applied to the holes, Part B with rods installed, assembled specimen resting upside down relative to specimen geometry in Figure 3; (b) filler and weep hole in Part B; (c) filler and weep hole and direction of flow for Part A.

A custom-designed drill “spreader” attachment was used to evenly spread the adhesive to the wall of the rod holes. This was accomplished by plunging the spreader attachment on the end of a long drill rod into a small amount of adhesive at the bottom of the hole and then slowly extracting, while simultaneously spinning the spreader. The first coat of adhesive, indicated in Figure 4a, was left to cure for approximately 24 h. The adhesive used to coat the holes in this first step was designated as CI-GV epoxy gel (Simpson Strong-Tie 2023a).

After the adhesive had cured, 9.5 mm (3/8 in.) diameter holes were drilled 25.4 mm (1 in.) from each end of the rod holes to serve as the “filler” and “weep” holes, where the adhesive was injected in the filler hole and flowed out the weep hole. Any debris from drilling the filler and weep holes was cleaned out with compressed air. The threaded rods were then placed into the holes in Part B, and adhesive was applied through the filler holes, as shown in Figure 4b. This method of forcing the adhesive into the bottom of the hole and allowing it

to flow to the top opening avoids formation and entrapment of air bubbles along the bonded length. The adhesive used in this study to fill the holes was the CI-LV FS epoxy (Simpson Strong-Tie 2023b).

Once the GiRs were installed in Part B and allowed to cure, the element was lifted and placed on top of Part A, such that the assembled specimen rested upside down (Figure 4a and Figure 4c) relative to how it is shown in Figure 3. The adhesive was then injected through the filler hole and flowed out the weep hole in Part A, as indicated in Figure 4c. As expected, some weep holes were not sufficient to drain all the adhesive being injected, and in some cases, adhesive would overflow and cure at the joint interface (Figure 4b). The effect of the excess adhesive in the joint interface is further discussed in the results section. While the two-stage adhesive application involved applying CI-LV FS epoxy over cured CI-GV gel epoxy, limited published data exists on the bond characteristics between these specific epoxy systems. The manufacturer's technical documentation does not indicate incompatibility between these products, and both are epoxy-based adhesives designed for structural applications. Post-test observations confirmed adequate bonding at the rod-to-timber interface

in all specimens, with no evidence of delamination between adhesive layers.

Once fabricated, specimens were stored inside OSU's Structural Engineering Research Laboratory at ambient temperature for 2 months prior to testing. The moisture content of the specimens was measured at the time of testing and had a mean value of 11.7% and a coefficient of variation of 4.7%, measured using a handheld surface moisture meter.

Test setup

Figure 5 shows an elevation and section view of the self-reacting test setup design. The main components include base connections, actuator and load cell, a transition W-shape section, a steel reaction buttress, the primary support beam, and out-of-plane restraints. Part B of each MPP specimen was restrained using steel connections to a 6096 mm (20 ft) long W14x159 reaction beam indicated as primary support beam.

The MPP specimen was attached to an actuator positioned so that the centerline of the actuator force was at the same elevation as the specimen joint interface. A 32 mm × 127 mm × 673 mm (1-1/4 in. × 5 in. × 26-1/2 in.) steel plate was inserted

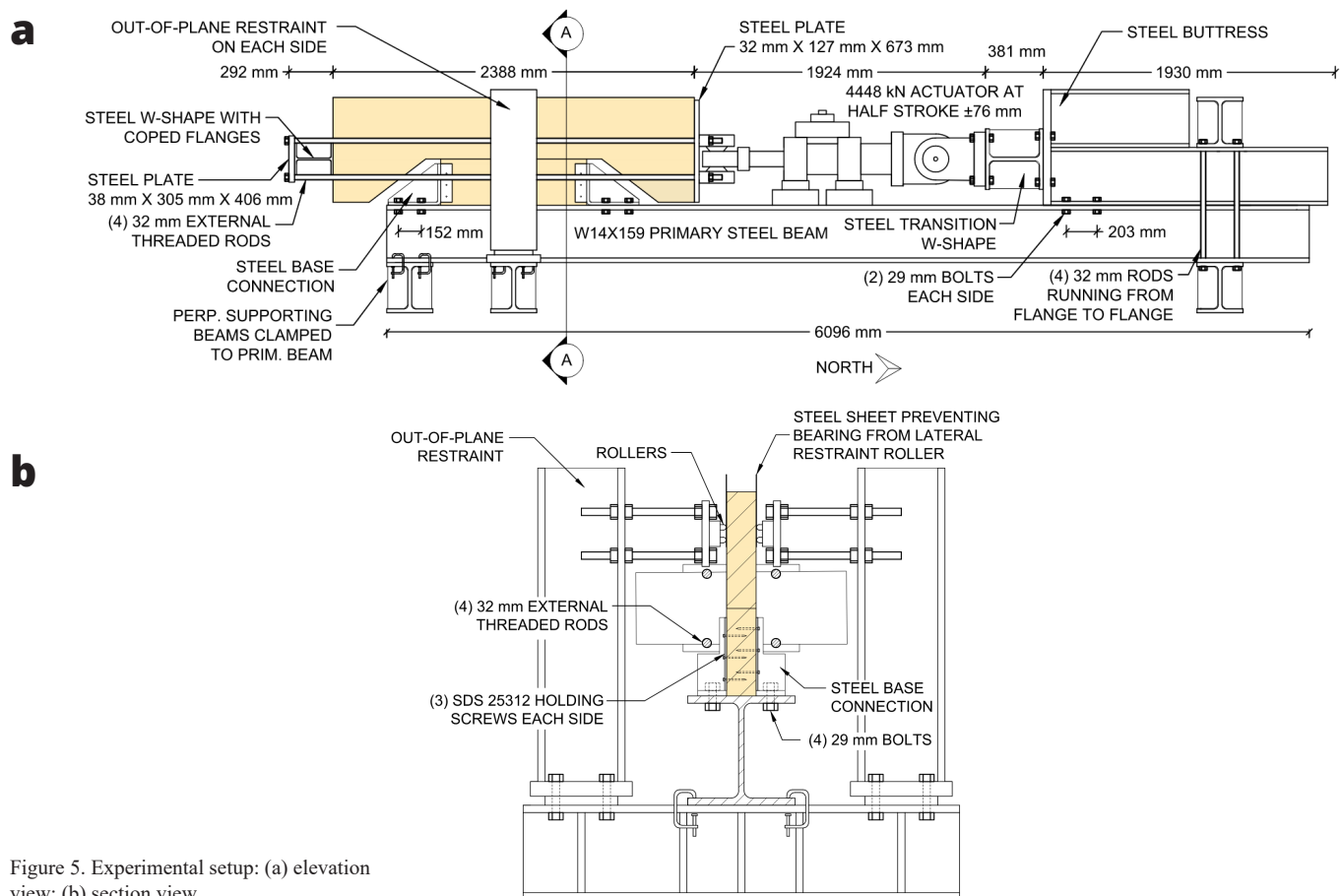


Figure 5. Experimental setup: (a) elevation view; (b) section view.

between the face of the actuator and the end of the MPP specimen to uniformly distribute the applied load and prevent fiber crushing at the load application point. Four 32 mm (1-1/4 in.) diameter steel rods connected the actuator at the north end of the specimen to a wide-flange beam at the south end of the specimen to allow the specimens to be pushed and pulled by the actuator. The actuator was attached to a steel reaction buttress. The buttress was restrained to the primary support beam with four 28.6 mm (1-1/8 in.) diameter steel bolts and additional wrench clamps. The bolts were spaced at 203 mm (8 in.) on-center and placed at the south end of the buttress. To promote lateral stability of the reaction, a W12x120 section was placed on top of the north end of the buttress and tensioned to another W12x120 section that was oriented perpendicular to the supporting beam beneath. The MPP specimen and steel buttress rested upon and were restrained to the primary support beam. The MPP specimen was attached to the primary support beam by means of a steel base connection that was bolted to the beam and screwed to the MPP specimen, using SDS 25312 holding screws. Underneath the primary support beam were three perpendicular W12x120 beams that were clamped to the primary support beam to provide lateral stability for the primary support beam.

To facilitate in-plane movement of the MPP specimen during testing, out-of-plane restraints were added to both sides of the specimen, as shown in Figure 5b. This restraint mechanism included roller bearings that allowed for Part A of the MPP specimen to move in-plane with minimal resistance, while restricting out-of-plane motions. In addition, two 1.5 mm thick steel bearing plates were fastened to either side at the contact point of the roller to prevent direct roller contact with the MPP.

Instrumentation setup

The responses of MPP specimens during interface shear tests were monitored with six instruments. Four sensors were used to measure the relative displacements between the top and bottom elements across the joint of the specimen, as shown in Figure 6, and two other sensors tracked the actuator-applied load and actuator displacement. Two displacement sensors (string potentiometers), SP1 and SP2, on the east and west face, respectively, measured horizontal relative displacements between Part A and B, and two linear potentiometers (Duncan-type potentiometers), labeled SP3 and SP4, measured relative vertical displacements between Part A and Part B.

Testing protocols

Testing featured two loading protocols. The first was a displacement-controlled monotonic loading protocol, and the second

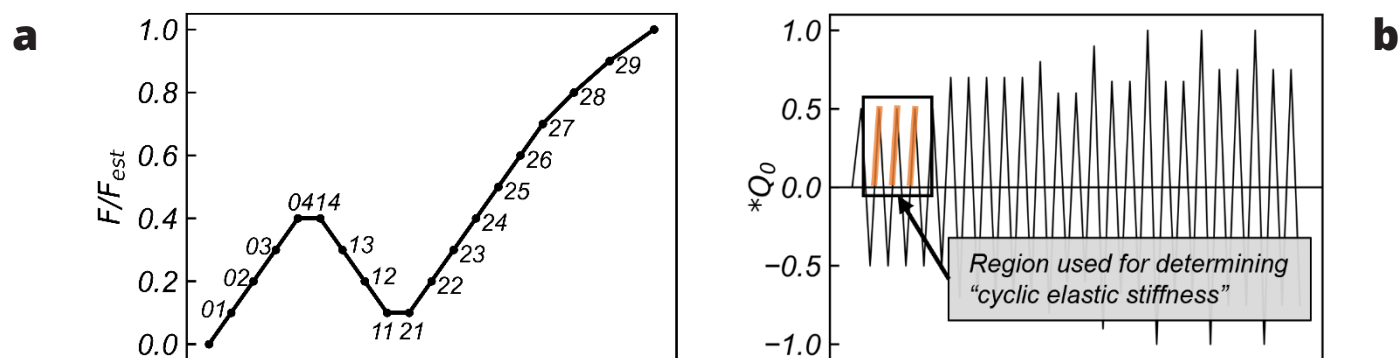
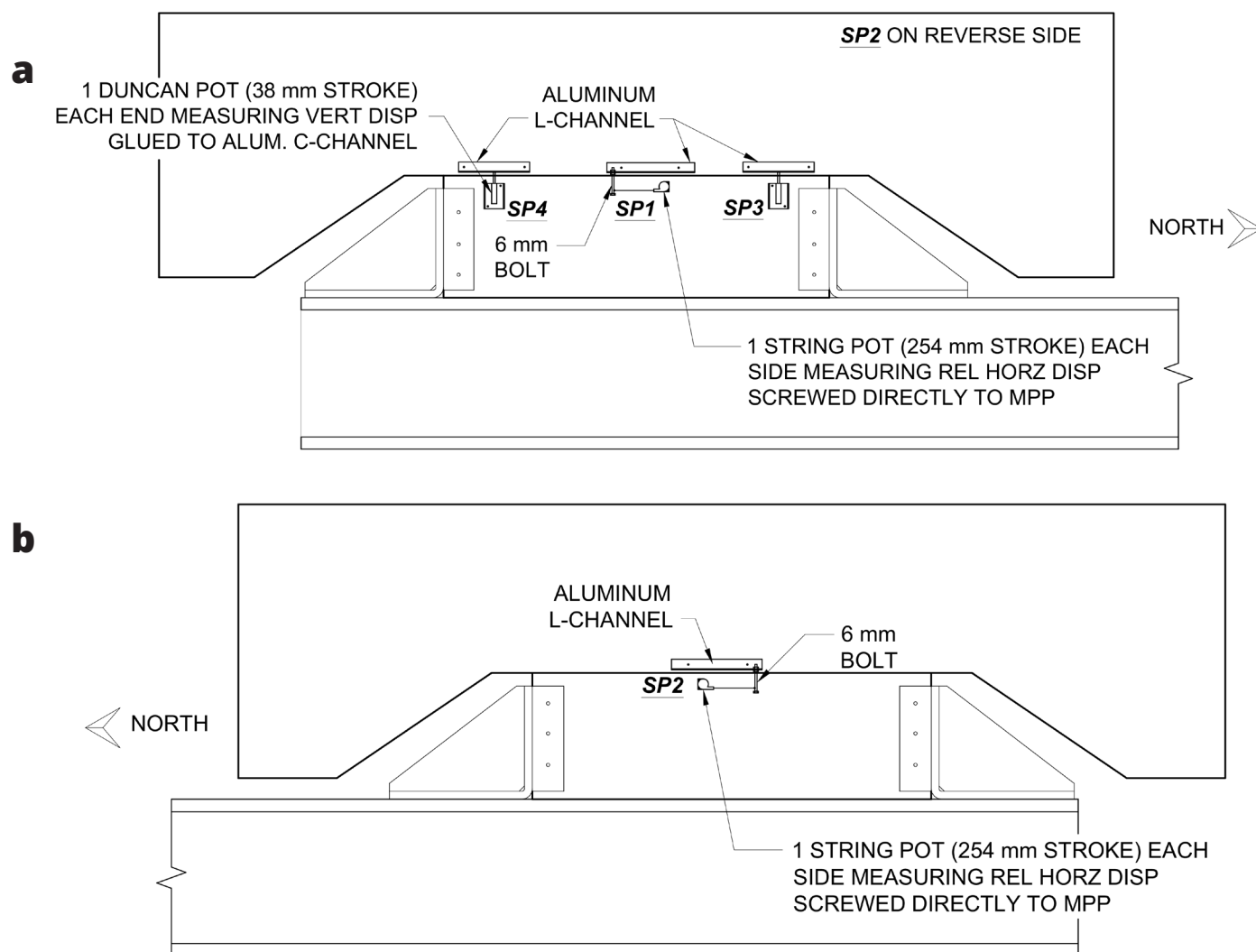
was a force-controlled reversed cyclic loading protocol. Figure 7a shows the monotonic loading protocol applied in accordance with ISO 6891 (ISO 1983). Under this first monotonic loading protocol, two specimens were tested. A F_{est} value of 89 kN was used for the two monotonic tests. The initial actuator displacement rate was first set at 2.5 mm/min, increased to 7.6 mm/min after 12.7 mm was reached, and lastly increased to 12.7 mm/min after 31.8 mm displacement was achieved. The last displacement rate of 12.7 mm/min was maintained until the post-peak load had reduced to 80% of peak load or until the maximum stroke of the actuator was reached ($\sim \pm 75$ mm), whichever occurred first.

Reversed-cyclic loading was performed in accordance with a force-controlled CUREE protocol (Krawinkler et al. 2001), as shown in Figure 7b, where the Q_0 value was set to the average peak load obtained from monotonic testing. Cyclic testing included nine specimens. The first cyclic test began at a displacement rate of 5.1 mm/min, was increased to 10.2 mm/min after five cycles, then was increased to 20.3 mm/min after another five cycles and maintained at this latter level for the remainder of the test. Subsequent cyclic tests for specimens numbered 2 to 9 were performed at a displacement rate of 20.3 mm/min for the entirety of the test.

