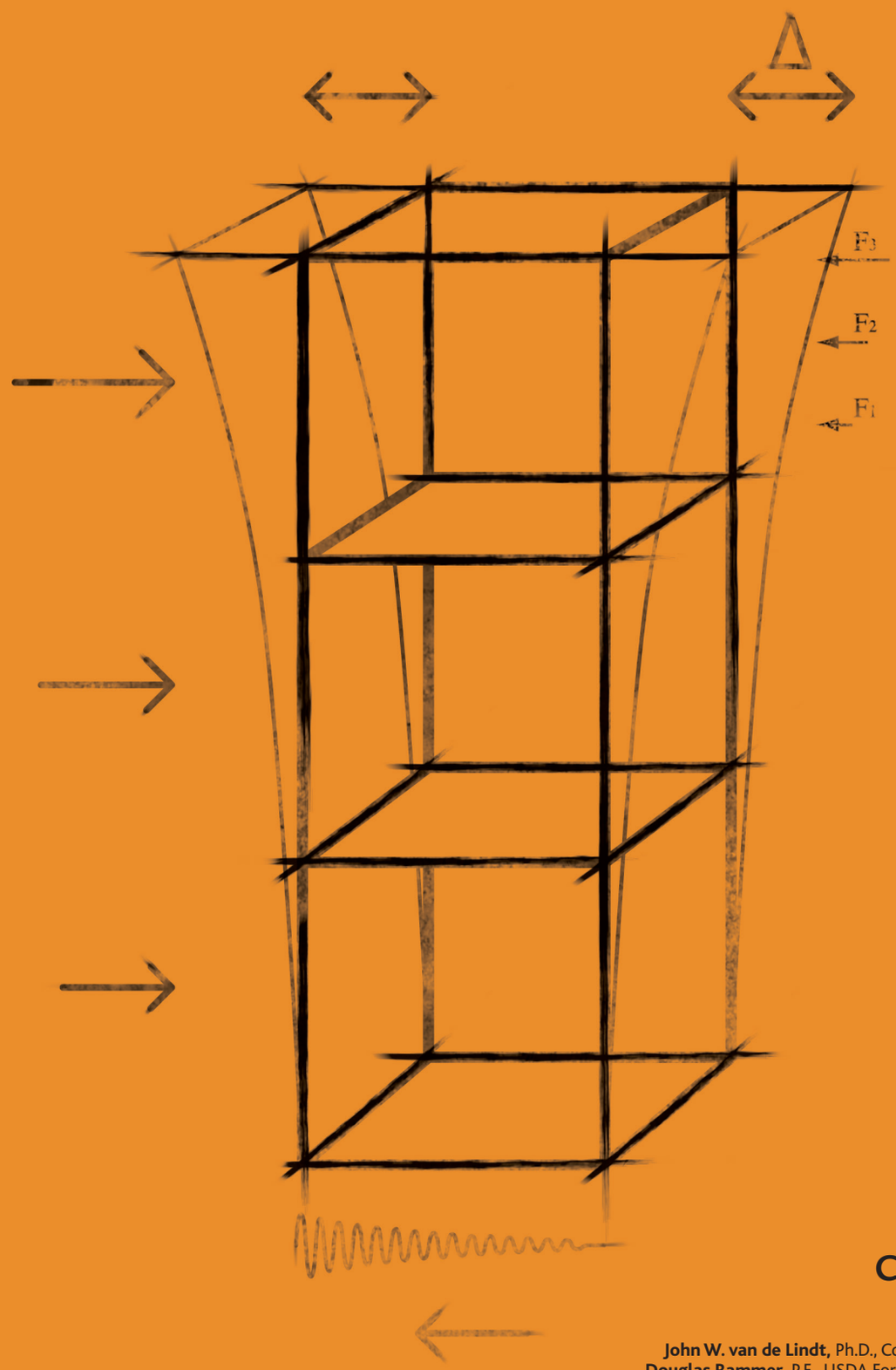

$$F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V$$

Lateral

Lateral design of cross-laminated timber buildings

CHAPTER 4

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ABSTRACT

Cross-laminated timber (CLT) is an innovative wood product that was developed approximately two decades ago in Europe and has since been gaining in popularity. Based on the experience of European researchers and designers, it is believed that CLT can provide the U.S. market the opportunity to build mid- and high-rise wood buildings. This Chapter presents a summary of past research and state-of-the-art understanding of the seismic behavior of CLT. As a new structural system to the United States, the design of CLT for seismic applications is expected to be made through alternative method provisions of the building codes. Efforts to develop seismic design coefficients for use in the equivalent lateral force procedures in the United States are underway. Nonlinear numerical modeling of CLT is presented and used to provide an indication of the effect of designing with different R-factors. Using nominal CLT wall capacity values derived from isolated wall tests, the illustrative example showed that an R-factor of approximately 2 can result in a low probability of collapse (less than 10 percent) at MCE intensity.

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1
INTRODUCTION

In Europe, a structural system called cross-laminated timber (CLT) was introduced about 20 years ago as an efficient and environmentally friendly choice for panelized construction in residential and non-residential buildings. Several buildings around the world, primarily in Europe, have been completed, including a nine-story CLT building in London, England (see Figure 1). The current approach used to construct multi-story CLT buildings relies on mechanical connectors (metal brackets and fasteners) to connect the wall and floor panels.

Requirements for the design of CLT for lateral load resistance due to wind and seismic loads are in the early stages of U.S. standardization for eventual recognition by the U.S. model building codes. Design for wind employs use of linear-elastic analysis of wind load effects and values of design resistance for members and connections derived by standard methods. For seismic design, methods for establishing both loads and resistance are more complex and require consideration of nonlinear behavior of the structural system. Lateral design of CLT for both wind and seismic are addressed herein; however, additional information is provided to address seismic loading and response commensurate with the increased complexity of evaluation of seismic performance of the structural system.

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Figure 1
Typical CLT panel, typical CLT construction and a rendering of the 9-story CLT building in England

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2
SHEAR WALLS AND DIAPHRAGMS

Resistance to lateral loads from wind and earthquakes in CLT structures is made through wall and floor panels designed as shear walls and diaphragms. Unlike typical wood-frame shear wall and diaphragm systems, which have design unit shear strength prescribed based on a specific construction (AWC, 2008), the design shear strength of CLT shear walls and diaphragms is determined directly based on principles of engineering mechanics using provisions of the National Design Specification (NDS) for Wood Construction (AWC, 2012) for connection design (see Chapter 5 entitled *Connections in cross-laminated timber buildings*) and CLT panel design (see Chapter 3 entitled *Structural design of cross-laminated timber elements*). The unit shear strength of the CLT shear wall and CLT diaphragm will vary by factors such as fastener size, spacing, and location as well as the shear strength of the CLT panel itself. Deflection estimates based on principles of engineering mechanics or derived from testing should account for all sources of deflection including panel bending, panel and/or connector shear, and fastener deformation.

