Framework - A tall re-centering mass timber building in the United States

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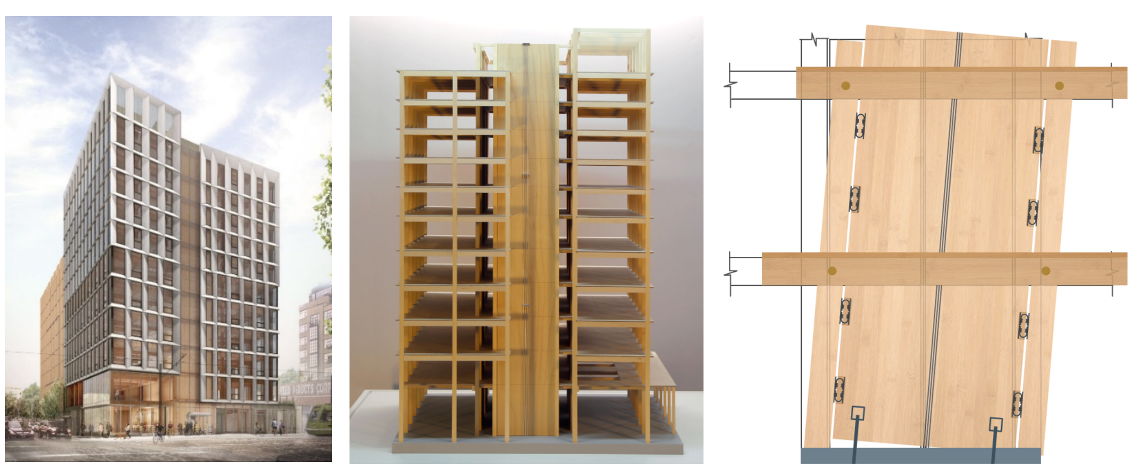
ABSTRACT: Framework is a 12-story, 140ft (43m) tall mixed use building to be constructed almost entirely out of mass timber, including both the gravity and lateral force-resisting systems, in a region of high seismicity in the United States (Portland, Oregon). Utilizing performance-based seismic design and nonlinear response history analysis, the structure's rocking/re-centering cross laminated timber walls were designed for enhanced, beyond-code-level seismic objectives. These enhanced objectives were targeted through more stringent criteria on deformation-controlled elements, design for replacement of energy dissipaters, limitations on residual drift, and a project-specific testing program completed at Oregon State University and Portland State University.

The momentum behind construction of mass timber buildings in the United States provides an opportunity to promote resilient/low-damage design which is consistent with the sustainability goals of many of these projects. This also follows naturally from the inherent rocking/re-centering behavior of mass timber walls. Furthermore, extending rocking mass timber walls to taller buildings is feasible; however, it requires an additional level of thoughtful design, explicit analysis and testing, and careful detailing, including consideration of the effective shear modulus of CLT, wall shear amplification due to higher mode effects, deformation compatibility of gravity connections, and CLT diaphragms.

1 INTRODUCTION

Framework is a 140ft (43m) tall mixed use building to be constructed on a quarter block in Portland, Oregon's Pearl District. See Figure 1a. It will consist of ground floor retail, five levels of office, five levels of residential, and a penthouse. The entire superstructure will be constructed of mass timber, including both the gravity and lateral force-resisting systems. See Figure 1b.



(a) (b) (c)

Figure 1. (a) Architectural rendering, (b) photo of scaled physical model, and (c) schematic view of the base of one rocking CLT wall for Framework. Rendering courtesy of Lever Architecture.

The lateral force-resisting system consists of rocking/re-centering cross-laminated timber (CLT) walls with glulam columns bounding each wall end, as shown in Figure 1c. The CLT walls are externally post-tensioned with threaded rods at the wall centerline and are connected to the bounding glulam columns through U-shaped Flexural Plate (UFP) connectors (Baird et al. 2014). The UFP connectors serve as the primary source of energy dissipation for the building while the post-tensioned threaded rods provide the restoring force. Glulam columns and beams along with CLT floor panels form the gravity force-resisting system. The floor panels and beams deliver gravity loads directly to the columns, permitting the CLT walls to move vertically during rocking without damaging or lifting the floor system. Together, the lateral and gravity force-resisting systems were developed and detailed using the principles of resilient/low-damage design, as discussed in more detail in this paper.