Data post-processing and reduction

Two monotonic tests were conducted to determine a reference force value Q_0 to be used for the reversed-cyclic loading. Reported results from the monotonic tests include peak load, displacement at peak load, and the yield strength (to be discussed more in the next section). The reported values for the cyclic tests include the peak load (kN), number of cycles to reach the peak load (#), and the cyclic elastic stiffness (kN/mm). The first two parameters are self-explanatory. However, for the cyclic elastic stiffness, multiple definitions could be used. The elastic stiffness was determined by taking the average of the slope of the pushing phase for cycles 2 to 4 of the load versus displacement graph, corresponding to regions highlighted in Figure 7b.

Reporting guidelines for experimental tests involving GiRs in wood structural elements are also provided in ICC-AC526 (ICC-ES 2021). While this document primarily relates to strength in tension for this connection type, the allowable loads in shear were determined using the definitions in the acceptance criteria document, including: *nominal shear strength*, which is the average of the peak loads; *allowable GiR shear capacity*, which is the nominal shear strength divided by a safety fac-



tor equal to 5; and *available GiR strength in LRFD*, which is determined by using a format conversion factor, K_{ps} , of 3.32, a resistance factor, ϕ , of 0.65, and a time effect factor, λ , of 1.00, all applied to the allowable GiR shear capacity. The format conversion factor, resistance factor, and time effect factor were determined in accordance with Table N3.3 in Appendix N of the 2018 NDS (AWC 2018).

Yield strength determination

The ability of the NDS equations to predict the yield strength of this connection type was examined through comparison with the yield strengths obtained from the monotonic tests. While several approaches for determining yield strength are available for timber connections (Kržan et al. 2023), two commonly used methods in North America were selected for this study: (1) the 5% offset method described in ASTM D5764 (ASTM International 2018); and (2) the equivalent energy elastic-plastic (EEEP) method described in ASTM E2126 (ASTM International 2005). The 5% offset method defines the yield point as the intersection of the load-deformation response with a line offset from the initial elastic stiffness by 5% of the fastener diameter, and it is commonly used for dowel-type connection testing and comparison with NDS yield predictions. In contrast, the EEEP method determines the yield point by using a bilinear idealization of the load-deformation response with equivalent energy dissipation, and it is widely used in cyclic connection testing and structural performance assessment. Both methods were used in this study to facilitate comparison with prior GiR connection research and to evaluate the sensitivity of the calculated yield strength to the selected interpretation method. Each specimen contained three rods; therefore, the predicted yield strength reported for the connection was taken as three times the predicted yield strength of a single rod.

The GiR connections in this study were loaded in interface shear, with each rod crossing the MPP panel-to-panel splice and loaded transversely to its longitudinal (strong) axis. This loading configuration is mechanically analogous to a single-shear dowel connection between two wood members, for which the EYM, as implemented in the NDS (AWC 2018), is directly applicable. The term “longitudinal splice” refers to the geometric arrangement of the panels, which were joined along their length and major strength axis, rather than to the loading direction relative to the rod. Because the load transfer mode (transverse rod loading) was consistent with EYM assumptions, the NDS yield limit equations were used to predict the reference lateral design values for the GiR connection.

Experimental results and discussion

Test results for the two specimens subjected to monotonic loading and nine specimens subjected to cyclic loading are reported in this section. In addition, comparison of the test results to existing design equations and use of acceptance criteria are described to facilitate future designs using these test results.

Monotonic loading test results

The first phase of testing featured two monotonic tests (MT1-2) to determine the specimen yield strength and reference force value (Q_0) for the cyclic CUREE protocol. The respective peak load and displacement at peak load recorded were 122 kN and 24.9 mm, and 126 kN and 16.5 mm, for an average peak load of 124 kN. Figure 8 shows the applied load versus interface relative horizontal displacement for both tests, where the displacement values were reported as the average of the SP1 and SP2 sensor displacements.

The yield strength for monotonic tests was calculated and compared using the 5% offset method and EEEP method. For the 5% offset method, the calculated yield strengths for the tests were 110 kN and 85 kN, for an average of 97.5 kN. For the EEEP method, yield strength values of 102 kN and 113 kN were calculated, for an average of 107.5 kN. Monotonic test results are summarized in Table 2.

Cyclic loading test results

The second phase of testing featured nine fully reversed-cyclic tests (CT1-9), where testing started on the pushing stroke, i.e., with the actuator applying a force on the specimen in the south

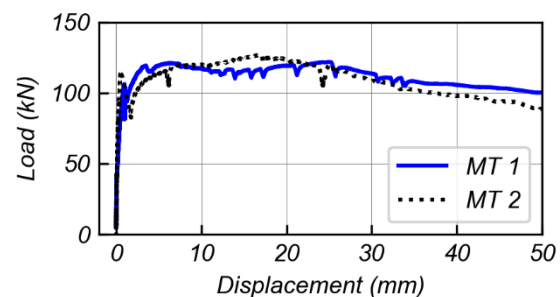


Figure 8. Monotonic tests: load versus relative horizontal displacement of the joint.

Table 2. Summary of monotonic test results.

MT No.	Peak load (kN)	Displacement at peak load (mm)	Yield strength, 5% offset (kN)	Yield strength, EEEP (kN)
1	122	24.9	110	102
2	126	16.5	85	113
Mean	124	20.7	97.5	107.5

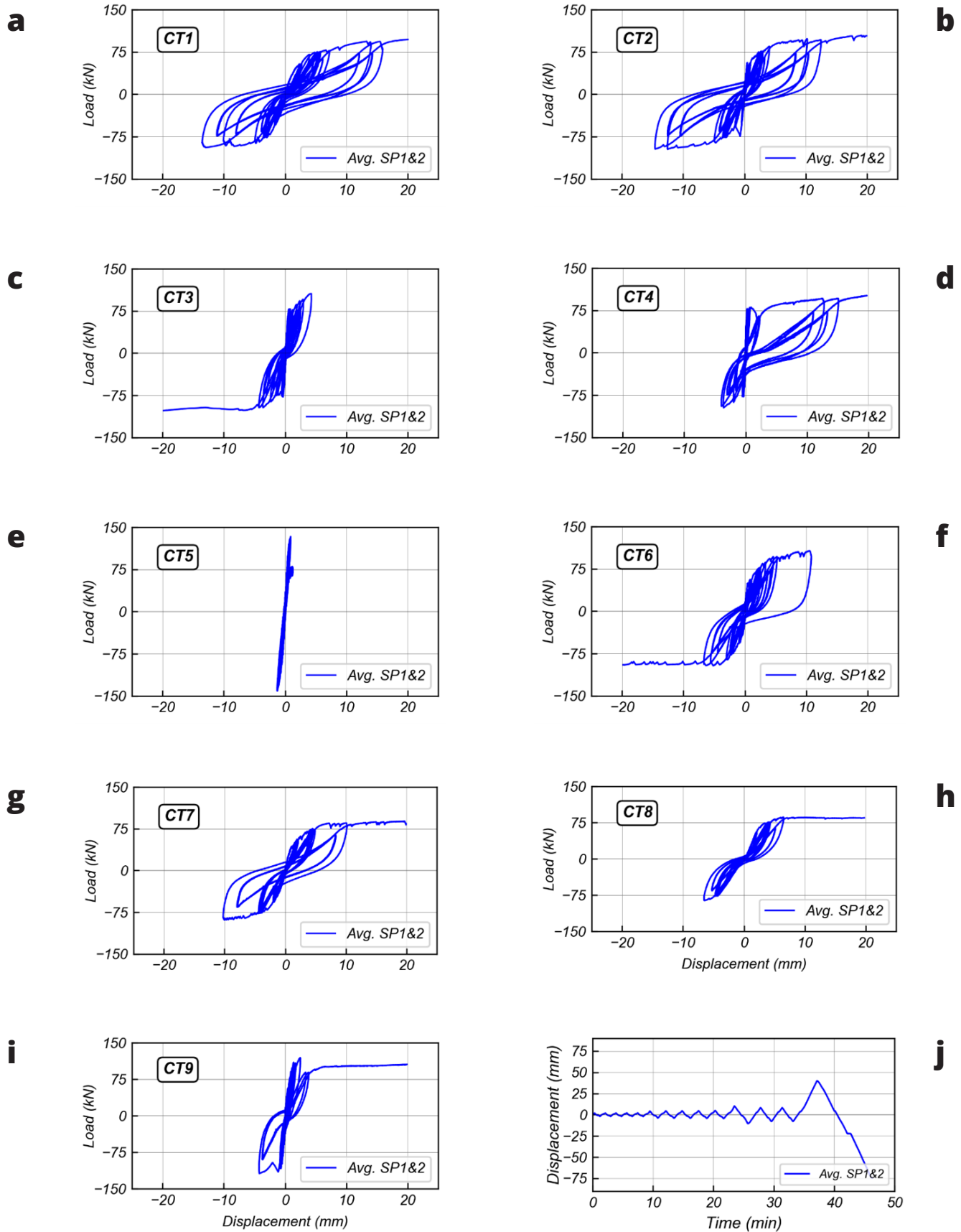


Figure 9. Measured load versus relative horizontal displacement along splice for all specimens tested under cyclic loading and example displacement versus time plot for specimen CT7.

direction. Figure 9a to Figure 9i show hysteresis plots, with recorded actuator loads versus the average of SP1 and SP2 relative horizontal displacement measurements at the joint interface, for the nine specimens plotted until 20 mm of relative displacement. It can be observed from these responses that CT3 and CT6 reached 20 mm on the pulling (negative) direction, while CT1, 2, 4, 7, 8, and 9 reached 20 mm on the pushing stroke. However, for CT5, the adhesive bond was not seen to fail prior to Part A twisting out-of-plane, leading to an early termination of testing, and no appreciable inelastic response. Figure 9j presents the relative joint displacement versus time plot for a representative specimen, where the largest relative displacements can be observed during the final cycle of testing.

The peak load and elastic stiffness values obtained from the cyclic tests exhibited two distinct groupings, hereafter referred

to as “Cluster 1” and “Cluster 2”. Figure 10 illustrates the separation of data into the two clusters, while the corresponding response values are summarized in Table 3 and Table 4, respectively.

The unintended accumulation of epoxy at the MPP panel-to-panel interface in a subset of specimens resulted in partial adhesive bonding across the interface, altering the load transfer mechanism from a purely mechanical dowel action to a combined adhesive-mechanical system. Because the goal of this study was to characterize the capacity of the mechanical GiR connection alone, the two groups were analyzed separately, with Cluster 1 providing the basis for the proposed design values. Accordingly, statistical comparisons were used primarily to identify trends, rather than to establish definitive population-level differences. Specimens exhibiting the additional interface epoxy (Cluster 2) consistently achieved higher peak loads than those without it (Cluster 1). Even with four to five specimens per cluster, the COVs of the peak loads were relatively small (7.2% for Cluster 1 and 11.5% for Cluster 2). Statistical comparison using a one-tailed t-test for two independent means indicated significant differences ($p < 0.05$) between the clusters in terms of peak load, elastic stiffness, and number of cycles to peak load (p -values = 0.0290, 0.000588, and 0.00651, respectively).

As mentioned, the differences observed between the two clusters are suspected to be attributable to variations in joint interface conditions resulting from specimen fabrication and assembly. Figure 11 and Figure 12 present representative pre-test and post-test interface conditions for a Cluster 1 specimen (CT8), while Figure 13 and Figure 14 show corresponding views for a Cluster 2 specimen (CT5). A comparison of the pre-test conditions shown in Figure 11 and Figure 13 indicates that Cluster 1 specimens generally exhibited a larger interface gap, within expected construction tolerances. Post-test observations further suggest that Cluster 2 specimens tended to accumulate epoxy between the adjoining parts near the GiRs, allowing an adhesive bond to develop across portions of the interface. This unintended adhesive contribution is therefore suspected to have altered the load transfer mechanism and contributed to the observed differences in response between the two clusters. Although the adhesive bond appeared to increase peak load capacity, its influence on elastic stiffness was substantially more pronounced.

Cluster 1 and 2 were also compared in terms of energy dissipation. Figure 15 illustrates the cumulative energy dissipation per cycle of testing. Energy dissipation per cycle was computed as the area enclosed by each load-deformation

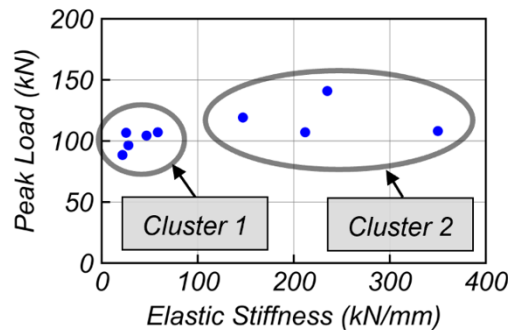


Figure 10. Clustering of cyclic test results.

Table 3. Cluster 1 test results.

CT no.	Peak load (kN)	Number of cycles to peak load	Elastic stiffness (kN/mm)
1	106.8	20.5	25.4
2	104.5	20	46.7
6	107.2	23	58.5
7	96.5	14.5	27.8
8	88.5	14	21.7
Mean	100.7	18.4	36.0
<i>COV</i>	7.2%	19.2%	39.3%

Table 4. Cluster 2 test results.

CT no.	Peak load (kN)	Number of cycles to peak load	Elastic stiffness (kN/mm)
3	107.2	23	212
4	108.1	29.5	350
5	141.0	38	235
9	119.2	28	147
Mean	118.8	29.6	236
<i>COV</i>	11.5%	18.2%	31.1%

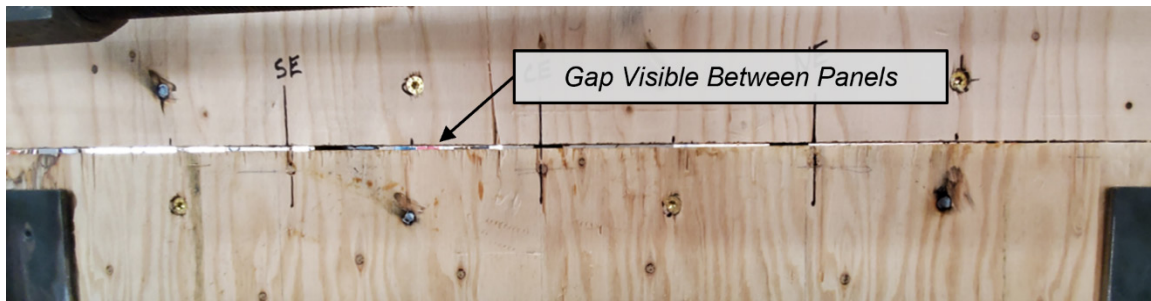


Figure 11. Example joint condition before testing for specimens in Cluster 1, showing visible gap between top and bottom elements (specimen from CT8 shown).

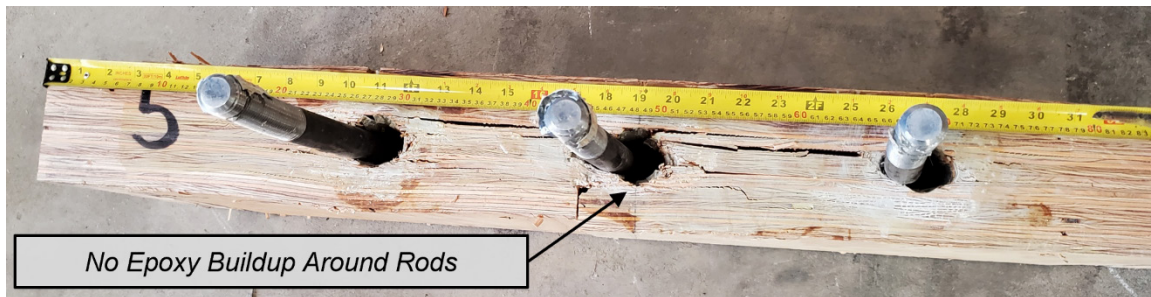


Figure 12. Example of panel interface after testing for specimens in Cluster 1 (specimen from CT8 shown).