Capacity design principles are recommended for design of CLT for seismic resistance to ensure predictable yielding in CLT wall panels and interconnection of CLT elements through fastener yielding, wood crushing, or a combination thereof prior to onset of undesirable brittle wood failure modes. This approach recognizes that wall panel-to-panel and panel-to-floor connections provide the primary source of yielding, while the wood wall panels themselves remain essentially elastic. Special seismic detailing of the CLT seismic force resisting system includes the following:

a) Fastener Requirements. Fasteners loaded in shear are designed such that the expected fastener yield mode is Mode III or Mode IV as defined in the NDS.

b) CLT Member Design at Connections. The nominal connection capacity determined in accordance with Item c) is used as the minimum design demand for any wood member limit states.

c) Nominal Connection Capacity. The nominal connection capacity in shear is determined in accordance with the following:

   For dowel type fasteners -- $n \times Z(K_F)(l)(C_M)(C_t)(C_{eg})$ where n is the number of fasteners; Z is the reference lateral design value for a single fastener; and K_F, l, C_M, C_t, and C_{eg} are adjustment factors specified in the NDS for format conversion, time effect, wet service, temperature and end grain, respectively.

d) Fastener Withdrawal. Where a connector relies directly on fastener withdrawal for load resistance, testing shall demonstrate that the fasteners can develop substantial yielding in the connector prior to full fastener withdrawal.

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Using the recommended capacity design approach, typical connections at Locations 1 through 4 (see Figure 2) should employ fasteners designed to yield per Mode III or Mode IV. The typical CLT connection locations include:

1 - Vertical joints between perpendicular walls

2 - Horizontal half-lapped joints between floor panels

3 - Connections between floor panels to the walls below

4 - Vertical half-lapped joints between wall panels

In addition, the design strength of wood strength limit states at the connection locations should not be less than required to develop the nominal connection capacity (i.e., minimum design demand determined in accordance with Item c). Wood strength limit states include row and group tear out, shear, and tension in accordance with the NDS. Employing the recommended design approach will promote connection designs with larger spacing between fasteners, larger end distances, and use of relatively slender dowel type fasteners in order to ensure fastener yielding prior to onset of brittle wood failure modes.

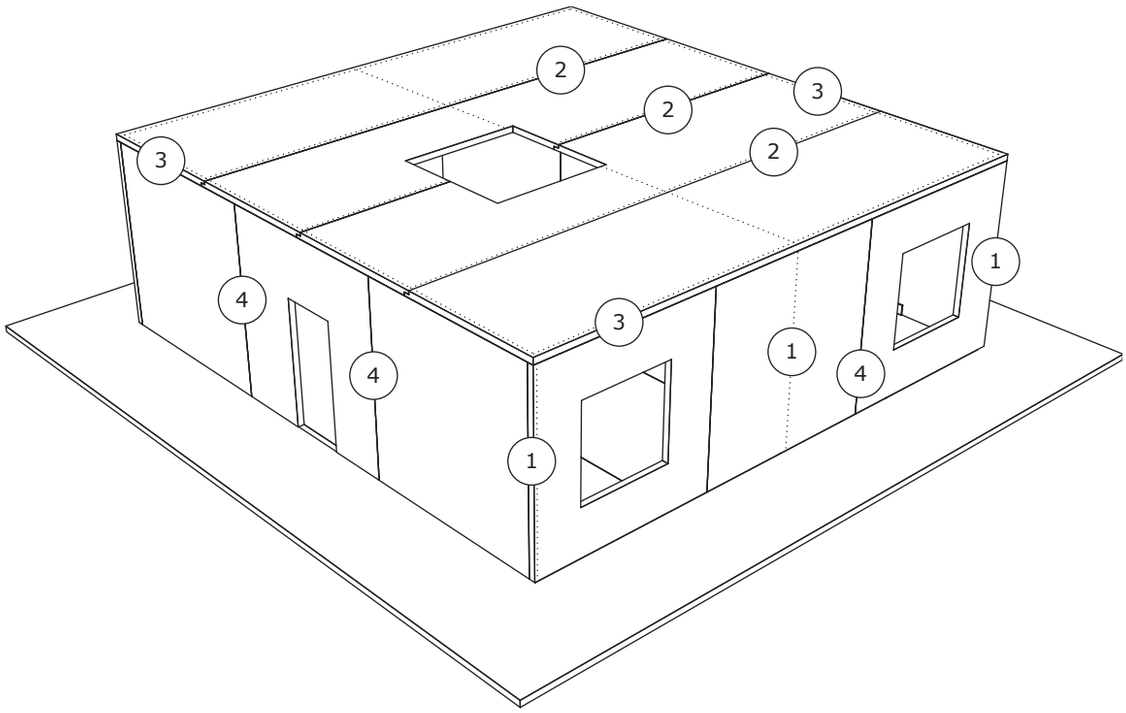

Figure 2
Typical story of a CLT structure with various connections between the panels (drawing courtesy of A. Ceccotti)

While capacity design concepts can be extended to design for wind and will provide for improved performance of the structure in extreme load conditions in the event that overload of the system occurs, it is not required given that the basic design approach for wind is to design for linear elastic response of the structural system.

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3
SEISMIC DESIGN

The equivalent lateral force (ELF) procedure is one of the most commonly used analytical procedures for seismic design of buildings in the United States. The procedure relies on system-specific seismic design coefficients, R, C_d, and Ω_o for determination of seismic loads and compliance with associated design requirements. While CLT seismic performance has been evaluated at the wall panel level using reversed-cyclic testing, at the system level using multi-story shake table testing, and numerically via structural modeling, there are no recognized seismic design coefficients for CLT in U.S. design standards or model building codes.

Efforts towards determination of these seismic design coefficients have only recently begun in the United States and a comprehensive evaluation of CLT using FEMA P695 Quantification of Building Seismic Performance Factors for determination of seismic design coefficients has not yet occurred. Until such time that the FEMA P695 evaluation is complete and codes and standards specifically address design of CLT through appropriate seismic design criteria, recognition of CLT use for seismic resistance will be through building code compliance pathways for alternative materials, design and methods of construction.

Expected compliance pathways for recognition of specific CLT designs for seismic resistance include the use of the performance-based design procedures described in ASCE 7-10 Minimum Design Loads for Buildings and Other Structures (ASCE, 2010), or demonstration of equivalence to an existing system in ASCE 7. Guidance for such evaluations can be derived directly from ASCE 7-10, FEMA P695, and FEMA P795 Quantification of Building Seismic Performance Factors: Component Equivalency Methodology. Regardless of the approach taken, information on CLT seismic performance from testing and results from modeling are necessary to guide decisions on the appropriate design of CLT for seismic resistance. The information contained in this Chapter is working toward that goal but does not yet provide a full suite of information to enable a designer to do a performance-based seismic design.

3.1 FEMA P695 Quantification of Building Seismic Performance Factors (FEMA, 2009)

For CLT systems employing CLT wall and floor components as the seismic force resisting system, the primary method for determining seismic design coefficients for eventual recognition in ASCE 7 and the model building codes is expected to be through an evaluation in accordance with the FEMA P695 (2009) methodology. The methodology is designed to establish seismic design coefficients for structural systems to meet the minimum seismic performance objective – less than 10% probability of total or partial collapse given occurrence of Maximum Considered Earthquake (MCE) shaking for Risk Category I and II buildings (ASCE, 2010). The methodology is rigorous and requires evaluation of numerous configurations of a given system to encompass a range of behaviors for that system. The key elements of the methodology include requirements for ground motions, analysis methods, test data, design, and peer review at each step in the process. While a significant portion of the methodology addresses evaluation of numerous configurations, the key elements of

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the methodology can also be applied for building specific evaluation of collapse risk to satisfy the minimum seismic performance objective. While still rigorous, the evaluation of a single building on a project-specific basis is considerably less effort than required for determination of broadly applicable seismic design coefficients. For additional information on P695, readers are referred to the full report (FEMA, 2009).