It is acknowledged that significant research on rocking mass timber walls has been completed outside of the United States, specifically in New Zealand. Since the purpose of this paper is to document the Framework project rather than provide a complete literature review, references not specifically pertaining to the design basis for Framework have been omitted.

2 PERFORMANCE OBJECTIVES

2.1 Performance-Based Design

Performance-based seismic design was pursued for Framework because the lateral force-resisting system, consisting of post-tensioned rocking CLT walls is not included in Table 12.2-1 of ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures*. Lateral force-resisting systems included in ASCE/SEI 7-10 Table 12.2-1 may be designed for earthquake effects using the prescriptive provisions in ASCE/SEI 7-10. Systems not included are still permitted as seismic lateral force-resisting systems except they must conform to the performance-based procedures of ASCE/SEI 7-10 Section 1.3.1.3. Performance-based fire design was also pursued for Framework but is outside the scope of this paper.

The performance-based procedures require that "structural and non-structural components and their connections shall be demonstrated by analysis or by a combination of analysis and testing to provide a reliability not less than that expected for similar components designed in accordance with the strength procedures" (ASCE 2010). For Framework, this is achieved by establishing performance objectives at the Design Basis Earthquake, the Risk-Targeted Maximum Considered Earthquake, and the strength-level wind event, as discussed in Section 2.2.1, 2.2.2 and 2.2.3, respectively, of this paper. The performance-based seismic design of Framework was further supported by project-specific testing at Oregon State University and Portland State University. An independent peer review team consisting of practicing structural engineers, an academic professor, and a seismologist, with expertise in performance-based and rocking/re-centering wall design was integral to the outcome of the design.

2.2 Performance Objectives Necessary to Meet Code-Equivalent Performance

2.2.1 *Design Basis Earthquake*

The Design Basis Earthquake (DBE) in ASCE/SEI 7-10 is defined as two-thirds of the Risk-Targeted Maximum Considered Earthquake (MCE_R). For the prescriptive design procedures in ASCE/SEI 7-10, earthquake effects are calculated by reducing the DBE demands by a response modification coefficient, R, which is dependent upon lateral force-resisting system type. Rocking CLT walls, however, do not have a code-established R. As discussed in Section 2.2.2 of this paper, the essential demonstration of equivalent-to-code performance is made through nonlinear response history analysis at the MCE_R. Yet, it is also prudent, though arguably unnecessary, to establish some level of minimum building seismic strength regardless of the deformation- and force-controlled acceptance criteria at the MCE_R. For Framework, a minimum building seismic strength is set by using the R for a code-approved lateral force-resisting system, unbonded post-tensioned precast concrete walls.

Unbonded post-tensioned precast concrete walls have a long history of development and research, most notably that of the Precast Seismic Structural System (PRESSS). ACI 318-11 Section 21.10.3 states that "special structural walls constructed using precast concrete and unbonded post-tensioning tendons […]

are permitted provided they satisfy the requirements of ACI ITG-5.1". This then allows such systems to adopt the $R = 6$ available for special reinforced concrete wall buildings, with detailed design procedures contained in ACI ITG-5.2. Focusing on system response, rather than material type, mass timber rocking walls behave similarly to rocking precast concrete walls (Ganey 2015). For example, the Precast Wall with End Column (PreWEC) system (Sritharan 2015) looks much like the CLT wall with bounding glulam column system employed for Framework. An $R = 6$ is therefore adopted for the DBE for Framework using the design procedures of ACI ITG-5.2 by observing that a rocking mass timber wall can emulate a rocking precast concrete wall.

2.2.2 *Risk-Targeted Maximum Considered Earthquake (MCE_R)*

The Risk-Targeted Maximum Considered Earthquake (MCE_R) was introduced in ASCE/SEI 7-10 to adjust from a uniform hazard to a uniform risk approach in seismic design. For a Risk Category II structure such as Framework, the explicit performance objective is a 10% probability of total or partial collapse given the MCE_R. This is assumed to be met by the prescriptive design procedures. While calculation of the expected probability of collapse is possible via FEMA P-695, this was not pursued for Framework. Instead, limits on global and local acceptance criteria were enforced consistent with state-of-the-art performance-based design guidelines (e.g. 2014 LATBSDC, Chapter 16 of ASCE/SEI 7-16, etc.).