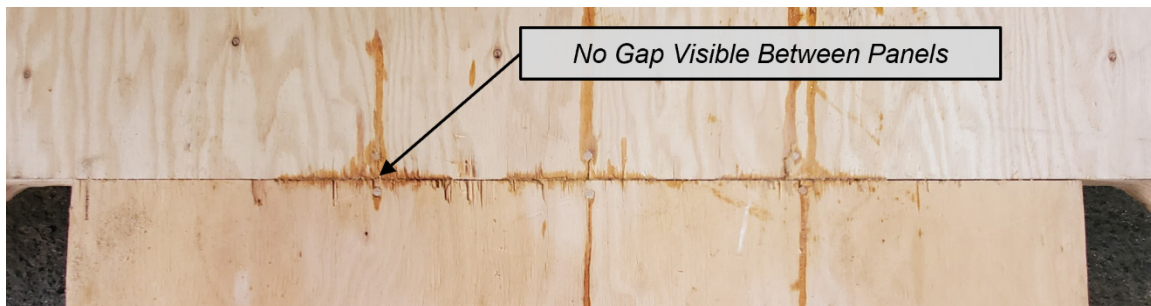


Figure 13. Example joint condition before testing for specimens in Cluster 2 (specimen from CT5 shown).

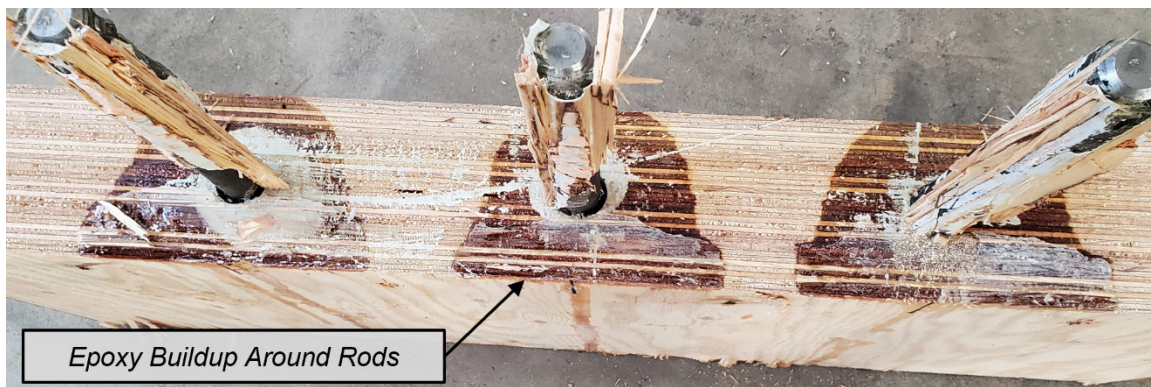


Figure 14. Example of panel interface after testing for specimens in Cluster 2 (specimen from CT5 shown).

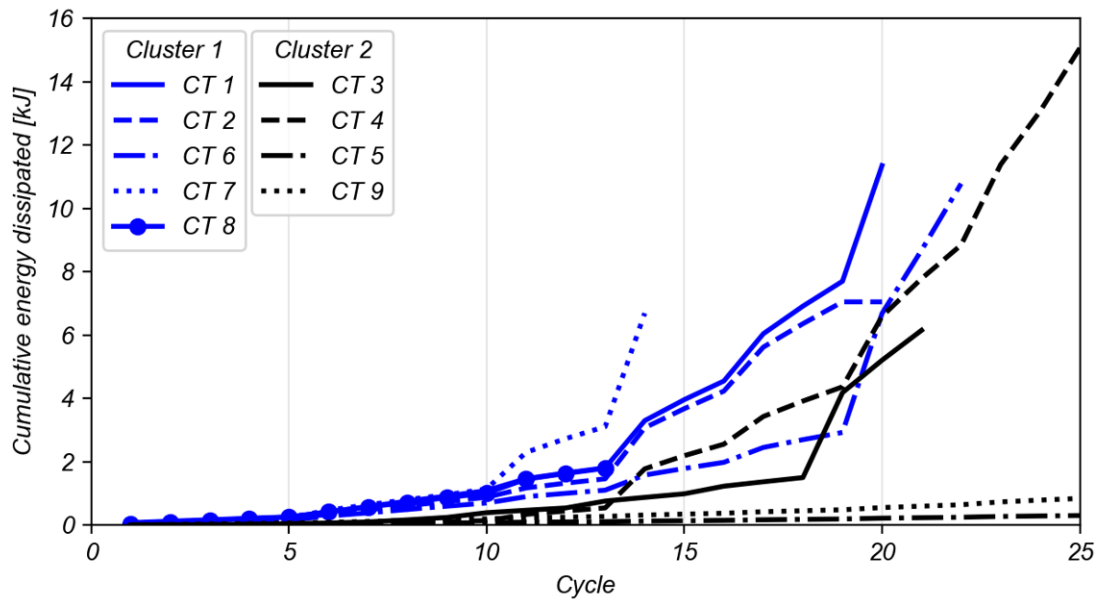


Figure 15. Cumulative energy dissipated in each cyclic test sorted by cluster.

hysteresis loop using trapezoidal integration of the recorded data, and the cumulative energy dissipation, i.e., the running sum of the per-cycle loop areas, was reported for the first 25 cycles of each test under the CUREE protocol. The purpose of this comparison was to evaluate the onset and accumulation of energy dissipation for the two interface conditions. Cluster 1 specimens dissipated energy from the early cycles of loading, whereas Cluster 2 specimens exhibited a delayed onset of energy dissipation, consistent with their initially stiffer, adhesive-dominated response prior to yielding (with CT5 and CT9 dissipating little energy). Given the limited number of specimens per cluster, statistical comparisons were used primarily to identify trends as discussed previously.

The condition of the GiRs was examined by dissecting the specimens at the conclusion of the tests. Some specimens contained rods that exhibited bending with a single plastic hinge located in Part A (yield mode III), while two plastic hinges were observed to have formed in other specimens (yield mode IV). Figure 16a shows an example specimen cross-section from CT3 displaying the formation of a single plastic hinge. Figure 16b shows an example specimen from CT2, where two plastic hinges can be observed. Plastic hinge formation was observed to occur at approximately 75 mm up/down from the joint interface. No significant correlation was found between yield mode and structural response of the connection.

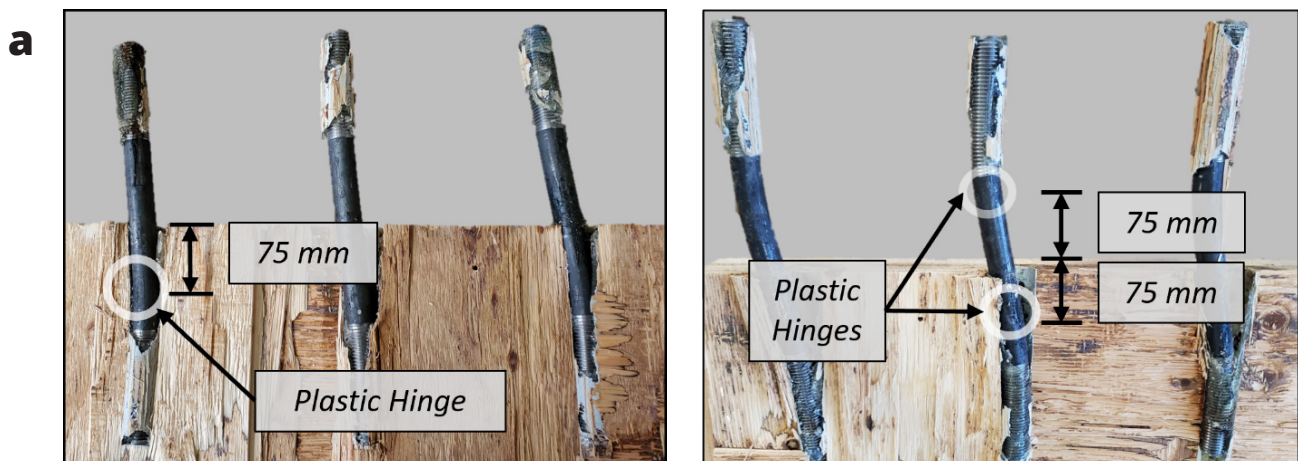


Figure 16. Cross section of dissected specimens: (a) CT3 exhibiting yield mode III; and (b) CT2 exhibiting yield mode IV.

Discussion and design values

The two clusters observed in the data demonstrate that the interface condition had a substantial influence on connection response. Specimens observed to contain epoxy accumulation at the panel interface (Cluster 2) were associated with increased peak load and elastic stiffness, relative to specimens without observed epoxy accumulation (Cluster 1). It is suspected that this behavior can be attributed to a combined adhesive-mechanical load transfer mechanism in Cluster 2, whereas Cluster 1 specimens were governed primarily by mechanical dowel action.

The distinction between the two clusters is important from a design perspective. Although the additional interface adhesion increased connection capacity, the amount of adhesive engagement was dependent on fabrication tolerances and epoxy accumulation that could not be reliably controlled or quantified using the proposed installation procedure. Consequently, reliance on this unintended adhesive contribution could lead to unconservative design assumptions. Because the objective of this study was to characterize the performance of the GiR connection itself, Cluster 1 specimens were considered more representative of the intended mechanical connection behavior and therefore provide a more appropriate lower-bound basis for the proposed design values.

Utilizing results from the Cluster 1 specimens, design values were calculated following the procedures established in ICC-AC526 (ICC-ES 2021) and described in the Data post-processing and reduction section. The nominal shear strength, taken as the mean peak load, was calculated as 100.7 kN for the 3-GiR connection and 33.6 kN for a single GiR. Applying the prescribed safety factor of 5 resulted in allowable GiR shear capacities of 20.1 kN and 6.7 kN for the 3-GiR connection

and a single GiR, respectively. The available shear strengths in LRFD were determined by applying format conversion factors to the allowable values, resulting in available strengths of 43.5 kN for the 3-GiR connection and 14.5 kN for a single GiR.

The reference lateral design values from the NDS were calculated to compare with experimentally derived design values (AWC 2018). For this calculation, the predicted yield strength was first determined using NDS yield equations presented in Table 1, and a dowel bearing strength of 18 MPa for MPP loaded in-plane, with veneers oriented perpendicular to the major strength direction (Miyamoto et al. 2020). NDS yield mode IV governed this calculation and produced a predicted yield strength of 85.3 kN for the 3-GiR connection. Finally, a reduction value ($R_d = 4$) was applied per Equation [1], and a reference lateral design value of 21.3 kN was obtained (P/R_d), corresponding to 7.1 kN per GiR. This value exceeds the allowable GiR shear capacity by a modest 5.6%. However, it is important to note the difference in the values of the NDS (AWC 2018) reduction term (4.0) and the factor of safety adopted in the ICC-AC526 (5.0, in ICC-ES 2021), which highlights the importance of testing to validate predicted and design values. Table 5 summarizes results for the nominal shear strength, allowable GiR shear capacity, available GiR strength in LRFD, and the NDS calculated reference lateral design value for the 3-GiR connection and for a single GiR.

The NDS predicted yield strength (AWC 2018) was also compared to experimentally derived yield strength values (presented in Table 2). This comparison showed that the mean values of the 5% offset and EEEP methods exceeded the NDS prediction by 14% and 26%, respectively. From pre-test visual inspection, it was suspected that monotonic specimens may also be influenced by adhesion at the interface. To confirm, a one-tailed t-test for two independent means was performed to

Table 5. Derived and predicted design values for Cluster 1 specimens.

GiR quantity	ICC-AC526, nominal shear strength (kN)	ICC-AC526, allowable GiR shear capacity (kN)	NDS, reference lateral design value (kN)	ICC-AC526, available GiR strength in LRFD (kN)
3	100.7	20.1	21.3	43.5
1	33.6	6.7	7.1	14.5

Table 6. Comparison of peak load between monotonic and cyclic tests.

Comparison group	Mean peak load (kN)	Difference (%)	p-value	Statistical difference
Monotonic	124	--	--	--
Cyclic (Cluster 1)	100.7	-18.7	0.0063	Significant
Cyclic (Cluster 2)	118.8	-4.2	0.344	Not significant

compare the monotonic test group with the cyclic test groups. It was calculated that the difference between peak load values for monotonic testing and Cluster 1 specimens was statistically significant $p < 0.05$ ($p = 0.0063$), while not statistically significant for Cluster 2 ($p = 0.344$), as summarized in Table 6. This finding agreed with observed joint conditions and provided additional evidence that monotonic testing specimens were similar to Cluster 2 specimens, explaining the higher experimentally derived yield strength. Using a dowel bearing strength of 18 MPa for MPP loaded in-plane with veneers oriented perpendicular to the major strength direction, NDS yield Mode IV (AWC 2018) governs and gives a predicted yield strength of 28.4 kN per rod. The mean experimentally evaluated yield strengths from the monotonic tests were 97.5 kN by the 5% offset method and 107.5 kN by the EEEP method, exceeding the NDS prediction by 14% and 26%, respectively. The comparison is based on the monotonic test results only. Cyclic tests were not included because the CUREE cyclic protocol is intended to characterize ductility and energy dissipation, rather than yield strength, and extracting yield points from a cyclic envelope introduces additional uncertainty. These results indicate that the NDS EYM provides a conservative estimate of the yield strength of the GiR splice connection for the configuration tested.

Several limitations should be acknowledged when interpreting the findings of this experimental program. First, the sample size was limited, and the comparisons presented herein are exploratory in nature and should be interpreted accordingly. Even with four to five specimens per cluster, the coefficients of variation of the peak loads were relatively small (7.2% for Cluster 1 and 11.5% for Cluster 2). Second, the test program addressed a single rod diameter, embedment depth, and adhesive type, so the applicability of the proposed design values to other configurations has not been experimentally verified. Third, all tests were conducted under short-term loading at ambient conditions; the long-term performance of these connections under sustained loads and environmental exposure, such as moisture cycling and temperature variation, remains to be characterized and is identified as a priority for future research. Additionally, the NDS yield capacity predictions presented here use conventional NDS dowel bearing strength values (F_{em}) for MPP (AWC 2018), which do not account for the full dowel-adhesive-wood load transfer sequence characteristic of GiR connections. Prior studies on axially loaded GiR connections have demonstrated that using adhesive-wood interface bearing strength, characterized through dedicated bearing tests, improves the accuracy of EYM predictions.

Future work should include bearing tests at the adhesive-MPP interface to refine the EYM-based design methodology for GiR shear connections in veneer-based panel products.