3.2 FEMA P795 Quantification of Building Seismic Performance Factors: Component Equivalency Methodology (FEMA, 2011)

For CLT components such as a CLT wall panel, determination of seismic design coefficients may be made through the application of the FEMA P795 methodology. Procedures of the FEMA P795 methodology do not evaluate compliance with the 10% probability of collapse performance objective of a building's complete seismic force resisting system directly as done in FEMA P695. Instead, a direct comparison of seismic performance of the "proposed" component to a "reference" component of a recognized seismic force resisting system is made to determine whether the proposed component provides equivalent seismic performance. Proposed components found to be equivalent are then judged to be suitable substitutes for components of the reference seismic force resisting system and can therefore utilize seismic design coefficients (SDCs) applicable to the reference seismic force resisting system. Reference components and reference seismic-force-resisting systems are defined by provisions of ASCE 7, the building code, and reference design standards. Key parameters in the evaluation include ultimate deformation, strength, initial stiffness, and ductility. Like the FEMA P695 methodology, peer review is a requirement of the methodology.

The extent to which proposed CLT wall panels can be substituted for a reference component without changing the character of the seismic force resisting system is a consideration under the FEMA P795 methodology. While precise rules do not address the extent of the substitution permissible under the methodology, use of FEMA P795 for recognition of CLT wall panels is expected to be limited. The exclusive use of CLT wall panels in combination with CLT floor panels as part of a CLT building system is considered to be outside of the scope of evaluation per the FEMA P795 methodology. For additional information on P795, readers are referred to the full report (FEMA, 2010).

3.3 Performance-based Seismic Design (ASCE, 2010)

Performance-based seismic design procedures of ASCE 7 may be applied to designs employing CLT for seismic resistance. Under requirements of ASCE 7, the reliability of the proposed component must not be less than that expected for a similar component designed in accordance with the strength procedures of ASCE 7. Requirements for analysis, testing, documentation, and peer review are also included in the procedures. Commentary to these procedures identifies the minimum performance objective for Risk Category I and II structures as 10% probability of total or partial collapse given occurrence of MCE shaking.

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4

LITERATURE REVIEW ON LATERAL RESISTANCE FOR CLT DESIGN

Lateral load-response testing of CLT has focused primarily on seismic loading, often reversed cyclic tests or shake table testing of components and, in a few cases, on full structures. Most results can be applied to wind, but again it is stressed that current design for wind load seeks to keep the structural response in the linear-elastic range. More specifically, seismic CLT testing has included both testing of individual wall panels and multi-story shake table testing of both a low-rise and a mid-rise CLT building. A significant amount of component testing and analysis has been conducted in order to better understand CLT behavior when subjected to cyclic loading. Information from these studies will provide a basis for design recommendations for appropriate use of CLT as well as provide supporting information for future implementation of the FEMA P695 methodology. Significant CLT studies are summarized in Appendix A as follows:

4.1 The SOFIE Project

This study included tests on connections, CLT wall panels, and multi-story shake table testing of a 3-story and a 7-story CLT building. In that specific project, it was experimentally demonstrated that the behavior (strength, stiffness) of the panels was heavily dependent on the location and behavior of the connectors.

4.2 University of Ljubljana, Slovenia

This study investigated various boundary conditions in order to develop load versus displacement relationships ranging from cantilever to pure shear wall behavior. The effect of vertical loads and anchorage on the CLT wall behavior was also systematically investigated. The SAP2000 commercial software was used to develop models of 36 different types of walls. Shake table testing to identify overall dynamic properties of CLT sub-assemblies was also part of the project.

4.3 Karlsruhe Institute of Technology

This study included CLT wall panel tests and performance comparison to “traditional” timber-frame construction.

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4.4 FPInnovations Study

This CLT wall panel testing program consisted of a total of 32 monotonic and cyclic tests of CLT wall panels in 12 different configurations employing different wall-to-floor, wall-to-wall, and two-story connection details. Appendix A provides a full description of the FPInnovations study that was reproduced, in part, from the Canadian edition of CLT Handbook (2010). Several key observations from the above studies, and particularly the FPInnovations study, are provided below.

General observations of CLT behavior determined from cyclic testing of wall panels and multi-story shake table testing can be summarized as follows:

a) Connection of the wall panel to the supporting element provides the primary source of yielding, while the wall panels themselves remain essentially elastic. In other words, the wall panel itself behaves almost as a rigid body when racking. Connection yielding (whether from individual fasteners, shear connectors or overturning restraint) provides the primary source of deformation; however, wood failure at connections may occur where fasteners are too closely spaced.
b) Cyclic in-plane shear behavior of wall panels is not degraded by the presence of axial load. Walls tested with axial load had increased initial stiffness and peak shear capacity. Although not routinely observed in testing to date, toe crushing of the walls should be considered in design.
c) Significant ductility was observed where boundary conditions allowed rigid body rotation of wall panels. This ductility is achieved through the connectors between the wall panel and floor/ceiling diaphragm.
d) Failure mechanisms observed in multi-story shake table tests were concentrated in the wall panel connections.

In Appendix A of this Chapter, a greater level of detail is provided for testing undertaken as part of the FPInnovations study. Many of those tests utilized the CUREE cyclic load protocol, which facilitates comparison to other wood systems tested using the CUREE protocol (Krawinkler, 2000). Additionally, the experimental plan focused on the key issue of the connection of the wall panel to supporting elements and influence on cyclic performance. Detailed descriptions of the connectors, fasteners, observations from testing including photographs, and load deformation response for CLT wall panels are provided in Appendix A.

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5
NUMERICAL MODELING OF CLT

This section provides a relatively simple but reliable modeling technique for determining the numerical response of CLT building systems. Although complex nonlinear finite element models may also be established for CLT systems (as discussed in the literature review), the method presented herein seeks a balance between model complexity, accuracy, and ease of integration in both research and design practice. The modeling techniques presented in this section were used in performing the example which is included as Appendix B of this Chapter.

5.1 Kinematic Model for CLT Walls

Existing wall test observations indicated that, under lateral loading, the shear deformation of the CLT wall panel itself is insignificant compared to the deformation of panel connections. In other words, lateral displacement of a CLT wall is mainly caused by panels rotating as a rigid body about the corners, as shown in Figure 3. Such rotation of CLT panels installed in between the floor and ceiling panels will be confined by the floor and ceiling diaphragm panels to some degree when compared to isolated wall tests. The rotational behavior has been observed in complete CLT structure lateral loading tests at FPInnovations.

Figure 3
Rocking behavior of a two-panel CLT wall during testing (FPInnovations)

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A simplified model was developed and is explained in this Chapter; it can be used to estimate the hysteresis of the wall systems comprised of CLT panels. Designers should be aware that the effect of some of the model assumptions can be significant and are subject to future refinement. The assumptions used to develop this model include:

- CLT wall panels behave as in-plane rigid bodies.
- Under lateral loading, CLT wall panels will rotate individually around the bottom corners to develop lateral displacement at the top of the wall.
- There is no relative lateral slip between the wall and floor (or ceiling) panels.
- The gravity force acts vertically through the center of the CLT wall panels.
- Panel connectors (hold-downs, brackets, etc.) will be deformed during the rocking motion of the wall and develop the hysteresis for the wall panel system.

These kinematic assumptions are shown in Figure 4. Based on free body equilibrium, the lateral resistance of a CLT wall is represented as a scaled summation of the load-slip resistance of all the connectors engaged in the rocking movement of the wall. The scale factor for each connector is a function of their location and the geometry of the panel.

