An Improved Design Approach for Mass Timber Shear Walls

Using CLT Panels and Shear Fuse Connections

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Cross-laminated timber (CLT) panels are widely employed for floor structures, but they are far less likely to be incorporated as shear walls to resist lateral loads. Current code design provisions established in the "Special Provisions for Wind and Seismic, 2021" (SDPWS) are primarily intended to ensure adequate strength of platform-framed wall assemblies, which rely on steel plates with screwed connections to transfer shear and overturning forces. This type of connection results in wall assemblies prone to localized failures with a distinct lack of ductility. As such, the seismic force reduction factor (R) for CLT walls in ASCE 7-22 is limited to a relatively low value of four (4), which reflects the limited ductility in the wall panel connections. This tends to lead practicing engineers toward designs that are inefficient and may exhibit poor seismic performance, making them uneconomical and impractical for typical applications. The current code-basis for seismic-force-resisting systems (SFRSs) using CLT walls does not consider rocking behavior with a reliable energy dissipation mechanism.

This work focuses on an alternative approach using balloon-framed CLT shear walls with rocking behavior and yielding fuse plates (YFPs) with trapezoidal cutouts (also known as "butterfly" or Krawinkler fuses). This type of assembly promises more reliable seismic behavior with better utilization of the inherent shear strength of the CLT panels, which could make it a more economical and effective seismic-force-resisting system than what is currently considered.

The proposed shear wall assembly comprises at least two CLT panels oriented vertically and connected with steel YFPs. Shear deformation is intended to occur vertically at the interfaces between panels, dissipating input earthquake energy in a stable, ductile manner. The YFPs are designed as fuses that limit the shear force resisted by the wall assembly to ensure that non ductile mechanisms are avoided. At wall boundaries, the CLT panels walls are connected to the foundations with pinned hold-downs that are capacity-designed to remain elastic while resisting overturning forces. This configuration allows the connected CLT panels to resist lateral loading through stable rocking motion with robust energy dissipation.

The design was recently proposed for a project in San Francisco, CA. Since the proposed approach did not follow the SDPWS provisions, a peer-reviewed performance-based approach was adopted to demonstrate conformance with the building code and obtain the building permit. Validation of the rocking CLT shear wall system was initially established using nonlinear static pushover analysis assuming an R-value of four (4). Based on the observed performance, it was evident that the system could be designed for a higher R-factor. Following the peer-review process, the study was further extended to verify performance with nonlinear response history analysis (NRHA) and demonstrate that designing to an R-value of eight (8) would provide acceptable performance.

This paper presents analysis results comparing a building design based on the presumed R-value of four (4) and an alternative design based on an R-value of eight (8). Both designs utilize balloon-framed CLT walls with YFPs. The analyses demonstrate that the system designed for the higher R-value is more than adequate at dissipating seismic energy and controlling the overall response of the structure to meet the intent of the building code. Designing for the lower R-value offers markedly enhanced performance relative to the minimal code requirement.

Our goal is to further the development of this non-proprietary lateral force-resisting system. The proposed rocking CLT wall assembly fully utilizes the strength of the mass-timber panels and provides reliable ductility and inelastic response. This approach provides significant gains in efficiency and performance over conventional platform-framed CLT shear walls with steel plate connections, making it a more viable and economical alternative.

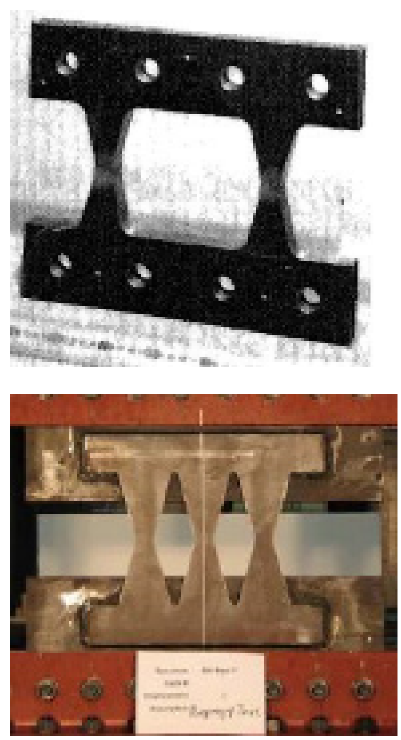
Introduction

Mass timber construction is growing in the United States, fueling design innovation and code changes. Mass timber components generally work well under compression loads but provide limited tensile capacity, similar to concrete components. This lack of inherent ductility poses a challenge in utilizing mass-timber components directly in a seismic-force-resisting system (SFRS). ASCE 7-22 adopts CLT shear walls with a high aspect ratio as a standard seismic force-resisting system. However, limited nonlinear mechanisms in the connections are reflected in the low value of the response reduction factor (R = 4). This has constrained the potential benefits of using mass-timber wall panels as the primary SFRS. Often, mass timber structures rely upon code-conforming steel or concrete SFRSs to take advantage of a higher R-value.

The research and design community has focused on rocking timber wall systems as an alternative approach. This type of system was originally modeled after the precast concrete rocking wall system, which was extensively researched and tested in the 1990s. The first version of the rocking timber wall system was tested using laminated veneer lumber (LVL) panels (Palermo et al. 2012). Post-tensioned rocking CLT wall systems have gained popularity among researchers recently due to their re-centering capabilities (Ganey et al. 2017, Akbas et al. 2017, Pei et al. 2019, Blomgren et al 2019). Rocking CLT walls can also be equipped with external energy dissipators to increase efficiency and reduce floor accelerations. Recent large-scale system-level tests have demonstrated the resilient re-centering behavior of such systems (Ceccotti et al. 2013).