Conclusion

Interface shear tests were performed to characterize the structural behavior of GiRs in MPP. Monotonic loading tests were conducted per ISO 6891 (ISO 1983), and fully reversed-cyclic loading was performed according to a force-controlled CUREE protocol (Krawinkler et al. 2001). The following summarizes the primary findings and key observations from the experimental investigation and subsequent analytical evaluation:

1. Cyclic test results were grouped into two clusters based on a statistically significant difference in measured peak load and elastic stiffness. The mean peak load and elastic stiffness for specimens in Cluster 1 were 100.7 kN and 36.0 kN/mm, respectively, and 118.8 kN and 236 kN/mm for specimens in Cluster 2. From pre- and post-test observations, it is strongly suspected that the condition at the interface of the two parts of the specimens (accumulated adhesive) was the primary factor separating clusters.
2. Cluster 1 was used as the basis for conservative design values because it more closely represented the intended mechanical dowel-action behavior. For a single GiR, the nominal shear strength, allowable GiR shear capacity, and available shear strength in LRFD were calculated as 33.6 kN, 6.7 kN and 14.5 kN, respectively. However, the response observed in Cluster 2 is also informative and may represent an upper-bound condition associated with unintended interface adhesion. Additional testing is needed to more accurately characterize the structural response of this condition.
3. Comparison of experimentally derived design values with NDS-based predictions showed reasonable agreement. Using NDS yield mode IV (AWC 2018) and a dowel bearing strength of 18 MPa for MPP, the predicted reference lateral design value was calculated as 7.1 kN per GiR, which exceeded the experimentally derived allowable GiR shear capacity by 5.6%.
4. The experimentally evaluated yield capacity exceeded the NDS prediction by 14% based on the 5% offset method and by 26% based on the EEEP method (ASTM International 2005), indicating that the NDS EYM provides a conservative estimate of the shear capacity of the GiR splice connection for the configurations tested.

5. The divergence in connection response between Cluster 1 and Cluster 2 specimens underscores the importance of adhesive control during fabrication. Cluster 2 specimens, which had epoxy accumulated at the panel interface, exhibited substantially higher elastic stiffness (mean 236 kN/mm vs. 36.0 kN/mm) and modestly higher peak loads (mean 118.8 kN vs. 100.7 kN) relative to Cluster 1. The interface condition therefore governed stiffness far more than peak load. In both cases, the specimens exhibited ductile behavior consistent with yield modes III and IV. However, the delayed onset of energy dissipation in Cluster 2 reflects the initially stiffer, adhesive-dominated response before yielding. Future testing should explicitly characterize both interface conditions as controlled variables, using separate specimen groups fabricated with and without intentional interface adhesion, to establish distinct design recommendations. In the interim, fabrication protocols should incorporate measures to prevent unintended epoxy accumulation at the panel joint, such as temporary interface spacers or interface sealing prior to rod installation.
6. The findings of this study highlight the sensitivity of GiR shear connection response to interface conditions and fabrication tolerances. Future work should further investigate MPP interface behavior associated with unintended adhesive bonding and develop improved EYM-based prediction methods that better represent the dowel-adhesive-wood load transfer mechanism in veneer-based mass timber products.

Data availability statement

The data and codes generated related to this paper are available from the corresponding author upon reasonable request.

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Mass timber seismic lateral force resisting systems: Testing of a full-scale three-story structure

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Abstract: Over the last decade, the North American market for mass timber (MT) has diversified with the development of several engineered wood panelized products. Among these, a recent veneer-based product known as mass ply panel (MPP) has been introduced and certified per ANSI-PRG 320. To further demonstrate the potential of such engineered wood panelized products, a study was conducted to develop innovative solutions for enhanced design to low-damage seismic lateral force-resisting systems (LFRS). This paper presents an overview of the study, in which a three-story MT building was designed following performance-based design methods and constructed at Oregon State University (OSU). The gravity system includes laminated veneer lumber beams and columns and MPP floors. Two different LFRS for balloon-type MPP shear walls were studied. The first LFRS consisted of a pivoting MPP panel with steel buckling-restrained boundary elements (BRB) as energy dissipators. The second system utilized a rocking MPP panel with post-tensioned steel rods to provide self-centering capacity and U-shaped flexural plates (UFP) to dissipate energy. There were four testing phases, including two phases (Phases 0.1 and 0.2) that applied lateral loads to the gravity system only, before and after testing of LFRS; Phase 1, which included lateral testing of the building structure with an BRB-MPP shear wall; and Phase 2, which included lateral testing of the building structure with an UFP-MPP shear wall. In all four phases, the building was subjected to quasi-static cyclic tests following a CUREE protocol up to maximum 4% roof lateral drift. A summary of building details, experimental setup and key results is presented in this paper.

Keywords: Buckling restrained braces; CUREE; Earthquake engineering; Lateral force-resisting system; Mass timber; Mass ply panel; Performance-based design; Quasi-static cyclic test; U-shaped flexural plates.

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Introduction

Mass timber (MT) buildings have experienced a surge in popularity in recent years, as is evident through successful projects employing exclusively mass timber structures or incorporating various mass timber components alongside steel and concrete systems (Kılınç 2025; Fernandez et al. 2020; Liven and Abrahamsen 2023). Ongoing advancements in research and technology are enhancing the seismic resilience of mass timber buildings. In line with efforts to design structures that withstand strong ground shaking with minimal damage, numerous tests have been conducted to assess the structural performance of mass timber buildings. One noteworthy example of a low-damage lateral force resisting system (LFRS) is the self-centering rocking wall system. This innovative system comprises a rigid, primarily elastic wall element, post-tensioning rods, and energy dissipation devices (Granello et al. 2020).

To assess the performance of self-centering rocking wall systems employing cross-laminated timber (CLT), a two-story mass timber building underwent testing at the University of California San Diego (UCSD) National Science Foundation's Natural Hazards Engineering Research Infrastructure (NHERI) shake-table facility in 2017 (Blomgren et al. 2019; Mugabo et al. 2021; Pei et al. 2019). Furthermore, a comprehensive body of literature documents component tests focused on self-centering rocking wall systems utilizing mass timber elements (Araújo R. et al. 2025; Brown et al. 2021; Ganey et al. 2017; van de Lindt et al. 2019; Pei et al. 2024; Sarti et al. 2015, 2016; Uarac et al. 2025). With the continuous production and certification of new mass timber panels (MTPs) and the introduction of additional designs for seismic force-resisting systems, there are increased opportunities for the development of innovative LFRS.

This paper summarizes a multi-phase research project that includes construction of a full-scale three-story building structure specimen designed to test two different LFRS. There were four testing phases, including two phases (Phases 0.1 and Phase 0.2) that applied lateral loads to the gravity system only, before and after testing of LFRS. In Phase 1, lateral loads were transferred through mass ply panel (MPP) diaphragms to an LFRS consisting of an MPP spine with vertically oriented energy-dissipating buckling-restrained braces (BRBs) in the first story. In Phase 2, the LFRS consists of an MPP, post-tensioning rods connecting the top of the panel to the foundation, and U-shaped flexural plates (UFPs) that provide energy dissipation capability. These UFPs are attached

between the MPP and boundary steel columns (designed to remain essentially elastic) that connect to the foundation. In both Phase 1 and 2, the LFRS is connected to the diaphragms of a mass timber gravity system, consisting of MPP floor slabs and laminated veneer lumber (LVL) beams and columns. The gravity connections were designed to remain damage free up to approximately 5% of inter-story drift ratio. The connection between the LFRS and the diaphragms only allows for transfer of lateral loads between the MPP wall and floor panels, so the rocking mechanism is not affected by the gravity system (Barbosa et al. 2021; Blomgren et al. 2019; Mugabo et al. 2021). The design includes force-based design procedures for the gravity system, and displacement- and performance-based seismic design procedures for the LFRS. Additionally, drift and force capacity design checks were performed for various components of the gravity system and connections.

The aim of the test program presented in this paper was to investigate the behavior of a full-scale three-story mass-timber building system equipped with two different LFRS, consisting of an MPP spine and a resilient rocking wall. The intention was to validate the design process and replicate the construction details required for practical implementation. The specific objective was to estimate the global load-displacement behavior and displacement capacity of the two different systems. Additionally, the test was conducted to evaluate the structural system's resilience by subjecting it to significantly high drift levels, surpassing the limits required by current building codes, thus examining and validating its robustness.

This study is part of larger experimental campaign on innovative lateral systems employing mass timber as part of the Emmerson Lab Launch Initiative (ELLI). An overview of the experimental campaign and notable results is presented in this paper. Detailed results and calibrated numerical models from different phases of ELLI can be found in (Araújo et al. 2025; Uarac et al. 2025).

Materials and methods

Design requirements

The test building represents a portion of an 18.3 m (60 ft) by 30 m (100 ft) three-story office archetype building located in Seattle, WA (47.582 N, 122.331 W), site class D ($S_{DS} = 0.97$ g, $S_{DI} = 0.61$ g). Design gravity loads consisted of dead loads of 2.87 kPa (60 psf) for the second and third floor levels and 2.63 kPa (55 psf) for the roof. Live loads considered were 3.11 kPa (65 psf) for the floor levels and 0.96 kPa (20 psf) for the roof. The total seismic weight included in the seismic design was

5,289 kN (1,189 kips), which was evenly distributed over six shear walls (in each loading direction) of the office building archetype. Note that the seismic weight did include live load per ASCE 7-16 (ASCE/SEI 2017).

For the seismic design, three hazard levels were considered in the overall project: (1) service level earthquake (SLE); (2) design earthquake (DE); and (3) risk-targeted maximum considered earthquake (MCE_R). However, in the design of the two different LFRS, these limits were applied differently. The pivoting mass timber-steel BRB system utilized only the DE and MCE_R drift targets, adopting story drift ratio limits of 2% per ASCE 7-16 (Table 12.12-1) and scaled by 3/2 (3%) for the MCE_R level (Araújo et al. 2025). The post-tensioned UFP wall system employed the same DE and MCE_R drift limits and additionally incorporated an SLE drift target of 0.5% to limit nonstructural damage (Uarac et al. 2025). Additional performance expectations were defined for three hazard levels (SLE, DE, and MCE_R) and are presented in the following sections for both the LFRS employed.

Gravity framing

The overall footprint of the test building was 12.19 m $\times$ 12.19 m (40 $\times$ 40 ft) (Figure 1). The first story height was 3.15 m (10 ft 4 in.) from the top of foundation beams to the top of floor, and the height of second and third stories was 2.74 m (9 ft), leading to a total building height of 8.64 m (28 ft 4 in.). The gravity system was designed symmetrically to create a two-bay by two-bay configuration. There were 12 LVL columns (178 mm $\times$ 178 mm (7 in. $\times$ 7 in.) total in each story. LVL beams, with cross-section of 133 mm $\times$ 559 mm (5.25 in. $\times$ 22 in.) spanned column axis to column axis in the north-south direction. In the first story, columns rested on LVL foundation

beams with a combined cross-section of 356 mm $\times$ 356 mm (14 in. $\times$ 14 in.). Beams and columns consisted of Douglas-fir 2.1E 3100 and 1.8E 2650 rated LVL (ICC and APA 2025), respectively. The spacing between adjacent collar beams along the building centerline in the loading direction was 490 mm (1 ft 7-1/4 in.), providing sufficient clearance between beams to accommodate the shear walls.

The beam-to-column and column-to-foundation-beam connections were steel column caps (Figure 2a and Figure 2b, respectively) were custom made and allowed rocking at the beam-to-column interface to accommodate large, up to 5%, inter-story drift ratios of the building. This connector design, which is like the one utilized by Pei et al. (2019), adopts vertical slotted bolt holes to minimize moment transfer and damage to the joints. In the east-west direction, a steel angle member was added to each beam-to-column connection to enhance out-of-plane stability of the beam (Figure 2a).

Diaphragm

The diaphragm at each floor level consisted of twelve 181 mm (7-1/4 in.) thick MPP panels, which spanned in the east-west direction, i.e., perpendicular to the loading direction and fixed to LVL beams with SDCP 221100 screws (ICC 2025) spaced at 305 mm (12 in.) on-center (Figure 2c). Adjacent diaphragm panels were connected by 149 mm (5-7/8 in.) wide spline connections of 19 mm (3/4 in.) thick plywood and two rows of SDCP 22434 screws (ICC 2025) (Figure 2d). The diaphragm was designed to remain essentially elastic for the various phases of this project. The lateral loads used for diaphragm design were determined following both the traditional and alternative design methodologies described in Chapter 12 of ASCE 7-16 (ASCE/SEI 2017), and in accordance with the SDPWS 2021 (AWC/ANSI 2020) for diaphragm shear force.

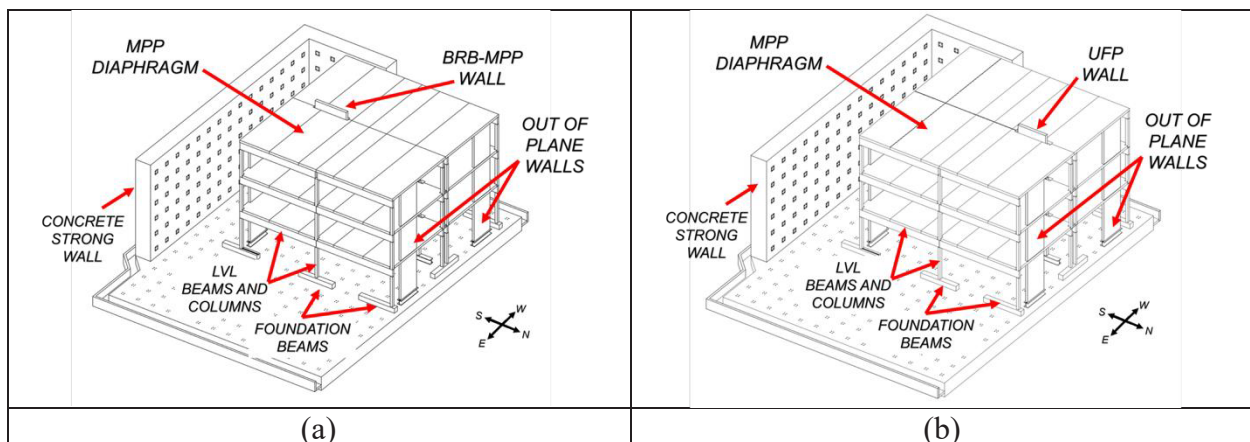


Figure 1. Three-dimensional view of test building: (a) Phase 1 – pivoting mass timber-steel BRB system; (b) Phase 2 – post-tensioned UFP wall.