Nonlinear response history analysis is performed for the MCE_R using PERFORM-3D and a suite of 11 two-component ground motions spectrally matched to a site-specific target spectrum. Gravity loads are included using the load combination $D+0.2L_0$ and inherent damping of the structure is limited to 2.5% of critical. Results from the 11 ground motions are condensed in two ways: (1) Suite mean demand is taken as the mean over all earthquakes of the maximum demand for each earthquake, and (2) suite maximum demand is taken as the maximum over all earthquakes of the maximum demand for each earthquake. A story drift ratio limit of 3% is enforced on the suite mean. The 3% story drift limit acknowledges that the ratio between MCE_R and the Design Basis Earthquake is 1.5 and thus the drift criteria of 2% at Design Basis Earthquake needs to be adjusted when being check at MCE_R.

Element actions are classified as either deformation- or force-controlled. Deformation-controlled actions for Framework include flexure in U-shaped Flexural Plate (UFP) connectors, crushing at the toes of the rocking CLT walls, and axial strain in the post-tensioned threaded rods. Force-controlled actions include demands in the reinforced concrete mat foundation, axial-flexure-flexure (PMM) interaction and shear in the rocking CLT walls (excluding crushing at the base), PMM interaction in the bounding glulam columns, demands in diaphragms, chords and collectors, and all member-to-member connections participating in the lateral force-resisting system (e.g. CLT wall splices, CLT wall to floor, post-tensioned rods to foundation, etc.).

Deformation-controlled actions are generally assessed on the suite mean, except that the suite maximum demand must not exceed the valid range of modeling for the component. Force-controlled actions are checked for an amplified suite mean in accordance with ASCE/SEI 7-16 Section 16.4.2.1, as well as the suite maximum. The amplification factor on suite mean demands vary from 1.5 to 2.0 depending on action criticality. For example, bounding glulam columns use an amplification factor of 2.0 because the consequence of failure is loss of gravity-carrying capacity while diaphragm collectors use 1.5. Where a simple mechanism can be identified that limits load delivery to a force-controlled component, capacity design principles are used instead of analysis results (e.g., UFP connection to column limited by UFP ultimate capacity).

Finally, deformation compatibility at the MCE_R story drift was enforced for the gravity system and select non-structural elements. As described in Section 3.3 of this paper, the glulam beam-to-column connection was designed to accommodate large story drift without compromising its ability to carry gravity load. Egress stairs were also detailed to accommodate movements exceeding two times the suite mean MCE_R story drift.

2.2.3 *Strength-Level Wind*

A site-specific wind study was conducted for Framework in accordance with ASCE/SEI 7-10 Section 26.5.3 to determine the 3-second gust velocity corresponding to a 700 year recurrence interval using regional climatic data. This permitted a reduction in basic wind speed from 120 mph (193 km/hr) to 97 mph (156 km/hr). A wind tunnel study was not performed.

Strength-level wind exceeded Design Basis Earthquake demands at the base of the rocking CLT walls. Design procedures for strength-level wind were taken, similar to the Design Basis Earthquake, from ACI ITG-5.2. It should also be noted that ASCE/SEI 7-10 does not include mandatory limits on building drift under wind demands. It is expected that design criteria for wind effects will garner additional scrutiny as mass timber buildings are built to greater heights in the United States.

2.3 **Voluntary Performance Objectives**

2.3.1 *Serviceability Wind*

A serviceability wind event was considered for Framework to target occupant comfort, although not required by ASCE/SEI 7-10. A 25 year recurrence interval wind event velocity was provided by the site-specific wind study referenced in Section 2.2.3 of this paper. The serviceability wind criteria included:

- Limit of 1/500 on story drift ratio when subjected to the 3-second gust velocity. This was based on recommendations contained in ASCE/SEI 7-10 Section CC.1.2.
- Limit of 0.015g and 0.020g on peak along-wind acceleration for residential and office floors, respectively, when subjected to the mean hourly velocity. This was based on recommendations contained in AISC Steel Design Guide 3 *Serviceability Design Considerations for Steel Buildings*.
- Disallowing wall rocking when subjected to the 3-second gust velocity. This was included for occupant psychological peace of mind. It was discussed among the design team that observing uplift of the rocking CLT walls, many of which are architecturally exposed at the base, may be disturbing to building occupants.