Kobori et al. (1992) tested a novel "Honeycomb damping system" which comprised a steel plate with openings resembling the shape of a honeycomb. This design created butterfly-shaped steel links connecting the two rigid ends (Fig. 1a) that would yield in shear to dissipate seismic energy. The full-scale tests showed stable hysteretic behavior and high ductility in such dampers. This damper was modified by Ma et al. (2010) by reducing the opening size, thus bringing the yielding links closer, and was called a "butterfly" plate (Fig. 1b).

(a)



(b)

Figure 1 (a) Honeycomb damper plate (Kobori et al., 1992) and (b) butterfly fuse plate (Ma et al., 2010).

Building Description

For this study, we have used an archetype structure representing a proposed 4-story academic building in San Francisco, California. The building plan area is 68 ft x 86.5 ft with a structural height of 55 ft and occupiable landscaped spaces on the roof. The gravity structural system for the building comprises concrete over metal deck floors supported on 24-inch deep glue-laminated timber (glulam) beams and 30-inch deep glulam girders. The girders transfer gravity loads to 15-inch square glulam columns supported on isolated concrete footings. The SFRS of the structure comprises 12.42-inch thick cross-laminated timber (CLT) wall panels. The total weight of the building is 2,230 kips. The site is designated as Site Class D, with SS = 1.5g and S1 = 0.6g.

Proposed Rocking CLT Wall Assembly

The wall assembly comprises rocking CLT wall panels connected along the vertical joints with YFPs. Each of the wall boundaries consists of a pair of glu-lam posts with a continuous steel-plate hold-down sandwiched between them (Fig. 2a). The bolted connections between the CLT panels and the boundary posts are capacity-designed with adequate strength to remain elastic under earthquake response. The hold-downs are anchored at the base with pinned connection to the foundation (Fig. 2c) to enable the connected CLT panels to pivot under lateral loading. A gap under the end-panels permits downward movement of the panel edge opposite of the pin connection, allowing for the rocking action. Inertial forces from the floor diaphragms are transferred to the CLT shear walls with simple steel plates and bolts (Fig. 2b).

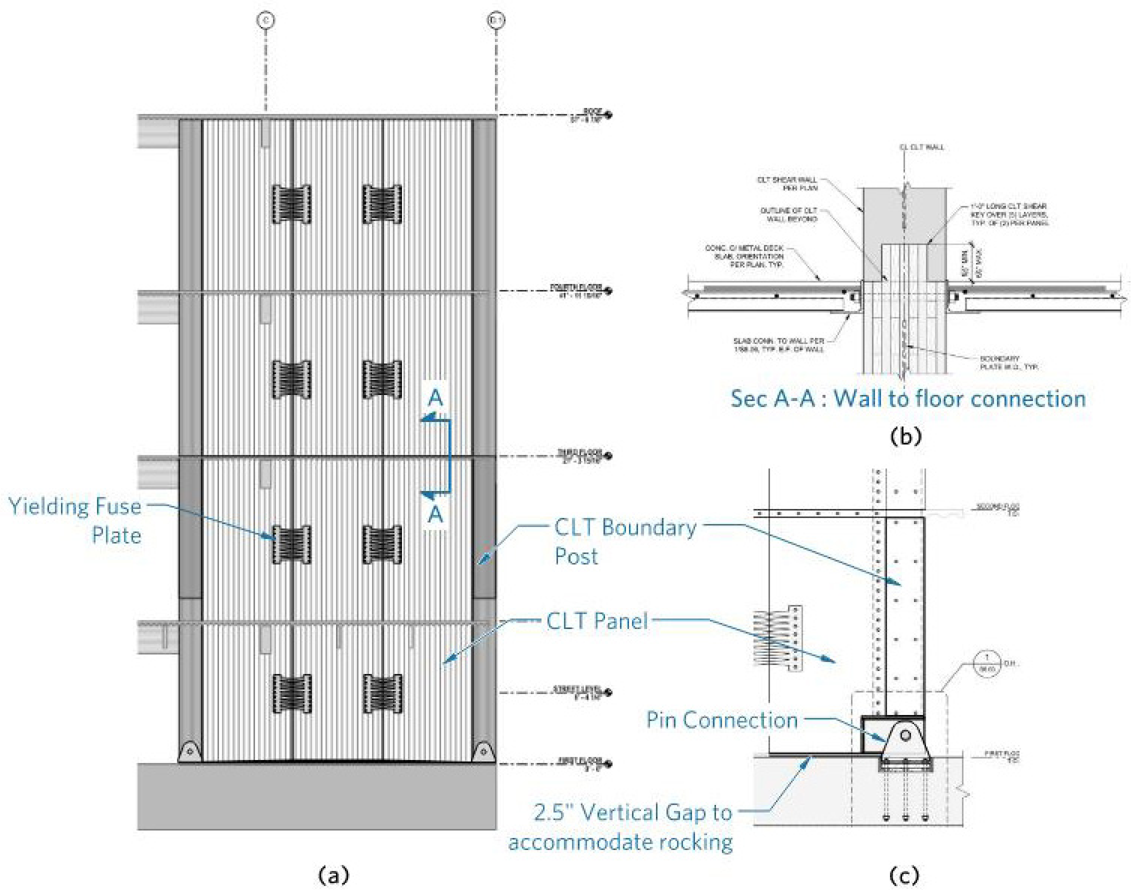


Figure 2 Proposed rocking CLT wall assembly: (a) elevation of 3-panel wall, (b) section through connection between floor diaphragm and CLT wall, (c) detail of boundary with pinned hold-down