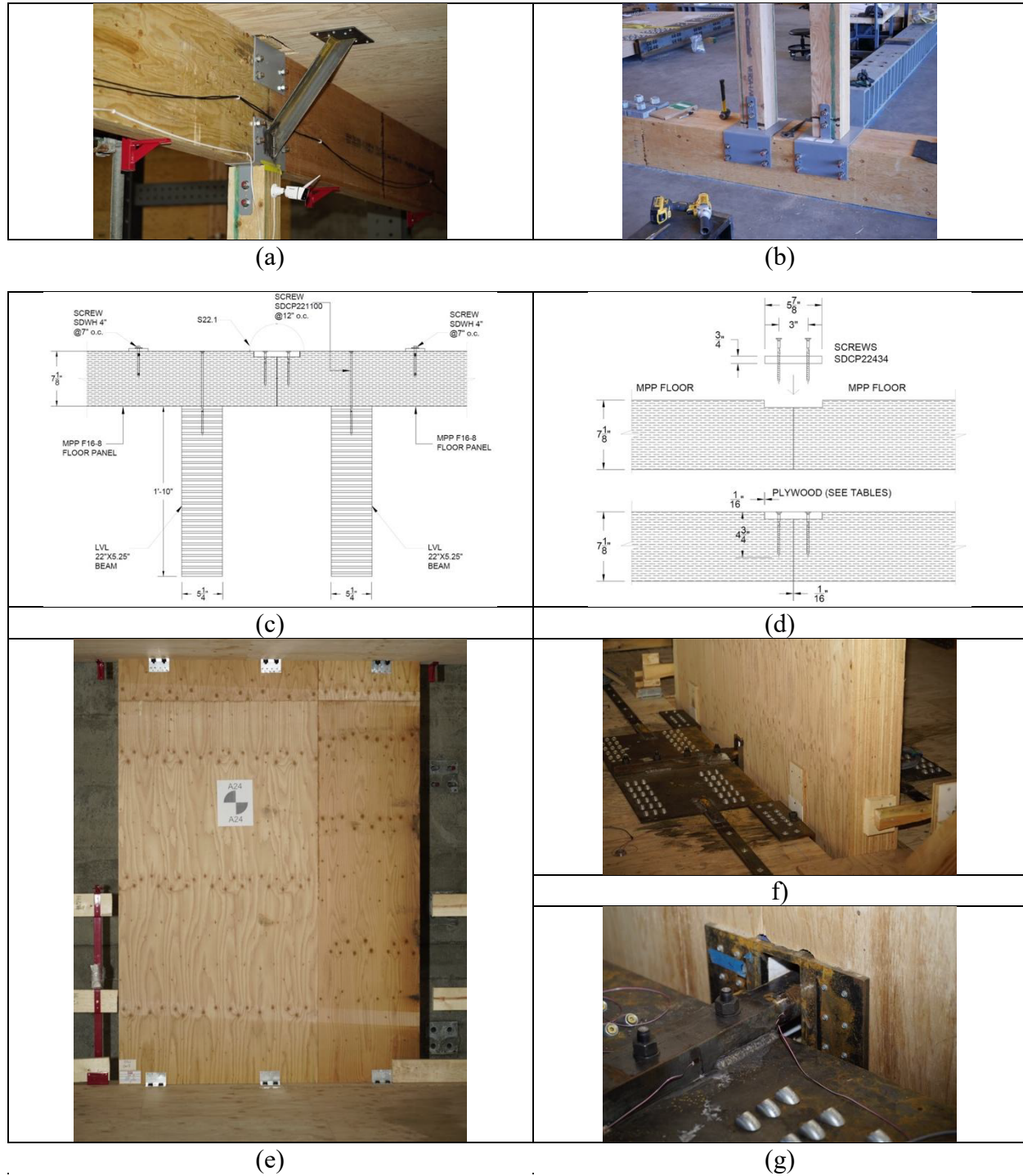


Figure 2. Building connection details: (a) beam-to-column connection; (b) column-to-foundation connection; (c) diaphragm-to-beam connection; (d) diaphragm spline connection; (e) out-of-plane wall connection; (f) shear transfer mechanism; and (g) steel window, shear key, wing plate and shear collector.

Out-of-plane walls

Four platform-type out-of-plane shear walls were placed at each story perpendicular to the applied loading direction to improve the torsional stability of the building (see Figure 1). Each out-of-plane wall was 127 mm (5 in.) thick, 1.83 m (6 ft) wide MPP

panel connected to top and bottom diaphragm panels using A3 connectors (Amini et al. 2018; AWC/ANSI 2021) (Figure 2e). Because lateral loading during testing was applied only in the north-south direction, these out-of-plane walls were designed to resist up to 10% of the estimated north-south capacity.

Wall-to-diaphragm connections

At each floor level, the lateral load was transferred from diaphragm to the shear wall through a slotted shear transfer mechanism (i.e., shear wall-to-diaphragm connection) consisting of a shear key attached to the panels and wing plates (Figure 2f). The design of wall-to-diaphragm connection was based on the connection design in the two-story building shake table test (Blomgren et al. 2019; Pei et al. 2019) with some modifications, including a two-sided load transfer, unlike the shake table testing, and where welding was added between the shear key and wing plate to increase the strength and effective stiffness of the connection. This design allows transferring effectively shear force from the diaphragms to the shear walls and constraining out-of-plane movement of the shear walls, while allowing uplift of the wall panel relative to the diaphragms. At each floor level, a steel window was inserted into a prefabricated cut-out on the MPP wall panel. A shear key was assembled through the window. The segment of shear key, which contacted with the steel window, was rounded at its four corners. In addition, low friction PTFE plates were glued to the inside faces of steel windows. These details allow smooth relative movement between shear keys and steel windows when the wall uplifted (Figure 2g). To transfer lateral load from diaphragm to the shear wall, each shear key was connected to one steel wing plate on each side of the shear wall by a combination of four 3/4 in. diameter tension-controlled bolts and weld connections. Each wing plate was attached to the floor panels by 60 inclined SDCF 22858 screws (ICC 2025). Shear collectors, including two steel plate segments on each side of shear wall, ran across diaphragm panels in a north-south direction and were tied down to diaphragm panels by SDWH 27400G screws (IAPMO 2025) spaced at 178 mm (7 in.) on-center and welded to wing plates for collecting and transferring lateral loads from the diaphragm panels to wing plates.

Pivoting mass timber-steel BRB system

The first LFRS included a hybrid of a 9144 mm $\times$ 2515 mm $\times$ 203 mm (30 ft $\times$ 8 ft-3 in. $\times$ 8 in.) MPP shear wall and two BRBs attached to the shear wall ends as hold-downs at the first story level, herein referred to as the hybrid BRB-MPP spine, essentially acting as stiff backbone to the structure. The BRBs were designed to entirely resist the base moment, while the MPP wall resisted the story shear. This BRB-MPP spine wall was conceived as a pivoting wall with a pin support located at the base midpoint to allow the wall to jointly pivot (Figure 3a). Shear transfer to the foundation was ensured by using two stiff steel shear key assemblies, designed as slip-critical connections

to the steel foundation beam. These assemblies prevented in-plane sliding and restrained out-of-plane displacement of the wall relative to the foundation (Figure 3b).

The design base shear, V_b , was estimated using the DE spectrum from ASCE 7-16 and a response modification factor of $R = 8$, consistent with BRB-frame systems (ASCE 7-16 [ASCE/SEI 2017] Table 12.2-1, item 25). The base shear was distributed at each floor level as equivalent static lateral forces based on the first-mode shape. Because the BRBs were intended to yield primarily under first-mode overturning demands, the MPP spine was proportioned to remain essentially elastic (Araújo et al. 2025). As discussed in that work, yielding of the BRBs does not limit higher-mode inertial forces, which can generate additional story shear demands beyond those associated with first-mode behavior. Therefore, the dimensions of the MPP panel were also verified for near-elastic story shears resulting from these higher-mode contributions to ensure the intended elastic response of the spine under all modes of excitation.

Strain demands in the yielding core of the BRBs were also limited to the minimum between 2.5% and 10 times the steel's yield strain at the MCER level to mitigate fracture in tension. The spine components (i.e., the MPP wall and BRB-MPP connections) were capacity designed for the expected force demands at a target story drift ratio of 4% to achieve enhanced performance at intensities higher than the risk-targeted maximum considered earthquake (MCER).

The connections at the ends of the BRBs were designed to transfer the tensile and compressive forces delivered by the BRBs. The connection plates incorporated horizontal slotted holes and custom slotted washers, a configuration that provided high axial stiffness, while allowing the required relative motion (Araújo et al. 2025). This detail ensured reliable force transfer without restricting the rotation of the BRB-MPP spine. The selected BRB-to-MPP connection assembly included two 10-mm (3/8-in) side plates connected to the MPP through screws inclined at a 45° angle. The side plates were connected to a 38-mm (1.25-in.) thick base plate (termed “bottom plate” in Figure 3b), which ultimately connected to a 19-mm (0.75-in.) thick gusset plate, forming a T-shaped connection. The gusset plate was bolted to the ends of the BRBs (termed as “BRB lugs” in Figure 3b). The provided detail takes advantage of the high strength and stiffness of inclined screws in tension, while compressive forces are transferred through the bearing of the MPP on the bottom plate. To promote contact between the MPP and the bottom plate, 10 SDCF27400 (ICC 2025) self-driving screws oriented upwards were added to the con-

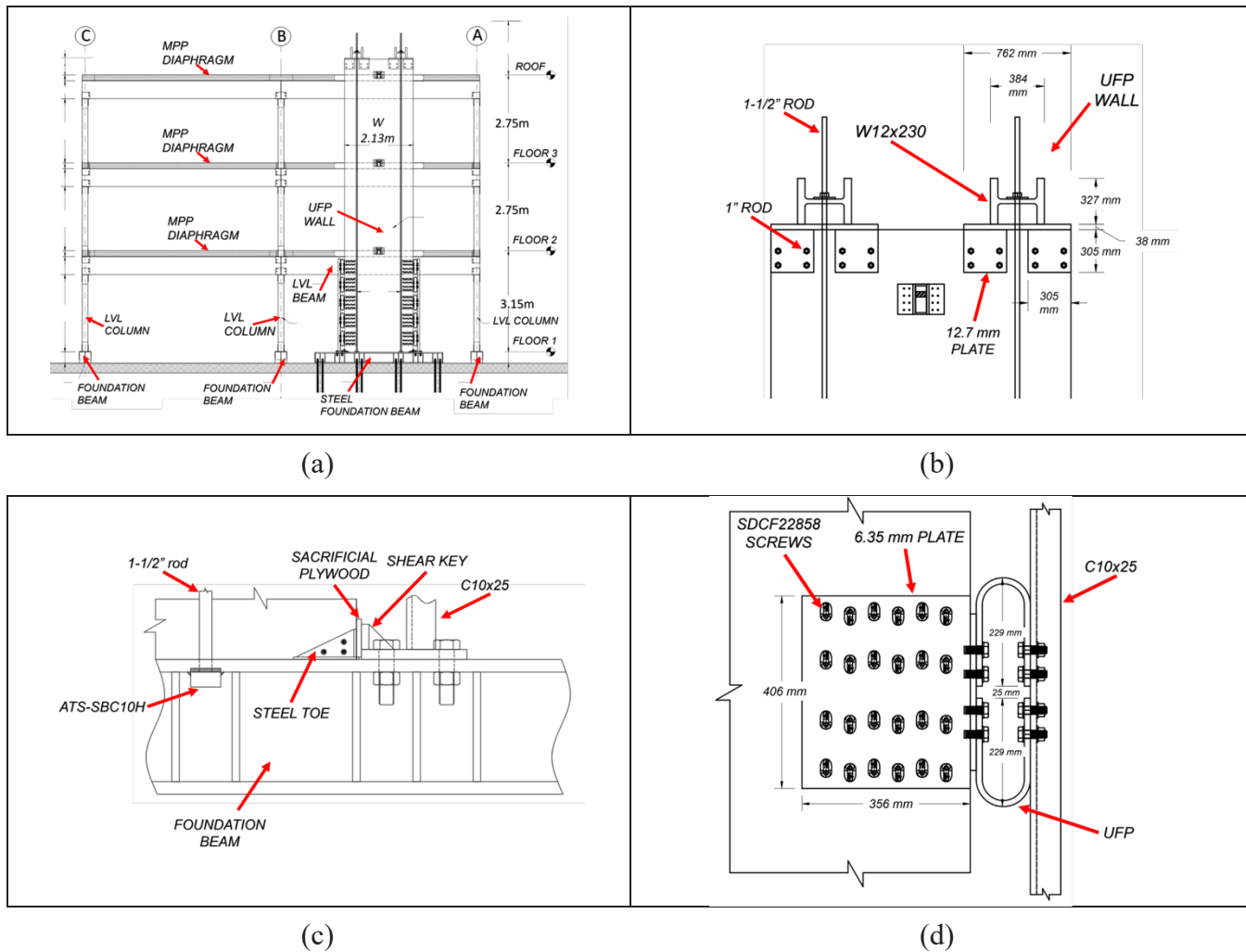


Figure 4. (a) Elevation view of test building with rocking wall; (b) steel anchor saddle at top of MPP wall; (c) steel anchor plate welded to foundation steel beam, shear key and steel toe details; and (d) UFP connection details.

sipate energy during a seismic event (Figure 4c). There were 20 UFPs [ASTM A572 Grade 42 (ASTM 2021); width of 152 mm (6 in.); thickness of 12.7mm (0.5 in.); and bending diameter of 102 mm (4 in)]. To prevent the sliding movement of the rocking wall, two foundation shear keys were designed and integrated to the base plates of steel columns, which were bolted to the steel foundation beam (Figure 4c). A gap of 13 mm (0.5 in.) was left between a shear key and each end of the wall and was filled by a sacrificial plywood piece to prevent the crushing of MPP in a minor, perpendicular-to-the-surface direction. In addition, a steel toe was included at each bottom corner of the rocking wall to reduce the crushing of MPP in a major direction at the rocking interface.

The UFPs were anchored to the wall through a welded connector assembly comprising two plates bearing against the

wall faces and a third plate at the wall edge. These three plates formed a U-shaped bracket that provided the seat for attaching the UFPs as shown in the Figure 4d. Inclined SDCF 22858 screws (ICC 2025) were used for U-shaped bracket-MPP wall connection, while 5/8 in. A325 bolts (ASTM 2025) were used for UFPs-steel column and UFPs-U-shaped bracket connections.

The performance-based design procedure for the UFP rocking wall LFRS was based on the direct displacement-based design (DDBD) procedure (Di Cesare et al. 2019; Pang et al. 2010) and utilized a flag shape hysteretic model (Nakaki et al. 1999; Priestley and Grant 2005) to estimate the effective, equivalent viscous damping for the whole wall system. For the DE level, an effective damping ratio of 10.36% was adopted, following the procedure described in Uarac et al. (2025). The

design base shear was estimated using the capacity spectrum approach (Pang et al. 2010), assuming an inverted-triangular displacement shape for the system (Newcombe 2011; Priestley et al. 2007). Strength design was used to select the shear wall component dimensions, number of UFPs and rods, and other design parameters. The moment capacity of the shear wall and stresses in the post-tensioned rods and MPP were checked by an analytical procedure based on the Modified Monolithic Beam Analogy (MMBA) (Palermo et al. 2004; Uarac et al. 2025). In this approach, a nonlinear finite-element model was not needed during this design stage.

There are three performance levels corresponding to three hazard levels (SLE, DE, and MCER) considered in the design. Beside the peak inter-story drift (which was discussed in the Design requirements section), the stresses and strains of the post-tensioning rods and MPP are additional design criteria for each performance level. The strains in the post-tensioning rods and MPP were limited to the elastic range at the SLE and DE levels, but were allowed to yield at the MCER level. For the MPP, an elastic-plastic model was assumed, with a yield stress of 43.01 MPa (6.23 ksi), yield strain of 0.0032 (DE limit), and ultimate strain of 0.0075 (MCER limit), based on results in Soti et al. (2021). The high-strength rods were assigned a yield strain of 0.0065, using a 0.2% offset from the elastic limit (ASTM 2017). The ultimate strain of 0.085 was taken from Ocel and Provines (2015).

Test building construction

The construction of the test building proceeded in several coordinated stages to integrate the gravity framing, diaphragm system, and the different LFRS used across the testing phases. Construction began with the installation of the building foundations to the strong floor. Steel beams were placed as foundation elements beneath the LFRS and the out-of-plane walls, while LVL beams were used as foundations beneath the gravity-system columns. With the foundation system in place, the out-of-plane walls, the LVL gravity framing, and some diaphragm panels of the first story were assembled. After the first-floor gravity system was erected, both LFRS, each installed with its respective energy-dissipation system (BRBs and the UFPs), were positioned in place but intentionally left unconnected to the diaphragm panels. This allowed the remaining floor panels and all diaphragm hardware to be installed without interference, and enabled the uninterrupted erection of the gravity system for the upper stories.

After installation of each lateral system, the remaining stories were erected, following the same sequence of installing LVL

columns, out-of-plane walls, beams, and diaphragm panels. This repeated procedure ensured consistent construction, detailing, and boundary conditions across all testing phases.

For each of the four testing phases, the corresponding lateral system was then connected to the diaphragm. In Phases 0.1 and 0.2, no wall was attached, in order to evaluate the gravity system response in isolation. In Phase 1, the pivoting mass timber-steel BRB system was connected, using the shear-transfer mechanism described previously. For Phase 2, the BRB wall remained in place but was left disconnected from the diaphragms, while the post-tensioned UFP wall was attached instead to evaluate the self-centering system.