The resistance F at lateral displacement D may be calculated as:

$$F(D) = \sum_{i=1}^{n} \frac{l_i}{H} f_i(d_i) + \frac{L}{2H} G \quad \text{and} \quad d_i = \frac{l_i}{H} D \qquad [1]$$

where L is the panel length; H is the panel height; D is the lateral displacement at top of the wall panel; l_i is the distance of the ith connector to the center of panel rotation; d_i is the deformation of the ith connector under rocking motion; F(D) is the nonlinear wall resistance force as function of D; $f_i(d_i)$ is the nonlinear force function of the ith connector as function of its deformation d_i.

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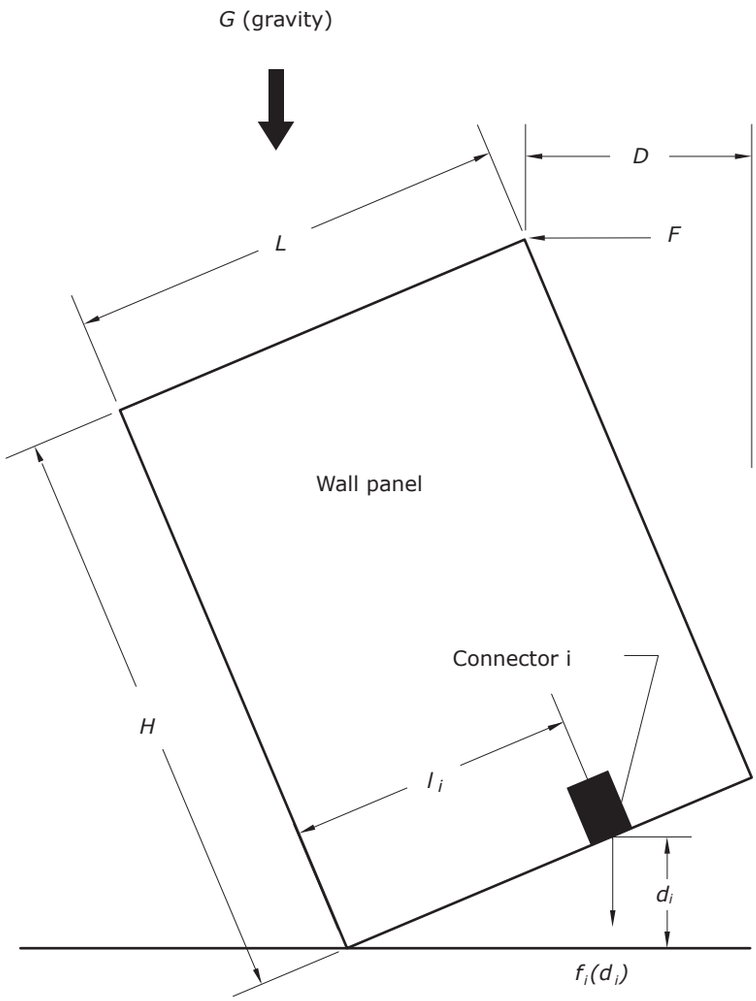

Figure 4
Kinematic model of a single CLT wall panel

The simplified kinematic assumptions in the model result in several limitations, including:

(1) The model is only valid up to a certain story drift level due to small angle approximation; and

(2) The length of the wall panel needs to be short enough that the assumed rotation in the model is valid (panels with large length to height ratios will not be able to rotate as assumed in the model).

The lateral response predicted by the model using the calibrated connector hysteretic parameters was felt to be accurate compared to the FPInnovations experimental results.

5.2 Wall Model Calibration Using Test Data

While the kinematic model provided a simplified means to calculate CLT wall lateral resistance, the load-slip resistance of inter-panel connectors between the wall and floor/ceiling is needed to generate wall responses. Such load-slip resistance curves can be obtained through connection tests or other means deemed acceptable for modeling purposes. In the development of this Chapter, the panel hardware connectors were back-calibrated from the FPInnovations CLT wall tests (Popovski, 2010).

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The connector hysteresis was assumed to follow the CUREE 10-parameter hysteretic model, which has been widely adopted for wood frame shear wall and wood connection modeling. The behavior of the model and each control parameter are shown in Figure 5.

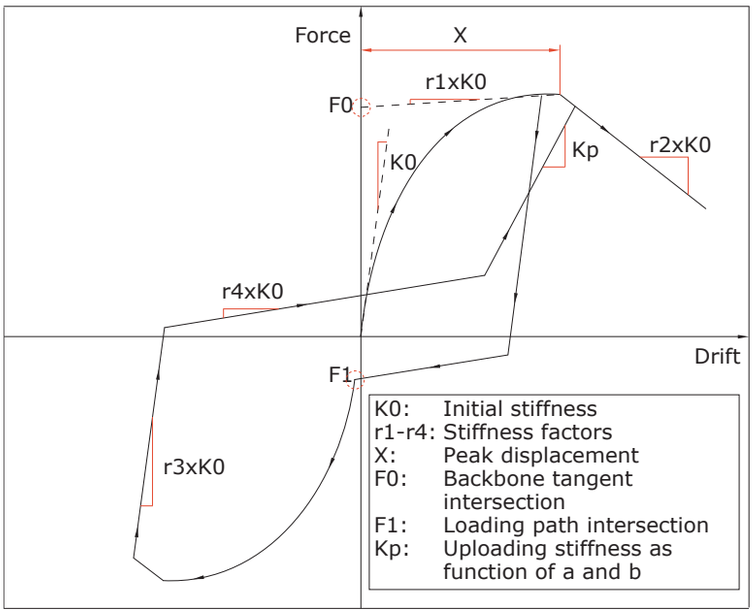

Figure 5
Hysteretic model for CLT connections

The reverse calibration procedure to find the connection parameters was conducted as follows:

(1) Numerically model the CLT experimental data for each wall with trial connection parameters.
(2) Subject the numerical wall models to the same displacement protocols used in the experimental tests.
(3) Compare the model backbone (monotonic) or hysteresis (cyclic) with experimental measurements.
(4) Adjust the connection parameters to improve model accuracy.
(5) Repeat Steps 2-4 until the model closely matched the observed experimental response.

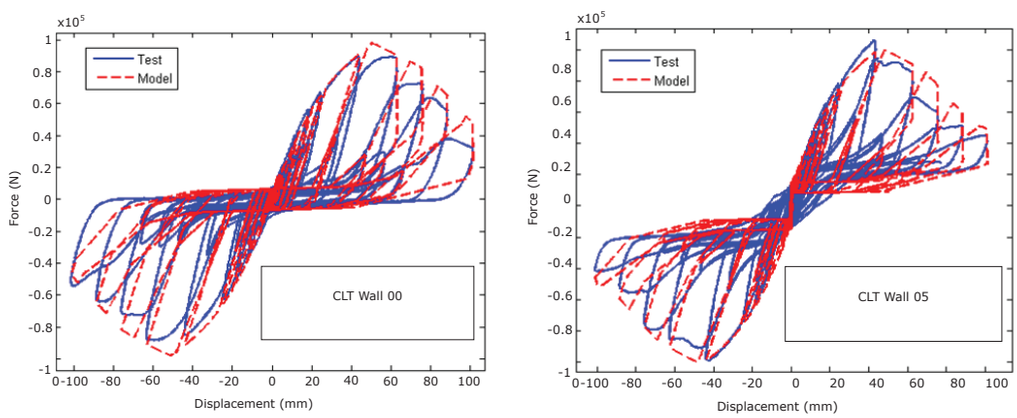

Figure 6
Calibrated wall models compared with tests (Popovski et al., 2010)

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Figure 6 shows examples of the calibrated model responses compared to test responses. The accuracy of the model was felt to be sufficient for design and analysis purposes. A group of connector parameters were obtained through this process and are listed in Table 1. These connector parameters can then be used to develop lateral responses for a given CLT wall design based on the kinematic assumptions stated earlier. However, it should be noted that all wall data available was for one aspect ratio as discussed in detail in Appendix A, thus the assumption of rotation in the kinematic model described herein must be kept in mind. Experimental dynamic responses of at least 3.5% have been observed to behave as described in this handbook.