2.3.2 *Serviceability Earthquake*

The performance objective of essentially elastic response under an earthquake having a 50% probability of exceedance in 30 years – equivalent to a 43 year return period event – was initially identified for Framework. However, it became clear that, due to the shape of the seismic hazard curve in Portland, Oregon, the 43 year hazard would not control in establishing minimum seismic strength. Instead, disallowing wall rocking, similar to the criteria for the serviceability wind event, became the only criteria for the serviceability earthquake.

2.3.3 *Repairability Earthquake*

The project goals for Framework included an emphasis on sustainability and resilience. These goals were met in the structural system in part through evaluation of the resilient/low-damage design at an earthquake having a 10% probability of exceedance in 50 years, equivalent to a 475 year event. A performance objective of repairable damage was achieved for Framework by performing nonlinear response history analysis using 11 two-component ground motions spectrally matched to the site-specific 475 year target spectrum. Since the design approach for force-controlled components at the MCE_R essentially ensure elastic performance, the additional criteria at the 475 year event focused on limiting residual drifts and deformation-controlled actions.

Suite median residual drift ratios were limited to 0.2%. This limit corresponds to the first damage state in FEMA P-58 for residual drift, described as “no structural realignment is necessary for structural stability; however, the building may require adjustment and repairs to non-structural and mechanical components that are sensitive to building alignment (e.g., elevator rails, curtain walls, and doors)”. The limit of 0.2% is approximately “equal to the maximum out-of-plumb tolerance typically permitted in new construction" (FEMA 2012). Suite median demand is taken as the median over all earthquakes of the maximum demand for each earthquake and was used, rather than suite mean, to acknowledge the fact that dispersion in predicted residual drifts tends to exceed that for other design quantities (e.g. maximum story drifts).

Further criteria at the 475 year earthquake event included tighter deformation-controlled limits and detailing to ensure repairability. For example, while wall crushing was permitted at the MCE_R, compression at the wall toe under the 475 year event was kept essentially elastic. The U-shaped Flexural Plate (UFP) connectors, the main source of hysteretic energy dissipation in the building, were designed to be replaceable. With such a repair and low residual drifts, the building could be returned nearly to its pre-earthquake state.

3 UNIQUE DESIGN ASPECTS OF A TALL, ROCKING TIMBER WALL BUILDING

3.1 Effective shear modulus of CLT

A comparison of in-plane stiffness of a CLT9 panel and a similar thickness reinforced concrete wall is made in Table 1. As can be seen, the effective in-plane flexural properties of CLT are about half that of cracked concrete. The effective in-plane shear stiffness of the CLT9 panel is much, much smaller than the concrete wall. This comparison illustrates the greater importance of shear deformations in the design and analysis of CLT wall buildings.

Table 1. Comparison of CLT to reinforced concrete in-plane stiffness.

	Reinforced Concrete f'_c = 5000psi	CLT9 DF No. 1 E = 1800ksi	CLT9 / Concrete
EI_{eff}	2015ksi * I_g	1025ksi * I_g	50%
GA_{eff}	1610ksi * A_g	(35 to 75 ksi) * A_g	2 to 5%

Notes:
1. Cracked section modifiers for concrete taken as 0.5 for EI_{eff} and 1.0 for GA_{eff}.
2. GA_{eff} for CLT based on Flaig and Blaß (2013) and Brandner et al. (2015).

In-plane shear stiffness of CLT has not been extensively researched. Currently, two formulations for estimating in-plane shear stiffness of CLT exist as described in Flaig and Blaß (2013) and Brandner et al. (2015). These formulations predict effective shear stiffness that is different by approximately a factor of two. As part of the Framework project, physical testing of large-scale CLT panels was conducted at Oregon State University to, among other things, better determine the expected shear stiffness of CLT.

Figure 2 shows the effective shear modulus results from the Oregon State University CLT panel testing compared against the value predicted by Flaig and Blaß (2013) and Brandner et al. (2015). First, observe that the effective shear modulus is a function of shear stress, with stiffness generally decreasing with increasing shear stress. Also observe that the Brandner et al. (2015) procedure more closely matches the results from the Oregon State University tests than the Flaig and Blaß (2013) formulation, even though both tend to underestimate the effective shear modulus within typical ranges for shear stress. It should be noted that the Oregon State University CLT panel tests were subject to constant axial compression during testing which may have contributed to a higher effective shear modulus.