Quasi-static building test

Test setup and loading protocol

Three actuators, which reacted against a 7.6 m (25-ft) tall by 18.3 m (60-ft) long strong wall, were attached to diaphragms to apply the load to the building. The loading direction, which is parallel to the investigated LFRS, is in the north-south (N-S) direction shown in Figure 1. A quasi-static cyclic displacement control-loading protocol (Figure 5) was used, which followed an abbreviated version of the CUREE protocol (Krawinkler et al. 2000). The displacement in the CUREE protocol was based on the actual building roof movement measured by the string pot at the wall line. The top actuator worked as a primary actuator and was set to displacement control, while two lower hydraulic actuators were set as secondary (follower) actuators and were set to force-based mode, following the primary actuator. The loading rate of 40.6 mm/min (1.6 in/min) was utilized for the primary actuator at the initial small cycles up to 0.4% drift. Then, the loading rate was reduced to 20.3 mm/min (0.8 in/min) for displacement cycles larger than 0.4%. The system of

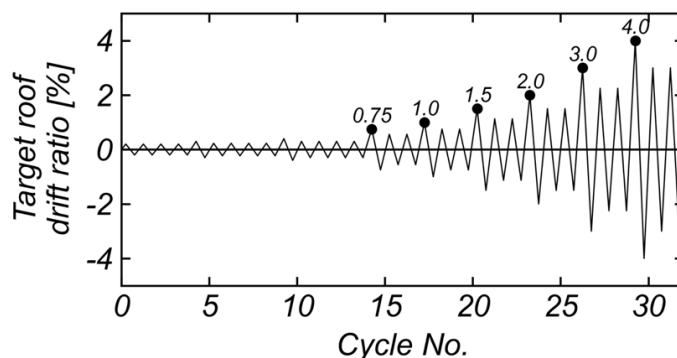


Figure 5. Abbreviated CUREE protocol utilized in the quasi-static cyclic test.

three actuators created a triangular force profile applying to the building and being proportional to the elevation of the actuators. The force feedback from the primary actuator was used to determine the force commands for the secondary actuators, such that the actuator at the first and second story were set to 0.357 and 0.679 times the primary actuator force, respectively. The load from actuators was transferred through steel loading beam to diaphragm panels on both sides of the shear wall line.

Instrumentation

The lateral loads applied to the test building were measured using load cells attached to the hydraulic actuators. The real-time reading data from central string potentiometers at roof level (SP11) was utilized to control the primary actuator movement following the loading protocol (Figure 6a). In addition, at each floor level, there was one string potentiometer connected between walls (SP1, SP5, SP9, Figure 6a) to record

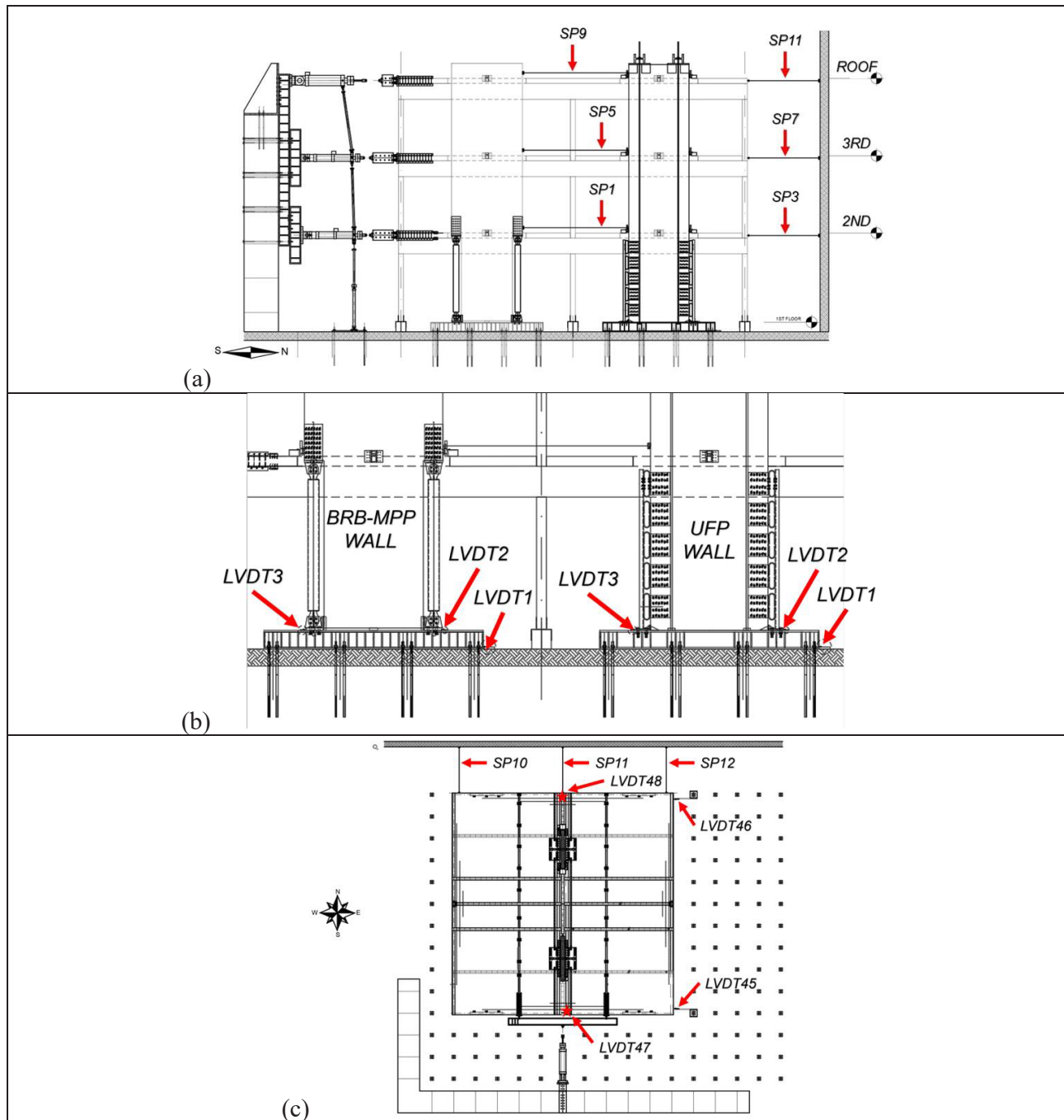


Figure 6. Instrumentation arrangements for observation of global building behavior: (a) string potentiometers for story and wall displacement; (b) LVDTs measuring beam foundation and wall sliding; and (c) string potentiometers LVDTs measuring torsion and out-of-plane movement at the roof.

the in-plane movement of the walls. At the ground level, three LVDTs were arranged to measure potential sliding movement of the steel foundation beam and two foundation shear keys (Figure 6b). Note that the same sensor IDs were used for Phase 1 and Phase 2.

At each floor level, there were three string potentiometers attached to diaphragm ends at the north side of the building to monitor the building displacements and potential building torsion and relative diaphragm movements on the north face. On the other hand, there were two LVDTs installed at the northeast and southeast corners of the diaphragm. Figure 6c shows the roof sensors as example of the setup.

The experiment involved tracking various aspects of the diaphragm’s performance during the lateral movement of the building, under the double cantilever scenario (N-S direction). A pair of LVDTs on each floor were used to measure the opening of the diaphragms near the ends. Figure 6c shows the roof setup for this purpose (LVDT47 and LVDT48). Additionally, LVDTs were used to measure relative movement of the wing-plate to the MPP and relative movement of the tongue plate to wing plate, while strain gauges were used to measure strains in steel components, such as collectors and tongue plates. Figure 7 shows the reference location of the LVDTs at the roof. Detailed instrumentation drawings can be accessed in DesignSafe (Sinha et al. 2024, 2025).

Table 1 summarizes the instrumentation used during the testing program across the four phases. The table lists the sensor types, quantities, general locations, testing phases in which they were deployed, and a general description of the measurements they captured. LVDTs were primarily used to monitor local and global displacements of wall panels, diaphragms, and foundation components. String potentiometers captured

in-plane wall movements and global floor displacements, while strain gauges measured axial strains in BRBs, PT rods, and diaphragms. The table highlights the comprehensive coverage of both local and global structural responses, ensuring that the behavior of critical elements was accurately recorded throughout the experimental program.

Additional instrumentation for BRB-MPP spine

The instrumentation at the base of the MPP spine consisted of two LVDTs at both sides of the MPP boundaries and one LVDT at the pivot support to measure uplift resulting from the tilting-mode behavior of the spine. LDVTs were also used to measure the axial deformation of the BRBs (two per BRB) and local deformations in the connections (four per BRB). Four strain gauges were installed in the elastic regions of each BRB (two on each face) and in the pivot-support rods (one per rod) to estimate axial forces in these elements. Other LVDTs not

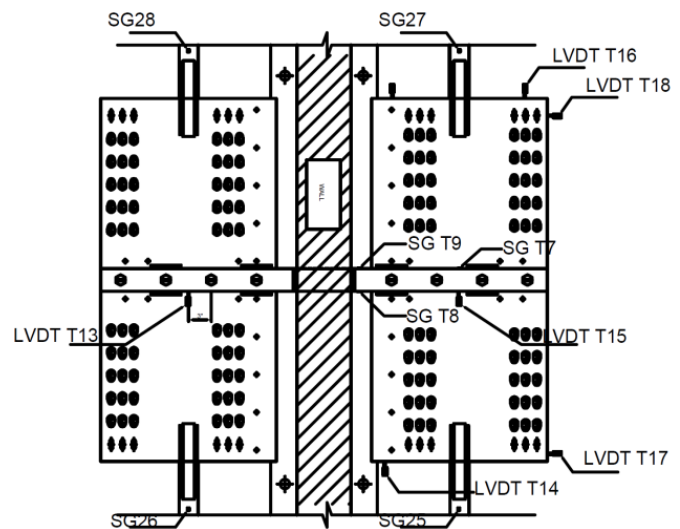
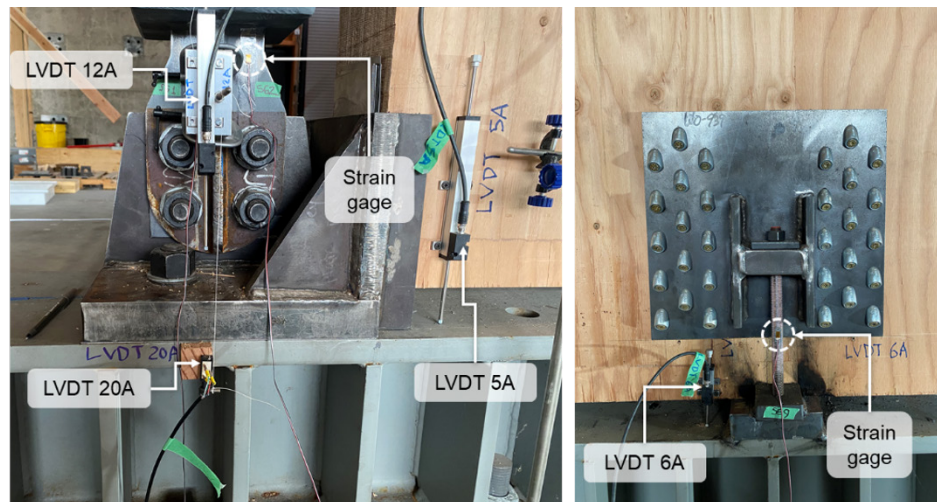


Figure 7. Wing and tongue plates instrumentation for roof.

Table 1. Instrumentation summary.

	Total quantity	Location	Phases(s) used	Measurement
LVDT	23	BRB wall	1	Local response at BRBs, wall panel, connections, and relative to diaphragm
	23	MPP wall	2	Local responses at the wall panel, UPFs, and relative to diaphragm
	6	Diaphragm-to-wall	1 and 2	Relative horizontal wall panel to diaphragm
	3	Foundation beam	1 and 2	Sliding
	6	Diaphragm	All	Global out-of-plane displacement
	30	Diaphragm	All	Local responses at the splines, collar beams, and shear key-to-wing plate
String pot	3	Wall-to-wall	1 and 2	In-plane wall displacement at floor levels
	9	Diaphragm	All	Global floor displacement
Strain gauges	8	BRB wall	1	BRB strains
	8	PT rods	2	PT rod strains
	24	Diaphragms	All	Strain at shear collector



LVDT 5A and 6A: uplift of the MPP as a result of the pivoting mode of the spine.
LVDT 12A: axial deformation of the BRBs from the center of the bolt group located at the bottom to the center of bolt group at the top.
LVDT 20A: relative deformation between the center of the bolt group located at the bottom and the top flange of the foundation beam.
Strain gages: strain in the elastic region of the BRBs and the pivot-support rod.

Figure 8. Instrumentation arrangements for pivoting wall.

shown in Figure 8 were installed at the base to measure lateral slip of the foundation beam and out-of-plane displacements at the base of the MPP spine.

Additional instrumentation for post-tensioned UFP wall

At the bottom interface of the rocking wall, there were seven LVDTs on the east face (Figure 9) and two LVDTs on the west face of the wall to measure the uplift movements. The data were utilized to determine the contact length of the rocking interface and helped to validate the design calculations. In addition, on each steel column, there was one LVDT to measure the relative vertical displacement between two branches of a UFP (Figure 9) and another LVDT to observe the gap opening between the steel column and the rocking wall at the top end of the column (Figure 9). In order to determine the tension forces in high-strength rods, two strain gauges were installed on the two opposite sides of each PT rod in the axial direction (Figure 9).

Test phases

In addition to the two main phases (Phase 1 and Phase 2) in the test program of the building, there were two preceding tests, Phases 0.1 and 0.2, that were performed to characterize the stiffness of the gravity system. Thus, in Phase 0.1, the shear walls were not connected to the diaphragm; the primary actuator at the roof imposed the displacement following the modified CUREE protocol up to the 0.84% drift level, corresponding to 72 mm (2.8 in.) to avoid any possibility in

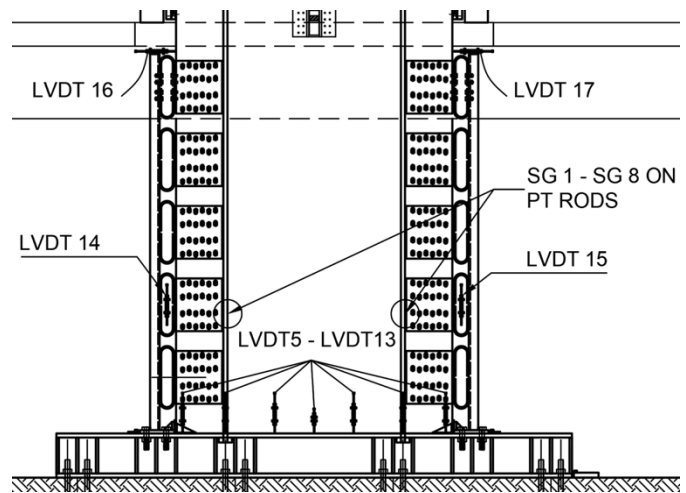


Figure 9. Instrumentation for rocking wall: (a) LVDTs at won East face, LVDT14 and LVDT15 measuring vertical UFP deformation, LVDT16 and LVDT17 measuring gap opening between steel column and wall panel, and strain gauges on high-strength rods.

damaging the gravity system. The purpose of this stage of testing was to determine the initial stiffness of the building gravity system and the contribution of the gravity system to the overall overstrength relative to the tested walls. In Phases 1 and 2, the pivoting BRB wall and post-tensioned UFP wall, respectively, were connected to the diaphragm through the shear key system at each floor level. The full loading protocol (up to 4% roof drift) was applied for the primary actuator in both phases to examine the structural performance of each LFRS,

as well as the building as a complete system. Similarly, Phase 0.2 was conducted after completion of Phases 1 and 2, with the shear walls again left disconnected from the diaphragms. In this stage, the roof actuator pushed the building to a 2% drift to assess whether any degradation had occurred in the gravity system during the main testing phases.