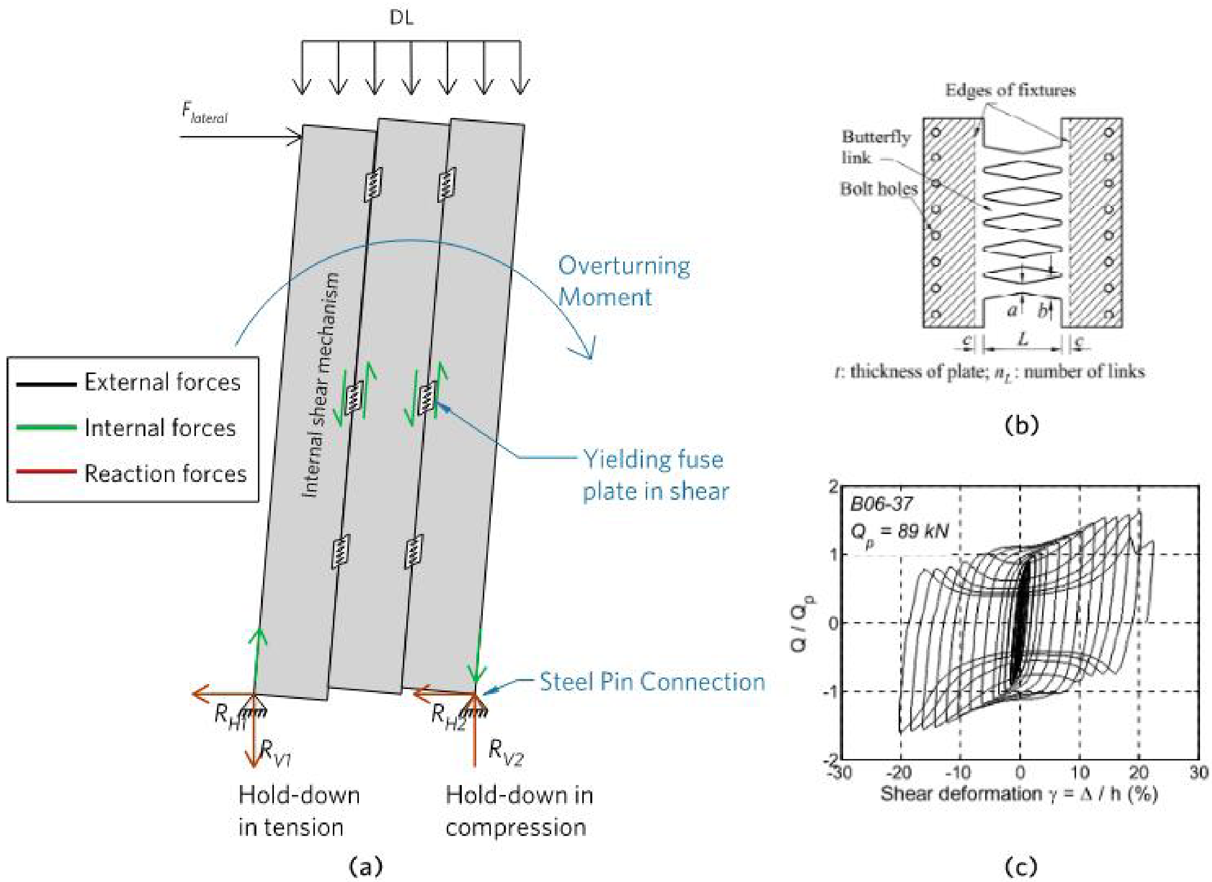


Figure 3 Mechanism of the proposed rocking CLT wall: (a) assembly with imposed lateral deformation; (b) geometry of a "butterfly" plate; (c) load-deformation relationship of a "butterfly" plate (Ma et al., 2010).

The YFPs experience shear deformation due to the relative vertical movement of the CLT panels (Fig. 3a) and dissipate seismic energy through flexural yielding of the individual links within the YFPs. A YFP, shown in Fig. 3(b), is conceptually similar to a set of steel beams in parallel with fixed ends. The two fixed ends of the fuse are attached to the adjacent rocking CLT wall panels. The strength and stiffness of the shear fuses are a function of steel yield stress, the number of links, the width of the fuse link end section, the thickness of the fuse plate, and the length of the fuse links. A series of experiments(Ma et al. 2010) conducted to quantify the behavior of the YFPs showed stable inelastic behavior and ductility (Fig. 3c). The published research recommends design equations to size the YFPs based on the results of full-scale experiments. The design team utilized these equations to design the fuses and kept the design parameters within the limits outlined in the research.

Limited Ductility Seismic Design (R=4)

CLT shear wall systems first appeared in the list of pre-qualified standard SFRSs in Table 12.2-1 of ASCE 7-22. There are two entries in the table, and the design team selected "Cross-laminated shear walls with shear resistance provided by high-aspect ratio panels only" as the most appropriate system, with recommended values of the Response Modification Coefficient, R, of four (4), Displacement Amplification Factor, C_d, of four (4), and Overstrength Factor, Ω_o, of three (3). The low R-value is based on the limited ductility achieved by a CLT shear wall system with screwed or nailed connections.