Table 1
Calibrated connector parameters

Connector Type	Hysteretic Parameters (lb., in.)									
	K0 (lb./in.)	r1	r2	r3	r4	F0 (lb.)	F1 (lb.)	X (in.)	a	b
HTT 16	25000	0.002	-0.30	1	0.05	9000	400	2.00	0.75	1.10
16D-SN	800	0.005	-0.20	1	0.01	800	40	2.50	0.50	1.10
SFS1	800	0.005	-0.40	1	0.01	750	20	1.80	0.50	1.10
SFS2	1600	0.005	-0.40	1	0.01	1200	40	2.00	0.50	1.10
16D-SN with half-lapped joint	900	0.001	-0.30	1	0.03	600	20	1.60	0.50	1.10
SFS1 with half-lapped joint	900	0.001	-0.60	1	0.03	562.5	10	1.15	0.50	1.10
SFS2 with half-lapped joint	1800	0.001	-0.60	1	0.03	900	20	1.28	0.50	1.10

Connectors calibrated were based on FPInnovations' test results and include:

(1) HTT 16: Simpson Strong-Tie HTT 16 hold-downs installed at corners of CLT walls
(2) 16D-SN: 16d spiral nails with D=0.153 in. (3.9 mm) and L=3.5 in. (89 mm)
(3) SFS1: SFS Intec screws D=0.16 in. (4.0 mm) and L=2.75 in. (70 mm)
(4) SFS2: SFS Intec screws D=0.20 in. (5.0 mm) and L=3.54 in. (90 mm)

When a CLT wall is made up of multiple panels, the panels' vertical interface is typically connected with a half-lapped detail. The impact of this step-joint detail on wall lateral resistance was phenomenologically captured in this calibration process using the equivalent connector parameters when multi-panel wall test data (also shown in Table 1 as "connector with half-lapped joint") was used.

5.3 Typical CLT Wall Configurations

As shown in Figure 7, several typical CLT wall configurations were considered as an example. Each bracket shown in Figure 6 is installed with either six 16D spiral nails or four screws, which is the maximum number of connectors per bracket used in FPInnovations' shear wall tests.

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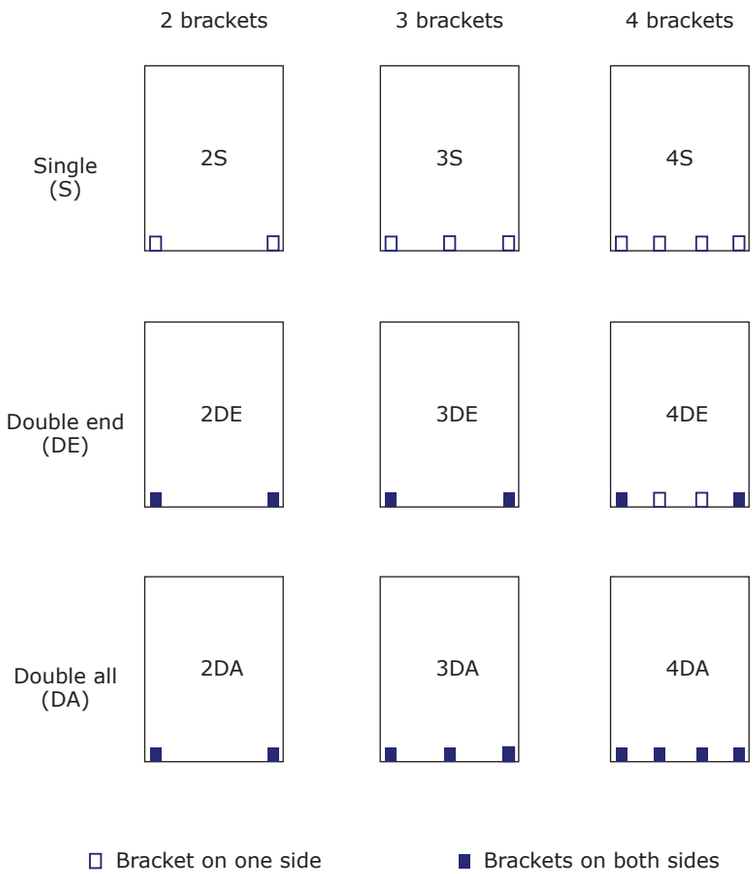

Figure 7
Typical CLT wall configurations (Note: a maximum of either six 16D spiral nails or four screws were used per bracket)

The height of all panels in the typical wall configuration is 8 feet. The length of a single panel can vary from 3 to 6 feet. For wall length equal to or greater than 8 feet, multiple 4-foot long panels are combined together such that the panels can rotate as discussed earlier. In Figure 7, the notation "S" stands for Single sided brackets for each location, "DE" stands for Double sided brackets at the End of the panel, and "DA" stands for Double All, meaning all brackets in the panels are double sided. It should be noted that, in the case of the wall panel with only 2 brackets, configurations DE and DA are identical.

For each wall configuration, the necessary lateral wall resistance parameters (or curves) depending on the requirements of the design application can be developed, including:

(1) Ultimate lateral strength of the wall for developing allowable load level for equivalent static force design (see example of allowable strength table developed later in this section and Appendix B for design example).
(2) Backbone curve of the wall for nonlinear push over analysis or displacement-based design (see Appendix B for a design example using Direct Displacement Design).
(3) Hysteretic response of the wall for more advanced nonlinear time history modeling and simulation (see Appendix B for an example using CLT wall hysteresis model in nonlinear time history analysis).

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5.4 Example CLT Allowable Capacity Table

The ultimate strength can be determined using the numerical model for each CLT wall configuration. Since, at this point, the design values for CLT walls are not defined in the United States, it was decided to proceed with the wall design resistances equal to the ultimate load divided by a factor of 2.5 for illustrative purposes; that is utilizing only 40% of the wall ultimate strength in the design example. From a drift perspective, this force demand occurs at 0.6% story drift compared to approximately 3.5% to 4% ultimate story drift for a 4-foot long panel of 8 ft. in height. In developing the model, it was assumed that there will always be hold-downs present at the ends of the wall (if a wall has multiple panels, only the two ends of wall will have hold-downs, not the ends of each panel). The resulting design resistance values (ASD and LRFD) for standard CLT wall configurations using three types of connectors are shown in Table 2.