Results and discussion

Gravity system contribution

The gravity system was subjected to displacement cycles reaching up to 0.84% drift of the loading protocol in Phase 0.1. The total base shear was calculated as the summation of forces from the three actuators. Figure 10a shows the base shear versus roof drift ratio hysteresis of the gravity system tested in Phase 0.1, while Figure 10b shows the hysteresis of the gravity system in Phase 0.2. A small positive shift in the base shear at the beginning of the second gravity-only test (approximately 1% of the specimen weight) reflects the need to re-zero the actuator based on the residual position from the preceding phase. Apart from this offset, the hysteresis responses from both tests showed no evidence of cyclic degradation. The peak base shears were 39.1 kN (8.8 kips) at 1% roof drift in the initial test and 46.7 kN (10.5 kips) at 2% roof drift in the final test. As expected for a system consisting solely of the gravity framing, only limited self-centering was observed.

The equivalent hysteretic damping of the gravity system per cycle was estimated after each cycle using the following equation:

$$\xi_{hyst,cycle} = \frac{1}{4\pi} \frac{E_{h,GS}}{E_{S0}} = \frac{1}{2\pi} \frac{E_{h,GS}}{V_{bt}\Delta_t} \quad [1]$$

where $E_{h,GS}$ is energy dissipated by the system in one complete cycle, E_{S0} is strain energy of the linear-elastic system at the target displacement, Δ_t , and base shear, V_{bt} , determined at the target displacement. The strain energy E_{S0} is the sum of the strain energy in positive and negative directions. Based on results in Phase 0.1, equivalent hysteretic damping of the gravity system per cycle increases from 0.049 to 0.075 with the increase of maximum roof drift ratio from 0.17% to 0.83%.

MPP pivoting wall

Performance

Figure 11a shows the total base shear-roof drift hysteretic behavior during Phase 1. The specimen maintained a nearly uniform distribution of story drift throughout the loading protocol, as intended. The building exhibited full and stable global hysteretic loops, including a complete cycle at the 4.0% enhanced-performance drift target and two subsequent cycles at 3.0%, without loss of lateral strength. The global response resembled other pivoting lateral systems supplemented primarily with BRBs, such as steel strongback frames (Simpson and Mahin 2018).

The BRBs governed the inelastic global behavior, while the MPP wall, diaphragms, and gravity system remained essentially elastic. BRB yielding occurred nearly simultaneously in tension

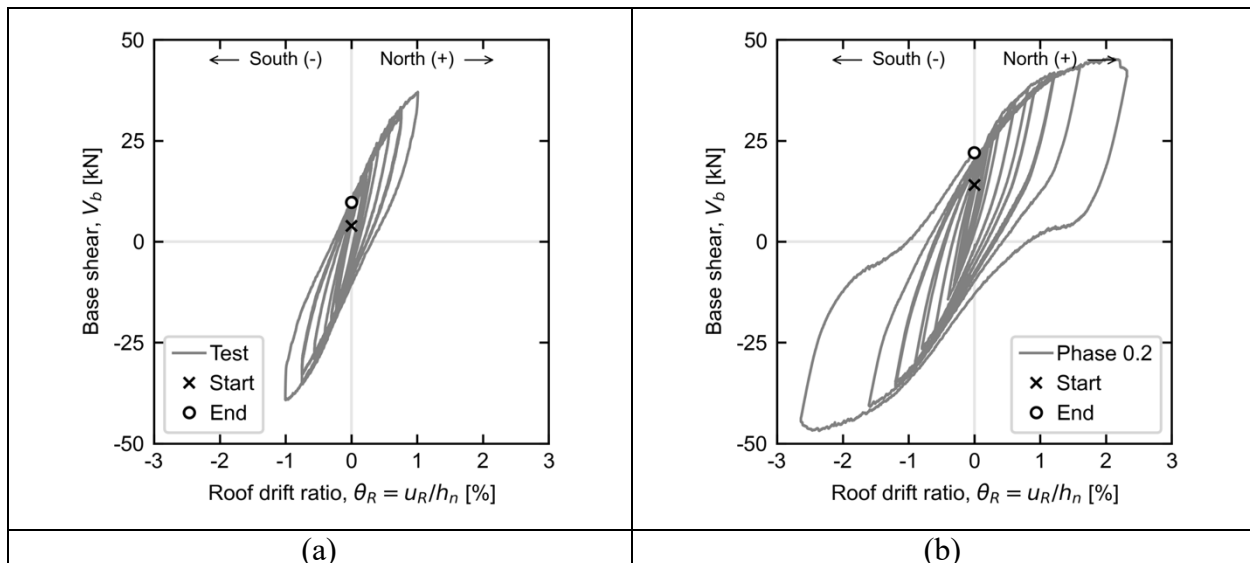


Figure 10. Cyclic behavior observed during: (a) Phase 0.1 and (b) Phase 0.2.

and compression at approximately 0.44% roof drift. As the BRBs yielded and strain-hardened, lateral strength increased, reaching a peak base shear $V_b \approx 222$ kN, approximately twice the design base shear. At 4.0% roof drift, the system achieved a ductility demand of 9.09 relative to the drift at BRB yielding. The force-deformation loops maintained a stable, nearly symmetric shape throughout testing. Minor asymmetries between push and pull directions, particularly in trailing cycles, resulted from test setup and actuator load-transfer details, not from structural system behavior.

Figure 11b shows the estimated cyclic axial force-deformation behavior of the BRBs. After yielding, the BRBs exhibited the expected asymmetric kinematic and isotropic strain-hardening behavior, with similar axial deformations in tension and compression. At the 4.0% target drift, BRB deformations reached approximately 40 mm (1.57 in.), resulting in strains of near 2% in the yielding core, well within design criteria. The BRBs met all AISC 341 (AISC 2022) acceptance criteria for cyclic testing: (1) stable, repeatable behavior with positive incremental stiffness; (2) no rupture, instability, or connection failure; (3) maximum compression-to-tension force ratio below 1.50; and (4) cumulative inelastic axial deformation capacity (CPD) exceeding 200 times the yield deformation (measured CPD = 331). Low-cycle fatigue analyses indicated significant remaining BRB fatigue life at test completion (Araújo et al. 2025).

Equivalent hysteretic viscous damping was estimated from dissipated energy per cycle (Jacobsen 1960). In initial cycles, the

hysteretic damping ratio remained relatively constant, around 0.06–0.07, associated with gravity system nonlinearity. After BRB yielding, damping increased to 0.32 at 2% roof drift and 0.41 at 4% drift, consistent with previous tests on systems employing BRBs as energy dissipators (Kim and Choi 2004; Sutcu et al. 2014; Zhu and Zhang 2008).

Residual drifts increased significantly after BRB yielding due to accumulated residual deformations, reaching 1.38% after unloading from 2.0% roof drift. While such residual drifts typically indicate significant repair costs, post-earthquake residual drifts are controlled by both cyclic behavior and ground motion characteristics and are generally less than quasi-static values measured in testing. For example, analytical estimates in Araújo et al. (2025), using an equivalent bilinear SDOF transformation (Kawashima et al. 1998; Macrae and Kawashimas 1997), suggested dynamic residual roof drift after the DE hazard level could be approximately $0.5\% \pm 0.3\%$.

Damage observations

Damage was concentrated in the BRBs in the form of permanent deformation, while all other components performed essentially elastically. Figure 12a shows initial state, while Figure 12b shows permanent deformation, and residual tilting near the gusset connection following testing. Permanent deformations in the yielding core appeared as the encased core protruding from the steel casing. Minor bending and superficial flaking developed at BRB end regions adjacent to bolted gusset plates, but did not affect hysteretic behavior. No brace-end fracture,

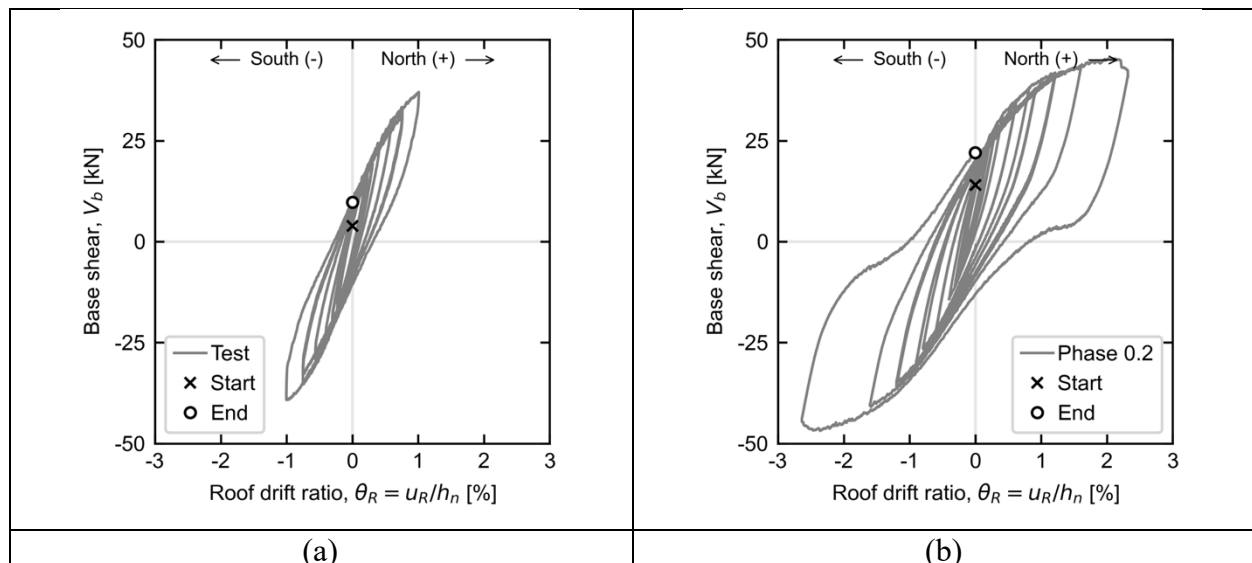


Figure 11. Cyclic behavior observed during Phase 1: (a) global base shear-roof drift ratio; (b) local BRB force-axial deformation.

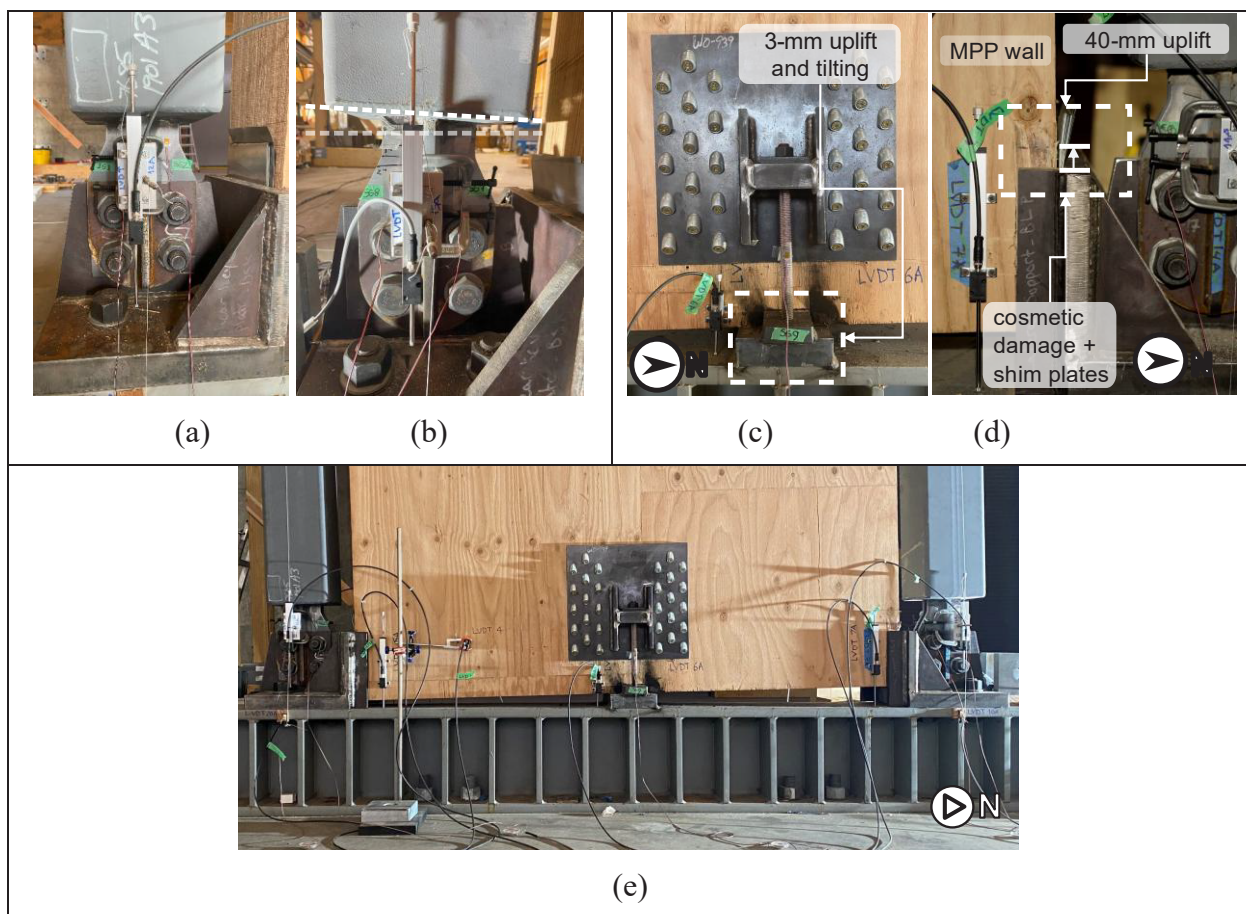


Figure 12. Damage observed in Phase 1: (a) permanent tilting of the wall at the end of the test; (b) uplift and tilting in pivoting support; (c) bearing marks in the wall toe; (d) bearing marks developed at base shear key interface; (e) final state of BRB walls at test completion depicting the residual drift.

buckling, or instability occurred throughout the protocol. Due to inelastic BRB deformations, the pivoting wall-BRB system remained tilted northward, with a residual roof drift of 2.25% at test completion; see Figure 12e.

The pivoting wall behaved as intended, with the MPP panel remaining essentially undamaged above the pivoting base. The pivot support enforced compatible, nearly symmetrical vertical deformation in both BRBs and limited wall uplift to a few millimeters; see Figure 12c. The wall base gap prevented concentrated compression zones at the wall toe, eliminating the crushing that often governs rocking-wall system performance. Only small, cosmetic bearing marks developed at base shear key interfaces, as shown in Figure 12d. Bearing damage in the MPP minor-strength direction was nearly negligible due to the sacrificial shim plates between the wall and shear key that distributed bearing stresses and accommodated rotation compatibility.

Post-tensioned MPP rocking wall

Performance

Figure 13a shows base shear versus roof drift ratio hysteresis for this test phase. The structure reached a base shear of 533.88 kN (120.02 kip) at 4.05% roof drift level and 520.98 kN (117.12 kip) at -3.59% drift level. Comparing with performance requirements, the building met all design requirement at the three design levels. All PT rods remained in the linear elastic range at performance levels 1 and 2 and exceeded the elastic limit strain at performance level 3.