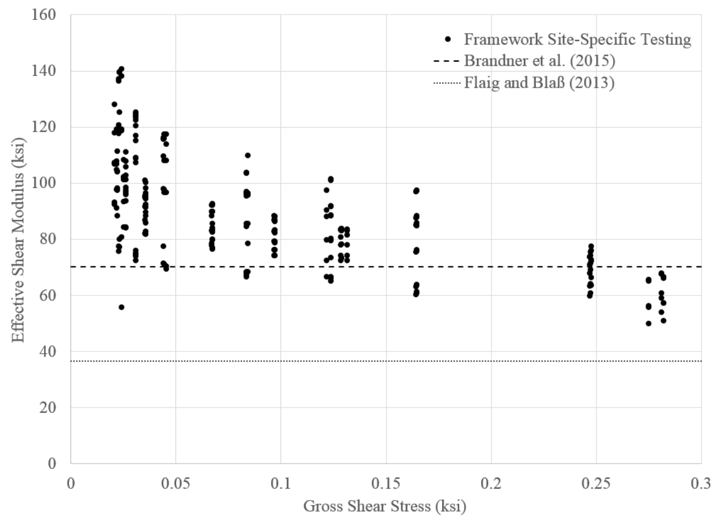


Figure 2. Effective shear modulus from Oregon State University CLT panel testing compared to Brandner et al. (2015) and Flaig and Blaß (2013).

To account for the dispersion in effective shear modulus, the Framework project used bounding analyses (i.e. analysis of all ground motions using separately an upper- and lower-bound stiffness). The design was required to satisfy the greater demands from either of the upper- and lower-bound analyses.

3.2 **Seismic Shear Demand**

It is well established that shear demands in tall concrete wall buildings, for which higher modes are significant, are underestimated by response spectrum analysis when a uniform response modification coefficient, *R*, is used across all modes. A similar observation was made for Framework. In fact, as discussed in Section 3.1, because of the reduced flexural and shear stiffness of CLT as compared to reinforced concrete, it is likely that the building roof height at which higher modes become more important is lower for CLT wall buildings than concrete wall buildings (assuming floor mass is similar).

As seen in Figure 3, the seismic shear demand from the nonlinear response history analysis for Framework significantly exceeds that from response spectrum analysis using an R = 6. While it is expected that the shear demand at the MCE_R (nonlinear response history analysis) would exceed that at the Design Basis Earthquake (response spectrum analysis using an R = 6) simply because the MCE_R equals 1.5x the Design Basis Earthquake, as seen in Figure 3, the difference is closer to 6.5x at the base. This is a result of several, cumulative effects. Firstly, the actual *R* is much lower than 6 because (a) wind demands exceed seismic demands, and (b) overstrength exists in the system due to overdesign, the difference between nominal and expected properties, and use of strength reduction factors, φ, less than 1.0. Secondly, and more significant than the first effects, rocking of the CLT walls only tends to reduce the response in the first mode. The second and third modes, which produce very low base moment but high story shears, act essentially elastically in the nonlinear analysis whereas they are assumed to be reduced by *R* in the response spectrum analysis.

While the introduction of a second rocking plane at a height corresponding to the peak second or third mode moment may be a solution to reducing the second and third mode shears, it is the authors' opinion that aspects of designing and detailing such a building still need further study before practical implementation.