Table 2

Example CLT wall design resistance[1] (kips)

Fastener Type: 16D-SN		Wall Length (ft.)							
		Single-panel Wall				Multi-panel Wall			
No. of Brackets[2]	Bracket Installation	3	4	5	6	2x4	3x4	4x4	5x4
2	S	2.6	3.5	4.4	5.3	4.5	5.8	7.1	8.4
	DE	3.9	5.2	6.5	7.8	7.1	9.7	12.4	15.0
	DA	3.9	5.2	6.5	7.8	7.1	9.7	12.4	15.0
3	S	3.1	4.1	5.1	6.2	5.5	7.3	9.1	10.9
	DE	5.7	7.6	9.5	11.5	9.9	13.1	16.2	19.4
	DA	4.8	6.4	8.0	9.6	9.1	12.8	16.4	20.1
4	S	3.6	4.8	6.0	7.2	6.5	8.9	11.2	13.6
	DE	6.2	8.3	10.3	12.5	11.0	14.7	18.3	22.0
	DA	5.8	7.7	9.6	11.6	11.2	15.9	20.7	25.4

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SFS1		Wall Length (ft.)							
		Single-panel Wall				Multi-panel Wall			
No. of Brackets[2]	Bracket Installation	3	4	5	6	2x4	3x4	4x4	5x4
2	S	2.1	2.9	3.6	4.3	3.3	4.1	4.8	5.6
	DE	2.9	3.9	4.9	5.9	4.8	6.4	7.9	9.4
	DA	2.9	3.9	4.9	5.9	4.8	6.4	7.9	9.4
3	S	2.4	3.2	4.0	4.9	3.8	4.9	5.9	6.9
	DE	4.6	6.1	7.6	9.2	7.1	8.9	10.7	12.5
	DA	3.5	4.6	5.8	7.0	5.9	8.0	10.1	12.1
4	S	2.7	3.6	4.5	5.5	4.4	5.8	7.1	8.4
	DE	4.8	6.5	8.1	9.8	7.7	9.8	11.9	14.0
	DA	4.1	5.4	6.8	8.2	7.1	9.8	12.4	15.1

Fastener Type: SFS2		Wall Length (ft.)							
		Single-panel Wall				Multi-panel Wall			
No. of Brackets[2]	Bracket Installation	3	4	5	6	2x4	3x4	4x4	5x4
2	S	2.7	3.6	4.6	5.5	4.5	5.8	7.2	8.5
	DE	4.1	5.4	6.8	8.2	7.2	9.8	12.5	15.2
	DA	4.1	5.4	6.8	8.2	7.2	9.8	12.5	15.2
3	S	3.3	4.3	5.4	6.6	5.5	7.4	9.2	11.1
	DE	6.0	8.0	10.0	12.0	10.0	13.2	16.4	19.6
	DA	5.1	6.9	8.6	10.3	9.2	13.0	16.7	20.4
4	S	3.8	5.1	6.4	7.7	6.6	9.0	11.4	13.8
	DE	6.5	8.7	10.9	13.2	11.1	14.8	18.6	22.3
	DA	6.2	8.3	10.4	12.6	11.4	16.2	21.1	25.9

[1] Tabulated design resistance is for ASD. For LRFD, multiply tabulated resistance by 1.6, which represents a soft calibration similar to AF&PA/ASCE 16 (1995).

[2] Brackets used were Simpson Strong-Tie ABR105; 4.125 in. x 4.125 in. x 3.5625 in.

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5.5 Model for System Level Performance Simulation

Shake table tests on complete CLT building systems have been limited to responses with moderate levels of story drift (e.g., 3.5%), therefore the Chapter discussion here is focused on maintaining story drifts at or below this level. Currently, system level behavior of CLT buildings under severely damaged or near the collapse stage has not been observed experimentally. The model proposed here for application to CLT systems was first introduced by Pei and van de Lindt (2009) to simulate shear-bending coupled response of stacked light-frame wood shear wall systems, and was incorporated in the software program SAPWood V2.0 for seismic analysis of mid-rise, light-frame wood buildings. The kinematic assumptions of the model are illustrated in Figure 8. A more detailed description of the model and use of the SAPWood program can be found in Pei and van de Lindt (2009) and in NEEShub (www.nees.org) where the software and users manual are available for free download.

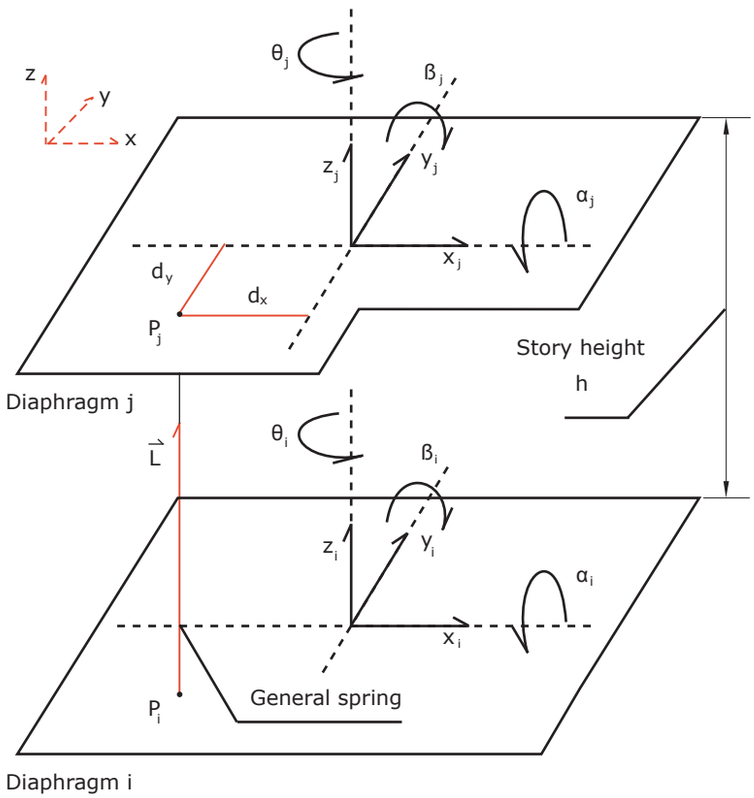

Figure 8
Simplified system level model for nonlinear time history analysis of CLT buildings

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Based on model assumptions, the dynamic response of a multi-story CLT building can be represented by multiple rigid diaphragm plates connected by general nonlinear springs. The shear resistance of CLT walls is represented by nonlinear hysteretic springs, while the components that resist uplift and overturning are modeled as vertical tie-down springs. The main assumptions for this model include:

(1) Dynamic response of CLT floor/roof diaphragms can be approximated as responses of rigid plates having 6 degrees of freedom.

(2) Lateral (shear) resistance of CLT walls can be represented in the model as hysteretic springs.

(3) Overturning restraint is provided and can be represented in the model as linear springs.

(4) Effect of finish materials on lateral resistance is not examined in this study.

Appendix B of this Chapter presents an example analysis that was conducted for a 6-story CLT building using SAPWood and subjected to a suite of earthquake ground motions to investigate its response. The building design was based on the ASCE7-10 equivalent lateral force procedure using assumed values of the response modification factor and wall resistance values in accordance with Table 2. The analysis in Appendix B does not include the effect of finishing materials or non-structural walls. CLT appears to be capable of being used for mid-rise building construction in seismic regions of the United States. To do so, the designer would need to justify the parameters used in design through the alternative means of the building code.