Figure 13b shows the average PT force in the pairs of rods at north and south sides of the rocking wall. The average initial PT forces in the north and south pairs were 157.8 kN and 155.5 kN, respectively. When the building moved towards the north direction, the average PT force in the south pair increased up to 624.3 kN, primarily due to the rocking of the wall, which

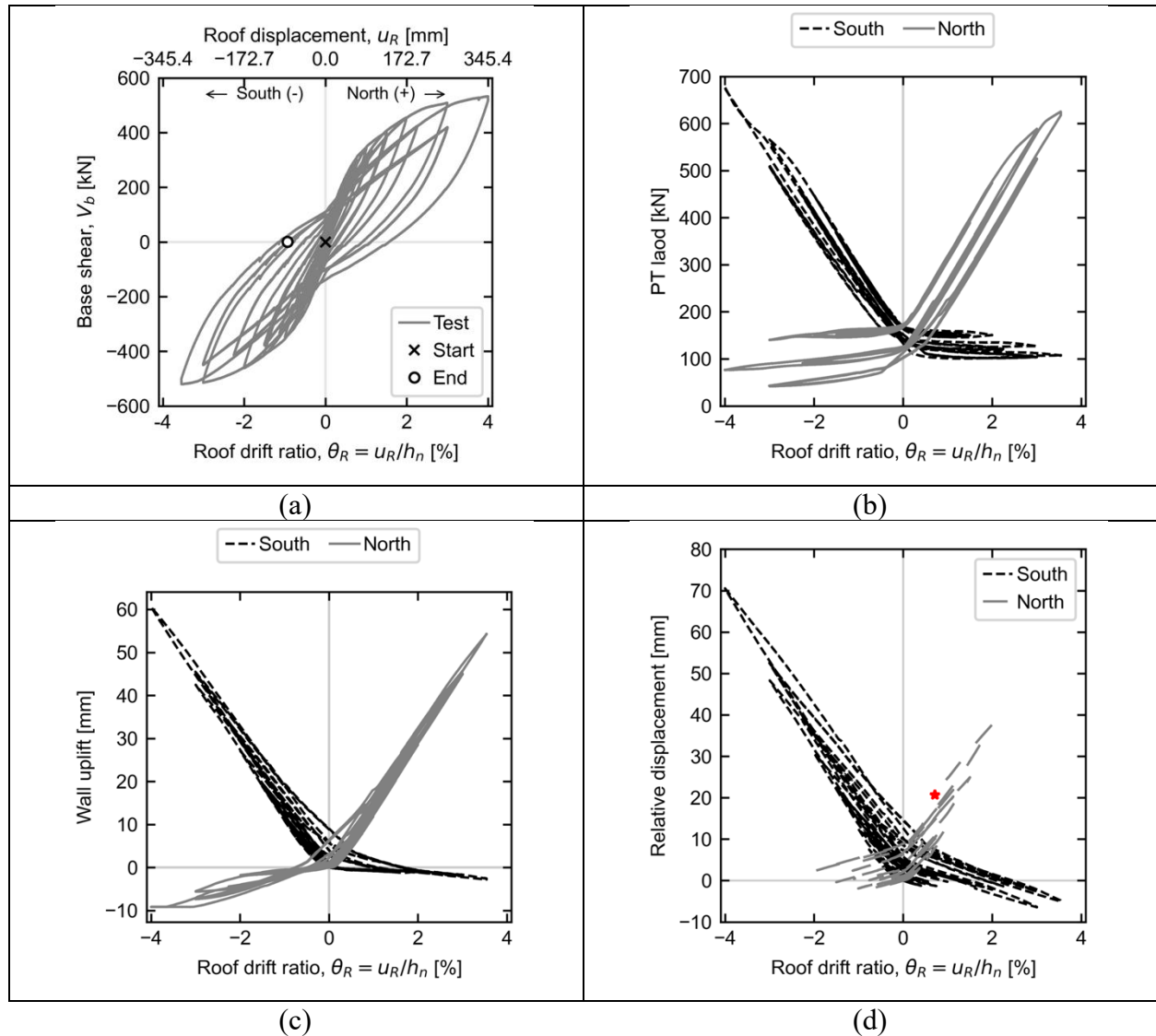


Figure 13. Cyclic behavior observed during Phase 2: (a) global base shear-roof drift ratio; (b) average PT rod strain deformation; (c) uplift measurements at bottom wall corners; and (d) UFP vertical relative displacement between the wall panel and boundary steel section.

was clearly visible as the wall uplift at the south corner wall base (see Figure 13c). The average PT forces on the north rods decreased 17.9% (to 129.6 kN) when the building reached the drift level of 2%. This reduction of forces was expected, as the panel rocking induced some compression in the wall, which led to shortening of the north pair of rods. After reaching 3% and 4% drift levels, the tensile forces in both north and south PT forces considerably decreased because of the yield in PT rods and crushing of the MPP panel at the ends of the wall panel that were in contact with the foundation as the wall rocked. At the end of the test, the average PT forces in the north rod pair dropped to 39.4 kN (a reduction of 25% from the initial value). At 4.05% drift, the south end reached an uplift of 61 mm (2.40 in.), and north end compressed up to 9 mm (0.35 in.).

Figure 13d presents the relative vertical displacements recorded from the UFP assemblies. The measurements indicate that UFPs on the south edge of the wall registered negative relative displacements, suggesting greater toe damage on the north side of the wall, which aligns with the negative displacements measured at the base on north side of the wall (Figure 13c). The red star in Figure 13d marks the last reliable data point before the sensor detached from its support.

The estimation of hysteretic damping was conducted using Equation (1), similarly as in Phase 0.1, and showed that the equivalent hysteretic damping per cycle varied from 0.011 to 0.081. The equivalent hysteretic damping ratios of new peak cycles of 1%, 2%, 3%, and 4% were 0.036, 0.054, 0.073, and 0.081, respectively.

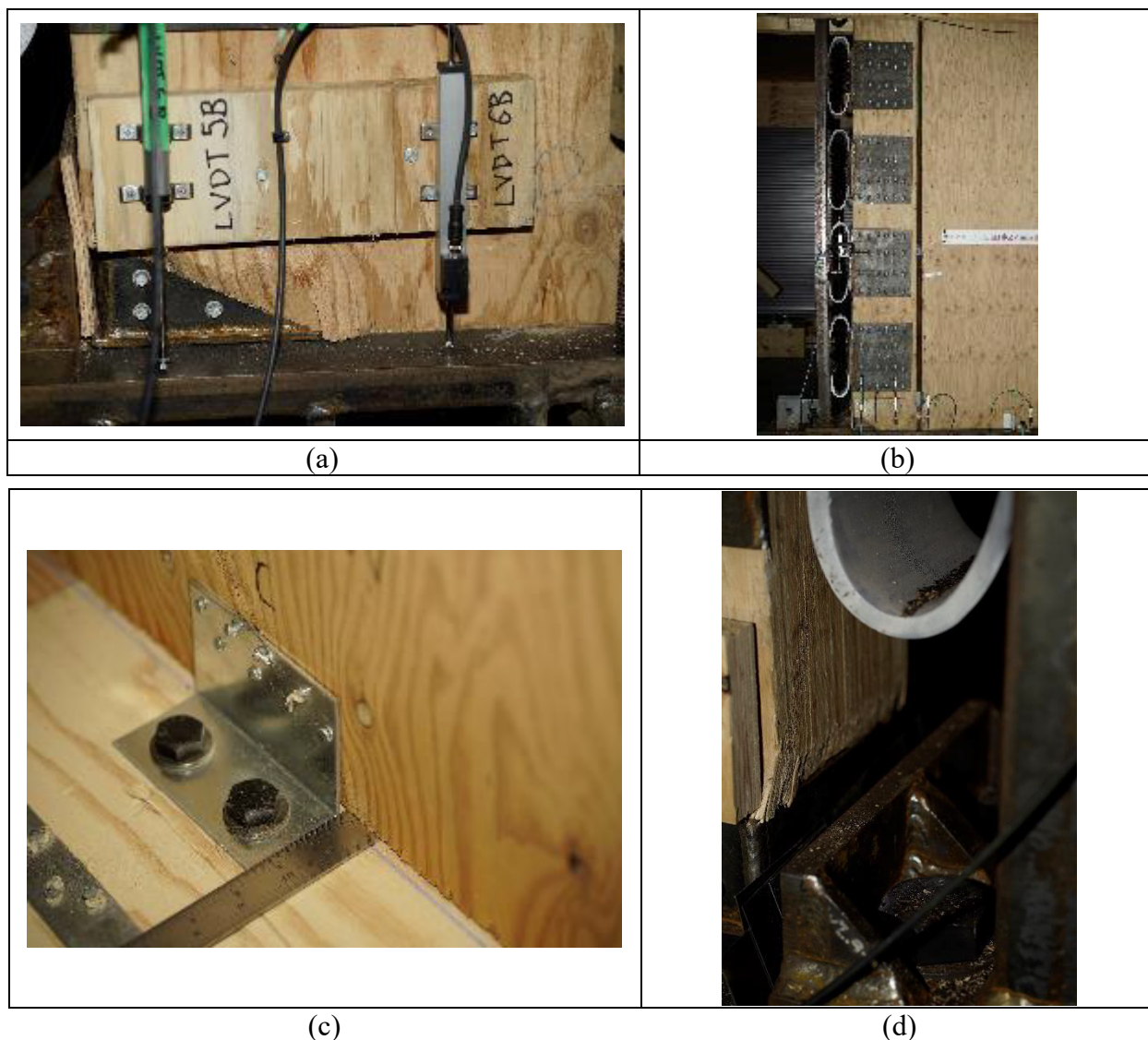


Figure 14. Damage observed in Phase 2: (a) Crushing at bottom corners of rocking panel; (b) deformation of steel side columns and UFPs; (c) nail damages and wall sliding at a bottom bracket connection between out of plane wall and diaphragm panels; and (d) mill scale at the bottom of the UFP.

Overall, the residual drift was less than or equal to 0.1%, 0.6%, and 1.7% at performance levels 1, 2, and 3, respectively. The results imply that the building experienced low-damage after the test, with permanent drift considerably lower.

Damage observation

There was no damage observed on the MPP rocking panel, especially at the area of rocking interface, at the 2% drift ratio level, which indicates that the panel was still in elastic range, as designed. After the 3% drift ratio level, crushing at the bottom corners of MPP panel (Figure 14a) was observed within a 200 mm (8 in.) wide and 80 mm (3 in.) high area from the vertical edge of the panel. Out of plane buckling of the exterior MPP layers along the edge of the steel toe was observed at the 3% drift ratio level. In addition, due to compression of the sacrificial

plywood shims placed between the wall and the steel shoes, sliding of the wall started occurring, which induced the side steel columns to bend and the gap between side columns and vertical edges of MPP panel to increase, especially around the mid-span of the columns (Figure 14b). This mode of residual deformation kept increasing due to residual deformations of the steel column, the UFPs, or both.

Out of plane walls showed minor damage, including a few pull-out nails and broken nail heads at bracket connections, and wall sliding in the loading direction (Figure 14c) after the peak drift level of 4.05%. A maximum sliding of 7 mm (1/4 in.) was observed among out of plane walls after the test. Figure 14d shows steel mill scale aggregation deposited at the bottom of the UFPs as a sign of yielding in the UFPs.

The MPP diaphragms remained essentially elastic throughout the entire test. No visible signs of slip in the plywood splines or openings between the diaphragm panels were observed. Similarly, the beam-to-column connections in the gravity system were able to accommodate story drift demands up to 4% through rocking without inducing damage to the LVL columns. At such a drift level, sliding of the slotted plates in the gravity connections, relative to the LVL columns, was noticeable due to rocking.

Comparison between two systems

A direct comparison between the two LFRS highlights distinct seismic performance characteristics associated with each design philosophy. The BRB-MPP pivoting wall system exhibited stable and fuller hysteretic response with substantial energy dissipation capacity resulting from the yielding of the BRB. The system reached a peak base shear of approximately 222 kN, while maintaining stable cyclic behavior up to 4% roof drift without significant degradation in lateral resistance. Equivalent viscous damping increased substantially after BRB yielding, reaching values approaching 0.41 at large drift levels, indicating efficient hysteretic energy dissipation. However, the enhanced energy dissipation was accompanied by increased residual drift, due to permanent BRB deformations, with residual roof drift remaining at the completion of testing. In contrast, the post-tensioned rocking wall system developed significantly higher peak lateral resistance, exceeding 530 kN at approximately 4% roof drift, while exhibiting a flag-shaped hysteretic response characteristic of self-centering systems. Although the equivalent viscous damping ratios remained lower, reaching approximately 0.08 at the largest drift levels, the system demonstrated substantially improved recentering behavior and reduced residual drift demands, compared with the BRB-MPP system.

The observed damage mechanisms further demonstrate the fundamental behavioral differences between the two systems. In the BRB-MPP pivoting wall configuration, damage was intentionally concentrated within the BRBs through stable yielding and strain hardening, while the MPP wall, diaphragms, and gravity framing remained essentially elastic throughout the test program. The pivoting mechanism successfully prevented significant toe crushing and localized damage in the timber panel, thereby supporting the intended low-damage design philosophy. Conversely, the post-tensioned rocking wall system concentrated inelastic demands at the rocking interface and within the UFP energy dissipators. While the system maintained near-elastic behavior up to moderate drift levels

and achieved an effective self-centering response, localized crushing at the wall toes, deformation of the boundary steel columns, and yielding of the UFP assemblies were observed at larger drift demands. Collectively, these results illustrate the inherent tradeoff between enhanced energy dissipation and residual deformation control. The BRB-based system provided superior hysteretic damping and energy absorption, whereas the post-tensioned rocking wall demonstrated improved recentering capability and reduced permanent deformation, both representing viable low-damage seismic design strategies for mass timber structures.

Conclusions

A three-story mass timber building was designed following a performance-based design methodology. The structure consists of MPP floor slabs, LVL beams and columns for the gravity framing, and two different LFRS solutions, including a MPP pivoting wall with BRB and post-tensioned MPP rocking walls with UFP energy dissipators.

1. The gravity framing, although not intended to participate in the LFRS, exhibited measurable lateral resistance and energy dissipation during testing. This unintended engagement contributed to system overstrength and enhanced damping at low drift ratios, highlighting the influence that gravity systems can have on the seismic response of self-centering mass-timber structures.
2. The hybrid spine combining a MPP wall with steel buckling-restrained braces (BRBs) provided effective lateral resistance: the BRBs yielded under cyclic loading, while the MPP wall remained largely elastic, allowing stable, repeatable hysteretic response with no evidence of cyclic degradation.
3. The capacity-designed connections and foundation shear-transfer assemblies used in the hybrid BRB-MPP spine demonstrated excellent robustness, maintaining structural integrity and stable behavior under drift demands that exceeded code-level earthquake intensities. This confirms that the system can reliably deliver enhanced seismic performance, while protecting the mass-timber components from inelastic damage.
4. The test results confirmed that the DDBD-based design approach for the mass-timber self-centering rocking wall provided reliable estimates of system response across the expected inelastic range. The methodology enabled controlled concentration of inelastic demands at the wall toes

and in the UFPs, while protecting the post-tensioning tendons and primary mass-timber components from significant damage. The system maintained near-zero residual drift up to moderate drift levels, and the measured equivalent viscous damping closely aligned with design predictions, demonstrating that simplified flag-shaped models can effectively represent the seismic behavior of self-centering systems with engaged gravity framing.

5. Cyclic testing of the three-story SCRW specimen showed that the wall achieved the intended rocking mechanism with stable performance up to 4% roof drift ratio. Inelastic demands remained localized, the PT rods stayed below yield-level strain, and the structure preserved its integrity despite minor residual drift and localized toe damage at large deformation levels. These results demonstrate that post-tensioned mass-timber rocking walls provide a robust, damage-controlled alternative for resilient seismic design.

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