Inelastic behavior in a screwed or nailed connection is typically characterized by localized failure of the fasteners, with limited energy dissipation and significant post-yield strength loss.

The seismic base shear was calculated using the Equivalent Lateral Force Procedure (ELF) in Chapter 12 of ASCE 7-22. The relatively low R-value resulted in high lateral force demands, and the number and length of CLT shear walls were determined such that the system would provide sufficient lateral strength. In contrast with light-frame wood shear wall structures where partition walls may conveniently be used as shear walls with the application of structural sheathing, the quantity of relatively expensive CLT walls correlates directly with the design shear demand. Thus, a conservative R-value tends to yield an overly expensive design.

The limited ductility design approach resulted in four (4) CLT walls with rocking panels in the X-direction (two assemblies with two and three panels, respectively) and three (3) CLT shear walls in the Y-direction (one assembly with three panels and two with two panels) (Fig. 4a). High-capacity YFPs were required to transfer the large design shear between the CLT panels. The high axial demands in the wall assembly hold-downs resulted in large concrete footings.

Optimized Seismic Design (R=8)

The yielding mechanism of the rocking CLT wall system (Fig. 3b) is similar to a steel eccentric-braced-frame system, which is adopted in ASCE 7-22 and assigned an R-value of eight (8) in Table 12.2-1. Since seismic demand based on ELF is inversely proportional to the R-value, doubling the seismic-force reduction factor cuts design seismic demands in half, reducing the quantity of CLT shear wall required and leading to a much more economical structural design.

An elastic design of the system was carried out using the ELF procedure and R = 8. Taking advantage of the reduction in seismic base shear, the number of wall assemblies was reduced significantly to two rocking CLT wall assemblies in both the X- and Y-directions, each with three panels (Fig. 4b). A nonlinear pushover analysis was utilized to adjust the capacity and layout of the YFPs to reduce torsion. Finally, nonlinear response history analyses were performed for both designs using a suite of 22 bi-directional ground motions selected to represent the site in terms of soil conditions, fault type, and distance to the nearest source. The following sections present a comparison of the seismic performance of the two designs.

Figure 4 Designed CLT shear wall locations: (a) Limited ductility design (R=4); (b) Optimized design (R=8)

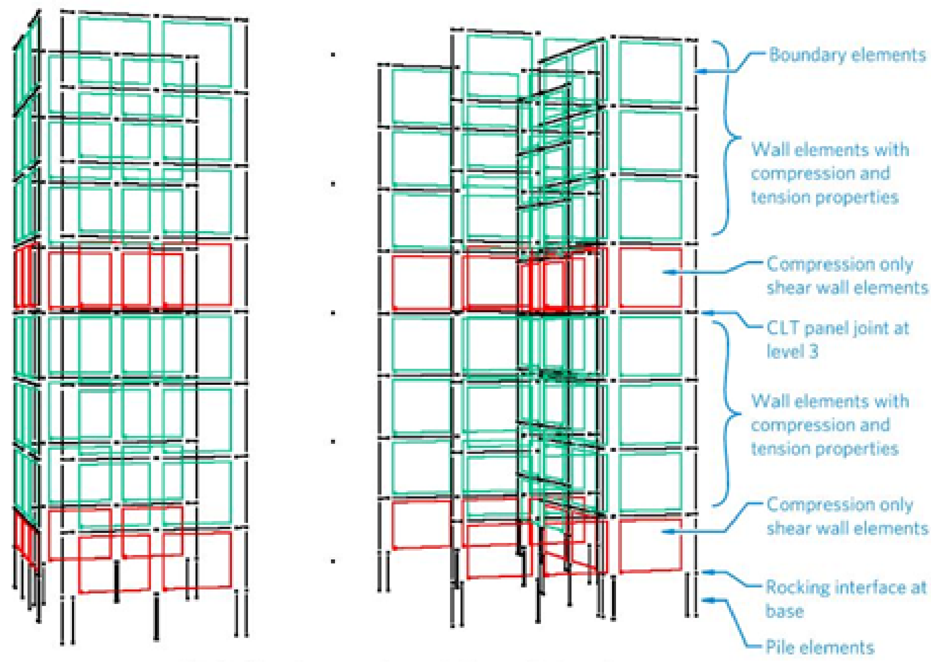


Figure 5 Nonlinear analysis model.

Description of Nonlinear Model

Nonlinear analysis models for both designs were prepared using the software platform Perform 3D by Computers and Structures Inc. The models (Fig. 5) include all elements of the CLT rocking wall system expected to contribute significantly to seismic behavior: CLT panels, YFPs, boundary hold-downs, CLT panel-to-boundary connections, and soil springs. The seismic mass of each floor was lumped at center-of-mass nodes. Floor nodes were assigned rigid diaphragm constraints.

All actions of the CLT wall panels were considered force-controlled. The panels were modeled with four-node shell elements with a nonlinear shear material, nonlinear fiber section for axial, and elastic properties for out-of-plane bending. Stacks of wall elements representing individual CLT panels were located adjacent to one another with nodes separated slightly to make room for the YFPs to connect the panels and form the wall assembly (Fig. 6). Care was taken between panels in each stack and at the base to allow for rocking; the wall shell elements were assigned compression-only behavior (shown in red in (Fig. 5) at the bottom of the CLT walls and immediately above Level 3.