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6
SUMMARY AND CONCLUSIONS

In this Chapter, the state-of-the-art knowledge on cross-laminated timber in the United States was presented. Information available to the authors on testing and modeling around the world was summarized in appendices and an example using a six-story building floor plan, originally designed using light-frame wood, was re-designed using CLT, based on the assumption of several different R-factors. At this early stage of CLT development in the United States, a comprehensive assumption of seismic design coefficients including R cannot be made, but the following conclusion can be stated based on the single example in Appendix B. An R-factor of approximately 2 appears to provide less than a 10% probability of exceeding 3.5% story drift for the single six-story illustrative example presented herein. CLT is believed to be a viable option for mid- and high-rise buildings (e.g., 15 stories) based on past testing, observations, and numerical modeling. A P695 study is in progress to develop seismic design coefficients and facilitate use of the equivalent lateral force method for seismic design of CLT in the United States.

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APPENDIX A
LITERATURE REVIEW

A1 European and Japanese Experience

Appendix A summarizes notable research efforts led by European researchers on seismic performance of CLT building systems.

A1.1 Experimental Studies

There were several notable experimental studies on CLT systems seismic performance carried out by European researchers over the last decade, including CLT wall tests and shake table tests of complete buildings.

The most comprehensive study to determine the seismic behavior of 2-D CLT wall panels was conducted at the University of Ljubljana, Slovenia. During the project that was partially supported by KLH Massiveholz GmbH from Austria, numerous quasi-static monotonic and cyclic tests were carried out on walls with lengths of 8 ft. (2.44 m) and 10.5 ft. (3.2 m) and heights of 8 ft. (2.44 m) and 9 ft. (2.72 m) (Dujic et al., 2004). Walls were subjected to combined constant vertical load and either monotonic or cyclic horizontal loads. Wall panels were tested with various boundary conditions which enabled the development of load vs. wall deformation relations from cantilever to pure shear wall behavior. Influence of boundary conditions, magnitude of vertical load and types of anchoring systems were investigated (Dujic et al., 2005, 2006). Differences in mechanical properties between monotonic and cyclic responses were also studied (Dujic and Zarnic, 2006), as was the influence of openings on the shear wall properties (Dujic et al., 2006, 2007). Two configurations of walls with equal dimensions, one with no openings and one with a door and a window, were tested under the same boundary conditions. Analytical models of CLT wall panels were developed in the computer program SAP2000, and were verified against the test results. The verified analytical models were used for a parametric study that included 36 mathematical models having different patterns of openings (Dujic et al., 2008). Results of the parametric study were used to develop mathematical formulas describing the relationship between the shear strength and stiffness of CLT wall panels with and without openings. CLT wall tests were also carried out by Karlsruhe Institute of Technology in order to compare the performance of such modern system vs. the "traditional" timber frame construction (Schädle et al., 2010).

As part of the University of Ljubljana CLT project, shake table tests were conducted on two one-story box CLT models at the Dynamic Testing Laboratory of the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, Macedonia. The intent was to make a correlation between the results from the quasi-static tests and the results from the shake table tests. Based on these tests, the main characteristics of the dynamic response of the tested models were determined.

A more comprehensive study to quantify the seismic behavior of low- and mid-rise CLT constructions was part of the SOFIE project, in Italy. This project was undertaken by the Trees and Timber Institute of the National Research Council of Italy (CNR-IVALSA) in collaboration with National Institute for Earth Science and Disaster Prevention in Japan (NIED), Shizuoka University, and the Building Research Institute (BRI) in Japan,

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and partially supported by the Autonomous Province of Trento (Italy). The testing program included tests on connections, in-plane cyclic tests on CLT wall panels with different layouts of connections and openings (Ceccotti et al., 2006), pseudo-dynamic tests on a one-story 3-D specimen in three different layouts (Lauriola and Sandhaas, 2006), shake table tests on a three-story, 23 ft. x 23 ft. (7 m x 7 m) in plan and 33 ft. (10 m) high building under different earthquakes (Ceccotti and Follesa, 2006), and finally a series of full-scale shake table tests on a seven-story CLT building conducted at E-Defense facility in Miki, Japan (The SOFIE Project, 2012). For additional details on the SOFIE Project, see Ceccotti et al., 2010.

Results from quasi-static tests on CLT wall panels showed that the connection layout and design has a strong influence on the overall behavior of the wall (Ceccotti et al., 2006). Hysteresis loops were, on average, found to have an equivalent viscous damping of 12%. Similarly to the cyclic tests, the pseudo-dynamic tests showed that the construction system is very stiff but still ductile (Lauriola and Sandhaas, 2006). It was found that the initial stiffness of the 3-D specimen with asymmetric configuration (openings of 13 ft. (4.0 m) on one side and 7.4 ft. (2.25 m) on the other) was similar to that of the symmetric configuration (openings of 7.4 ft. (2.25 m) on both sides), suggesting that the larger opening on one side did not affect the building stiffness very much. It thus confirmed that the behavior of the wall is due to the connections and not to the CLT panels for lower levels of lateral force. This again may bring to light the need for the panels to be able to rotate as discussed earlier.

Figure 9
Three-story CLT house tested at NIED Laboratory in Tsukuba, Japan

Shake table tests on the 3-story house conducted in the laboratories of the NIED in Tsukuba, Japan (Figure 9) showed that the CLT construction survived 15 destructive earthquakes without any severe damage (Ceccotti and Follesa, 2006). The collapse state definition for the tests was defined to be failure of one or more hold-down anchors, which was reached only during the last test that used the Nocera Umbra earthquake record with peak ground acceleration (PGA) of 1.2 g. An analytical model of the 3-story house was developed using the DRAIN 3-DX computer program. The model was compared to the behavior of the 3-story house during the shake table tests, and showed good correlation with the test results.

The next series of shaking table tests from the SOFIE project was conducted in October 2007 at the Hyogo Earthquake Engineering Research Center in Miki, Japan. The building had a floor plan of 44 ft. x 25 ft. (13.5 m x 7.5 m), and was seven stories high with a total height of 77 ft. (23.5 m) (Figure 10). The building was designed by the CNR-IVALSA team according to the European Code for Construction in Seismic Regions with an action reduction factor of 3 (Ceccotti, 2008; Pozza et al., 2009) and the action increasing factor of 1.5 relevant to strategic buildings. The building walls were made of CLT panels with a thickness of 5.5 in. (142 mm)

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on the first two floors, 5 in. (125 mm) on floors three and four, and 3.3 in. (85 mm) on the last three floors, where less loads were expected. The walls were connected to each other using self-drilling (tapping) screws. Each wall consisted of several 8.1 ft. (2.5 m) long panels connected together with screws. The floors were also made with CLT panels with a thickness of 5.5 in. (142 mm), and were connected to the walls with steel brackets and screws. High capacity through-floor panel-to-panel tie-downs were used extensively to provide overturning resistance. The testing consisted of several consecutive applications in all three orthogonal directions of two earthquake ground motions, including the record from the Great Hanshin-Awaji Earthquake from 1995, also known as the Kobe Earthquake (M=7.2) with 100% intensity (0.6 g acceleration in shorter X-direction, 0.82 g in longitudinal Y-direction, and 0.34 g in vertical Z-direction). The maximum story drift was 1.5 in. (38 mm) (1.3% story drift) in the Y-direction and 1.14 in. (29 mm) (1% story drift) in the X-direction, with the total deflection at the top of the building being 6.9 in. (175 mm) and 11.3 in. (287 mm), respectively, with the building fully recentering following the series of earthquakes, i.e. returning to zero.