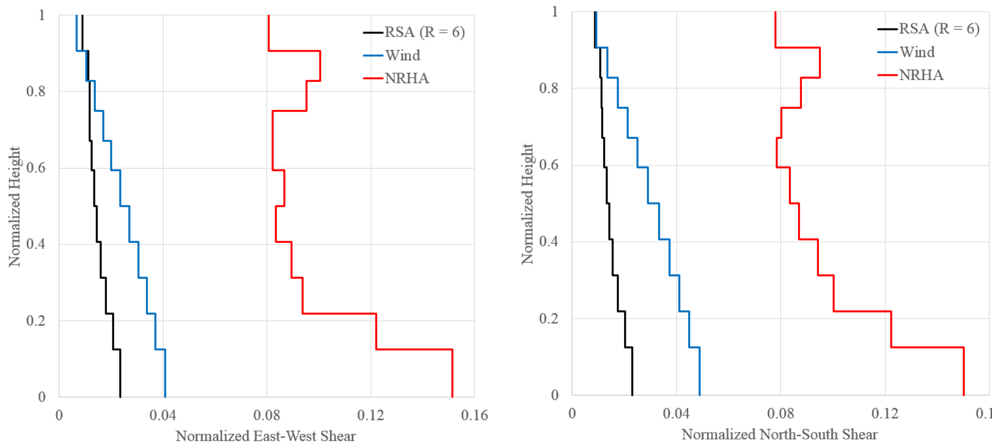


Figure 3. Seismic shear demand comparison for Framework between response spectrum analysis (RSA) using an $R = 6$, strength-level wind, and suite mean nonlinear response history analysis (NRHA). Shear is normalized by building weight. Height is normalized by roof height.

3.3 Deformation Compatibility of Gravity Connections

As described in Section 2.2.2 of this paper, it was necessary to design a glulam beam-to-column connection for Framework which could sustain the expected story drift at the MCE_R without loss of gravity-carrying capacity. In addition to structural constraints, there were significant architectural and fire design considerations for this connection. Much of the architectural appeal of mass timber is in the exposure of wood and the clean lines between CLT panels, glulam beams and glulam columns. For Framework, a tight fit between the glulam beam and column was desired to be fully exposed. However, the connection also needed to achieve a 2 hour fire rating on select floors.

Before the Framework project, a connection which met all of these structural, architectural and fire constraints, in addition to constructability practicalities, did not exist. Therefore, a connection was envisioned, utilizing a relatively simple bearing connection and disc springs, which separated the gravity-carrying capacity from the connection's lateral stability. Full scale, cyclic, quasi-static testing of three beam-column subassemblies, including expected gravity loading on the beam, was then conducted at Portland State University. Figure 4 shows the connection at the ±3% drift cycles.

Figure 4. Glulam beam-column connection tested at Portland State University. Connection shown at ±3% drift cycles.

The Portland State University testing successfully demonstrated the ability of the connection to undergo ±6% story drift ratio without loss of gravity-carrying capacity. It is possible the connection could accommodate even greater story drift had the actuator stroke capacity not been reached. Furthermore, the testing showed that story drift ratios on the order of ±2% could be realized without damage necessitating repair.

Gap opening at the beam-column interface does lead to beam elongation. In order to avoid potentially detrimental effects on the CLT diaphragm, the connection at the other beam end for Framework was designed to relieve the elongation. Since this connection was completely enclosed by a fire-rated soffit, and thus also not architecturally exposed, greater design freedom was possible.

3.4 CLT Diaphragms

While CLT diaphragms had been previously used in high seismic regions in the United States, the core-like configuration of rocking CLT walls on Framework presented a different support condition than that typically seen (i.e. simple span). It was therefore of interest to investigate diaphragm deformations and forces. This was evaluated through the use of a modeling technique for CLT diaphragms consisting of membrane elements for the CLT panels connected together by discretized nonlinear springs to represent the CLT panel-to-panel connections. For more information, reference Breneman, McDonnell and Zimmerman (2016).

It should be noted that diaphragm design in accordance with the prescriptive provisions of ASCE/SEI 7-10 does not ensure elastic diaphragm performance. For Framework, however, the CLT diaphragms were kept essentially elastic at the MCE_R by imposing the suite maximum acceleration at each floor. The maximum cap on diaphragm forces of ASCE/SEI 7-10 Equation 12.10-3 was neglected.

4 CONCLUSION

Framework represents an evolution of mass timber design and construction practice in regions of high seismicity in the United States. Lessons learned from the project are shared to aid future projects in thoughtful design, explicit analysis and testing, and careful detailing.

- Performance-based seismic design can be used to establish code-equivalent performance for rocking/re-centering mass timber wall buildings in the United States.
- Momentum behind mass timber construction provides an opportunity to promote resilient/low-damage construction, and design for repairability.
- As mass timber rocking/re-centering wall buildings are built to greater heights, it will be necessary to more closely scrutinize this building types' unique design aspects, including the effective shear modulus of CLT, wall shear amplification due to higher mode effects, deformation compatibility of gravity connections, and CLT diaphragms, among others.

5 ACKNOWLEDGEMENTS

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