Shear wall boundary elements were characterized as deformation-controlled components. These were modeled with two-node tension-only tie elements with nonlinear properties representing the steel plate sandwiched between glulam column sections. The connection between the wall panels and the boundary posts comprises shear bolts and was modeled with 12-inch long frame elements with nonlinear shear hinges. Soil springs were modeled using two-node concrete strut elements (compression only) with a backbone representing the subgrade conditions at the site, with proportional tributary areas assigned based on location.

YPFs provide the primary energy-dissipating mechanism in this structural system. These were modeled with two-node frame elements with a nonlinear shear hinge in the vertical direction.

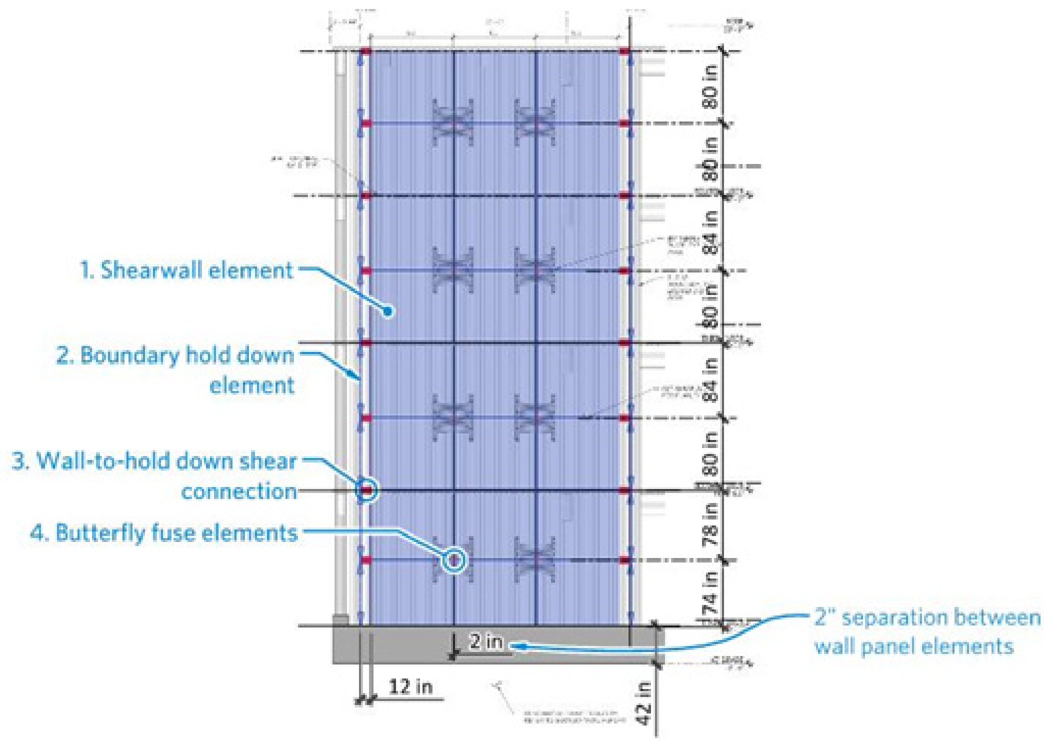


Figure 6 Idealized wall assembly in the nonlinear analysis model.

Nonlinear Pushover Analysis

A load pattern representing the vertical distribution of the seismic forces according to ASCE 7-22 chapter 12 was assigned at the center-of-mass nodes in the two principal horizontal directions. After a downward gravity load was applied to the structure, the lateral loads in the H1 (X) and H2 (Y) directions were applied independently to perform a nonlinear pushover analysis. The elastic drift demand (δe) was calculated as the point where the pushover curve crosses the value of the design base shear calculated using the ELF procedure. The inelastic drift (δx) was calculated by multiplying the elastic drift by the Deflection Amplification Factor (Cd).

The design base shear using the ELF procedure for the optimized design approach (R=8) was 290 kips as compared to 581 kips for the limited ductility design (R=4). This translates to a factored load demand in the range of 6 kips to 9 kips per linear foot of shear wall.

The nonlinear pushover curves (Fig. 7) show that both designs remain elastic at the design base shear level, and the (R=8) system shows significant excess capacity. Also, the estimated inelastic drift demand under a design basis earthquake remains below the onset of strength loss of the system and the Code drift limit in both cases.

Nonlinear Response History Analysis

The case study building was evaluated using nonlinear response history analysis (NRHA). A suite of 22 pairs of bi-directional historical ground motion records (44 records total) considered for the ATC 63 project (FEMA, 2009) and scaled to the MCE hazard level for the site was used for NRHA. The scaling of the ground motions was based on the median spectral acceleration of the maximum component of the 22 records. The period range for ground motion scaling was taken between 0.2T1 and 2.0T1, where T1 is the period of the fundamental mode of vibration for the building. A combination of 2.4% constant modal damping and 0.1% Rayleigh damping was used for the dynamic analysis. The seismic performance of the two designs was compared in terms of mean interstory drift ratio, YFP shear strain, and axial strain in the hold-downs.

The seismic performance of the limited ductility (R=4) design is shown in Fig 8. The transient story drift limit for NRHA of structures under MCE level hazard is 4% per equation 16.4-1 of ASCE 7-22. The mean peak interstory drift in both horizontal principal directions, recorded at three corners of the floors of the building, stayed under 1.5%, well below the prescribed limit. The mean peak shear strain in the YFPs (Fig. 8) exceeded the yield strain (0.016) and stayed below 0.037, which is far below the peak shear strain limit of 0.18. Many shear strain values were below the yield strain limit, suggesting that the YFPs are underutilized. The large number of walls associated with the R=4 design resulted in a strong lateral system that prevented the YFPs from experiencing more shear deformation. As intended, the axial strain in the hold downs at the wall boundary did not exceed the yield strain value for steel.

Analysis results for the model with the optimized design (R=8) are shown in Fig. 9. Since there are only two 3-panel rocking CLT wall assemblies in each principal direction, the structure is softer and experiences a higher drift demand than the limited ductility structure. However, the largest mean transient interstory drift was 1.9%, well below the drift limit of 4% per ASCE 7-22. In the preliminary seismic design with R=8, all wall assemblies had YFPs with eight (8) yielding links. However, two YFPs in one of the walls in the X-direction were modified to have ten (10) yielding links to reduce torsion in the structure. This higher stiffness in the X-direction is also reflected in the difference in the peak interstory demand in X- and Y-directions (Fig. 9).

The largest mean shear strain in the YFPs was 0.055, approximately 50% larger than the strain demands for the limited ductility design. The highest strain for any YFP under an individual ground motion was 0.14. The YFPs for the optimized design are more fully utilized compared to the limited ductility design, and far fewer YFPs remain elastic under individual ground motions.

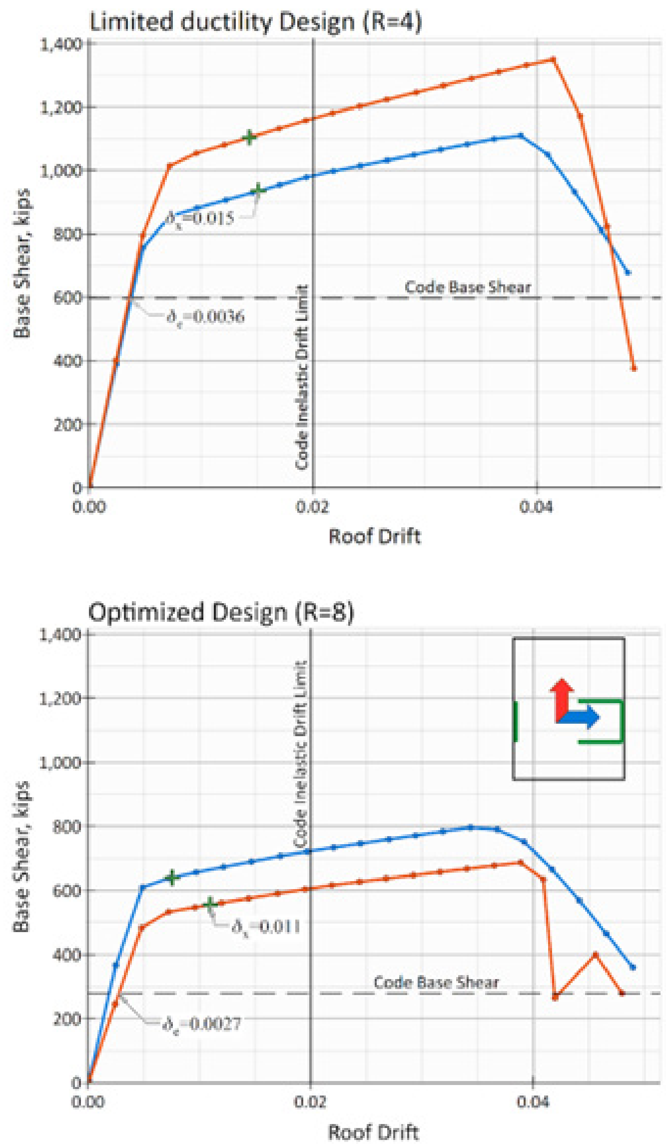


Figure 7 Nonlinear pushover curves for (a) limited ductility design (R=4); (b) optimized design (R=8).

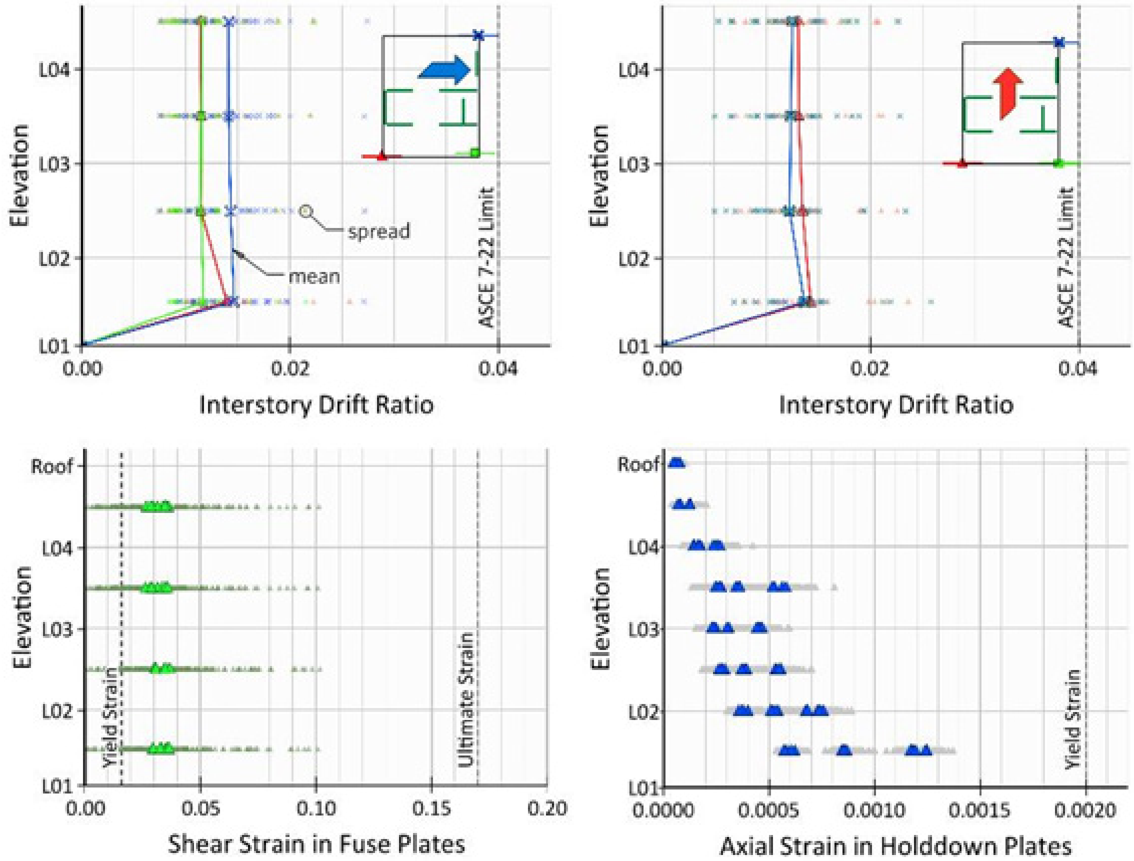


Figure 8 Performance of limited ductility design (R=4) under MCE level hazard.

The axial strain demand in the boundary hold-downs remained under 0.001, significantly below the yield strain. The peak axial strain demand in the hold-downs was lower than in the limited ductility design due to reduced overturning moment demand in the structure.

Low residual drift is desirable for the structure to remain serviceable after an earthquake. The NRHA for the optimized design resulted in mean residual drifts of 0.5% and 0.32% in the global X and Y directions, respectively (Fig. 9). ASCE 7-22 section 16.4.1.3 recommends 1% mean residual drift as the acceptance criteria for buildings exceeding 240 ft in height, but there is no guidance for buildings with lesser height. The mean residual drift demand on the building with the optimized design is sufficiently low to resume serviceability after an MCE-level earthquake, although the YFPs may need replacement and nonstructural components would likely need repair.

Service Level, Design Level, and Beyond MCE

The seismic performance of the building with the optimized design (R=8) was also assessed for multiple hazard levels, namely under the suite of ground motions scaled to 1/3 MCE, 2/3 MCE, and 4/3 MCE. A performance-based seismic design procedure generally requires acceptable performance criteria at the service level (SE) and maximum credible earthquake (MCE). Fig 10 shows the maximum values of the mean responses at the four selected seismic hazard levels.

The mean drift demand in the building remained below 2.5% for the 4/3 MCE hazard level, which is still well below the acceptance limit of 4% for an MCE-level earthquake.

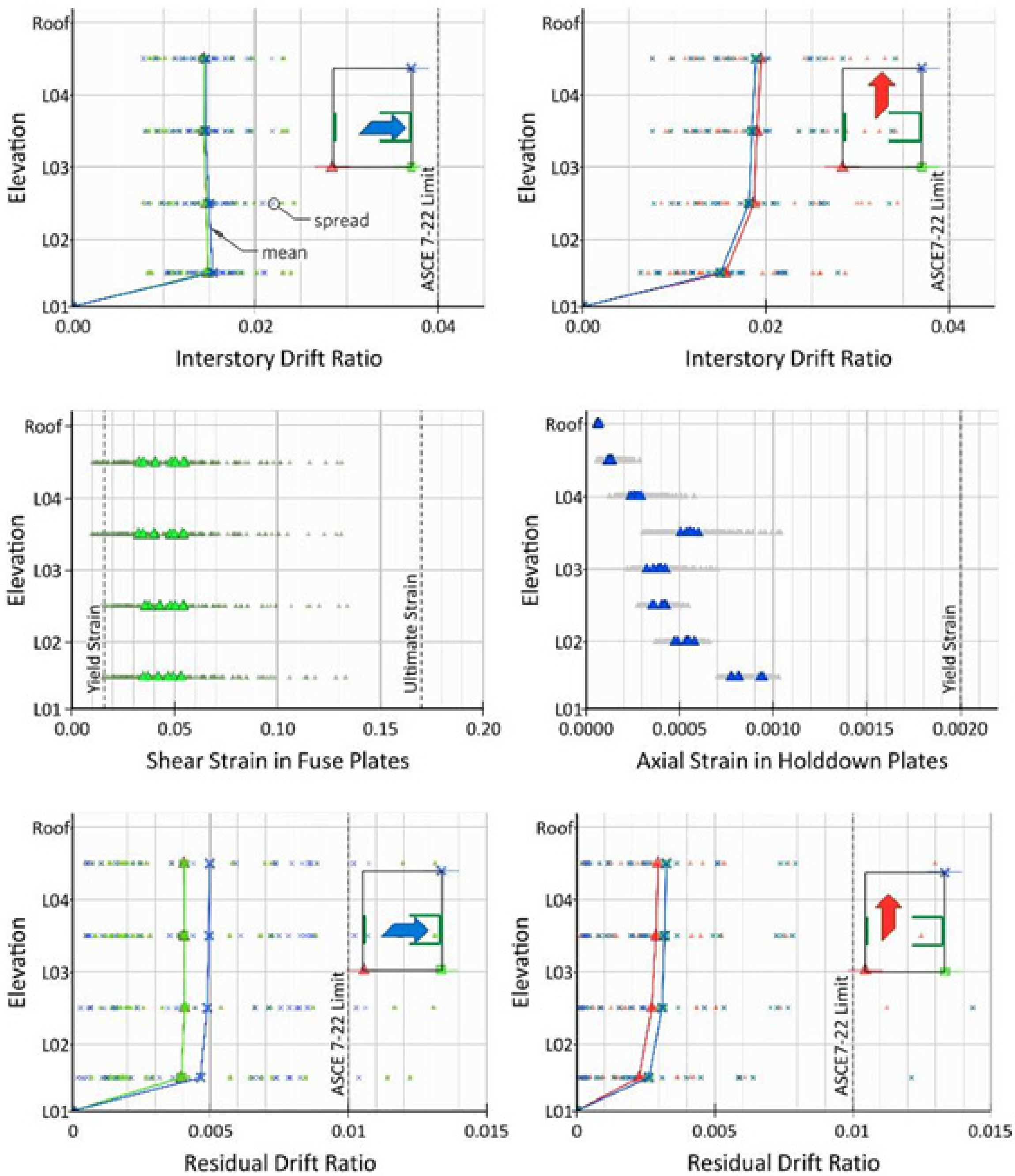


Figure 9 Performance of optimized design (R=8) under MCE level hazard.

The structure is expected to exhibit limited nonlinear behavior when subjected to frequent earthquakes (50% probability in 50 years). The fuse shear strain distribution at 0.33x MCE hazard shows that the structure behaves as expected, i.e. the mean shear strain demand remains below the yield shear strain value of the YFPs. The axial strain in the boundary elements remained below the yield strain value of steel.

Conclusion

These design and analytical studies demonstrate that the proposed novel rocking mass timber shear wall system can provide significant improvements in performance and economy over conventional platform-framed assemblies.

We hope to spur wider adoption of shear walls constructed of CLT and other mass timber panels by developing a seismic design solution that provides a reliable and economical alternative to the currently codified approach. Our strategy relies on making full use of the strength of the panels and coupling them with simple, replaceable, non-proprietary, yielding connections to create an assembly that can be widely adapted for a variety of applications.

With the additional strength and economy provided by the system, the rocking CLT shear walls using YFPs may prove to be a more suitable alternative for mass timber structures, replacing the need to resort to hybrid systems employing steel frames or concrete shear walls.

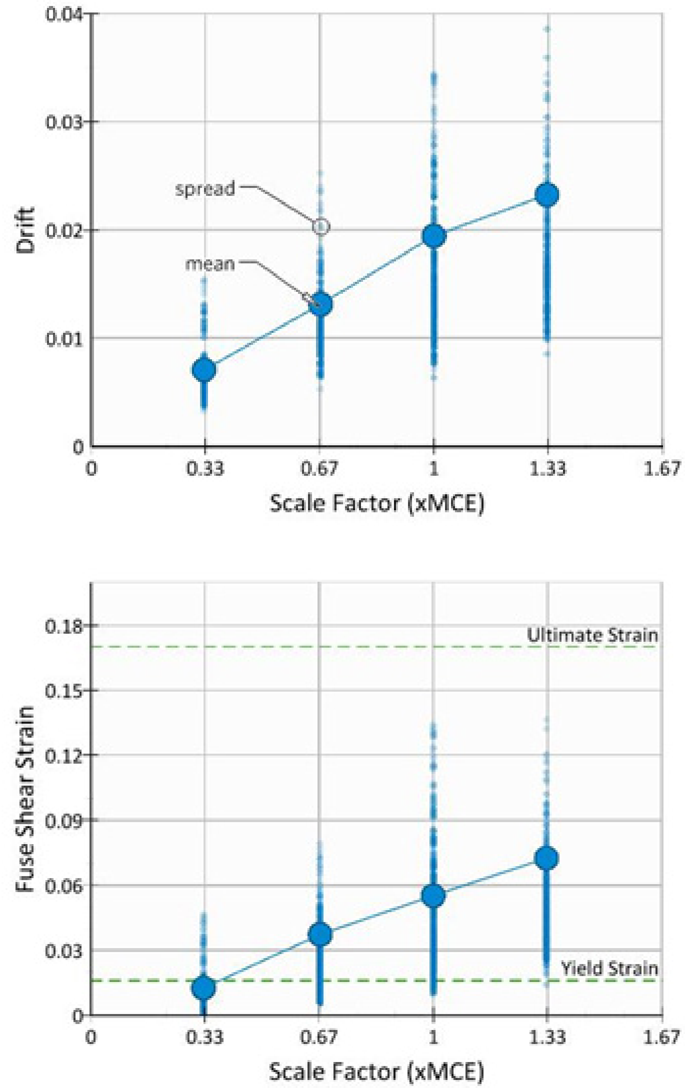


Figure 10 Comparison of mean response at multiple seismic hazard levels.

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