Figure 10
Seven-story CLT house tested at E-Defense Laboratory in Miki, Japan

A2 Canadian Experience

A2.1 Experimental Study

Following the introduction of CLT to Canada, several experimental research projects were carried out by FPInnovations to study the seismic performance of CLT wall and building systems. This section presents results from a CLT wall test program performed in Canada. System level tests were also carried out recently but the results were not available at the time this Handbook was published.

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A2.2 CLT Wall Tests by FPInnovations

In the testing program at FPInnovations in Vancouver, a total of 32 monotonic and cyclic tests were performed. All walls were 3-ply CLT panels with a thickness of 3.7 in. (94 mm). They were made of European spruce and manufactured at KLH Massiveholz GmbH in Austria. Since the CLT panels had to be shipped in a container, the panel dimensions were limited to 7.5 ft. (2.3 m), which was the height and width of the container. CLT walls with 12 different configurations were tested. Details about the testing matrix and the different Wall Configurations I to XII are given in Tables 3, 4 and 5. In Table 3, walls with aspect ratio of 1:1 are shown, while in Table 4 walls with aspect ratio of 1:1.5 are shown. In Table 5, two-story assemblies of 7.5 ft. x 7.5 ft. (2.3 m x 2.3 m) walls are presented along with tall CLT walls that had a height of 16.1 ft. (4.9 m) and a length of 7.5 ft. (2.3 m) (aspect ratio of 2.1:1). All wall specimens were assembled using hardware and dowel connectors shown in Figures 11 and 12. Four different types of brackets (A, B, C, and D) were used to connect the walls to the steel foundation beam or to the CLT floor panel below (Figure 11). Bracket A, Simpson Strong-Tie AE116, 3.5 in. x 1.9 in. x 4.6 in. (90 mm x 48 mm x 116 mm) (W x D x H), and Bracket B, Simpson Strong Tie ABR105, 3.5 in. x 4.1 in. x 4.1 in. (90 mm x 105 mm x 105 mm), are off-the-shelf products that are commonly used in CLT applications in Europe. Brackets C and D were custom made out of 0.25 in. (6.4 mm) thick steel plates to accommodate the use of timber rivets.

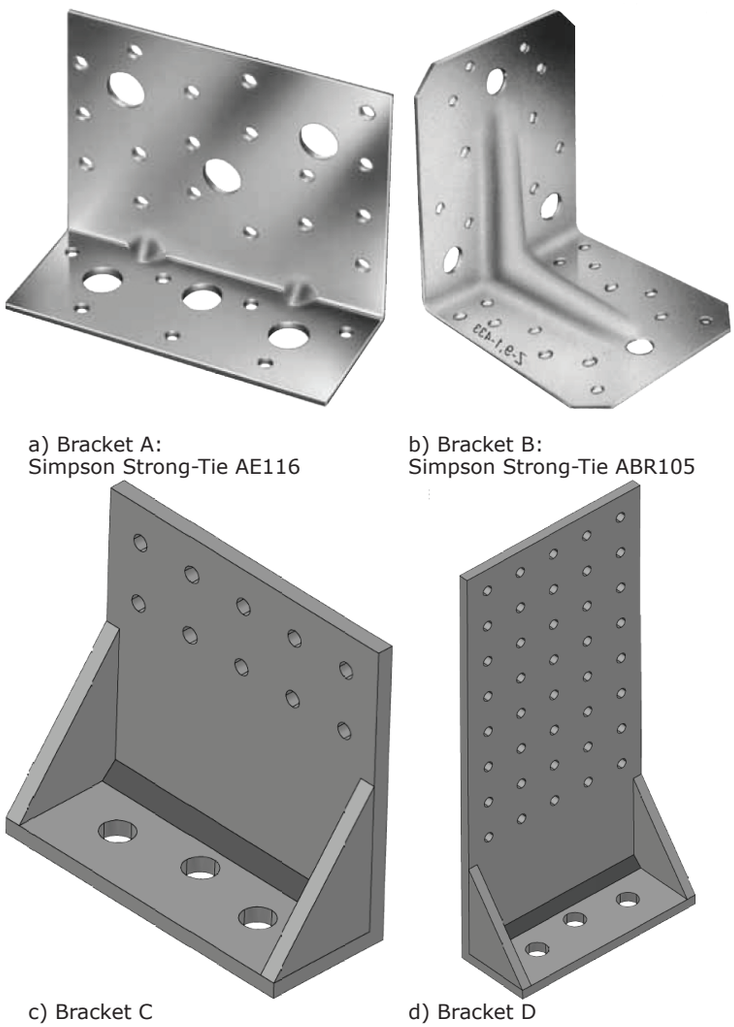

Figure 11
Brackets for CLT walls used in the tests

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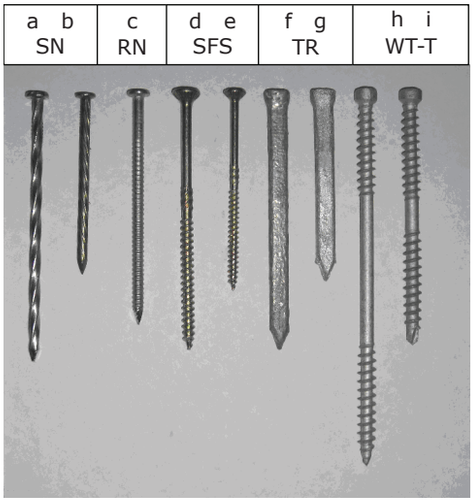

Figure 12
Fasteners used in the test program

(a) 16d spiral nail D=0.153 in. (3.9 mm) and L=3.5 in. (89 mm)
(b) 10d spiral nail D=0.13 in. (3.3 mm) and L= 2.48 in. (63 mm)
(c) Annular ring nail D=0.134 in. (3.4 mm) and L=3 in. (76 mm)
(d) SFS2 screw D=0.20 in. (5.0 mm) and L=3.5 in. (90 mm)
(e) SFS1 screw D=0.16 in. (4.0 mm) and L=2.7 in. (70 mm)
(f) Timber rivet L=3.5 in. (90 mm)
(g) Timber rivet L=2.56 in. (65 mm)
(h) WT-T screw 0.256 in. x 5.12 in. (6.5 x 130 mm)
(i) WT-T screw D=0.15 in. (3.8 mm) and L=3.5 in. (89 mm)

Walls in Configuration I had four brackets spaced at 28 in. (710 mm) o.c. Walls 00 through 03 used Type A brackets, which were connected to the wall using eighteen 16d spiral nails (SN) with D=0.153 in. (3.9 mm) and L=3.5 in. (89 mm) (Figure 12a). Wall 04 used Type A brackets and twelve annular ring nails (RN) with D=0.134 in. (3.4 mm) and L=3 in. (76 mm) (Figure 12c). Wall 05 used Type A bracket and eighteen SFS1 screws with D=0.16 in. (4.0 mm) and L=2.7 in. (70 mm) (Figure 12e), while Wall 06 used ten SFS2 screws with D=0.20 in. (5.0 mm) and L=3.5 in. (90 mm) (Figure 12d). Walls 09 and 10A used Type C brackets with two rows of five L=2.56 in. (65 mm) timber rivets (Figure 12g). In addition to three Type A brackets spaced at 21.3 in. (550 mm) o.c. nailed with eighteen spiral nails (D =0.153 in.; L = 3.5 in.), Walls 07, 08 and 08A of Configuration II had Simpson Strong Tie HTT-16 hold-downs at both ends. The hold-downs were nailed using eighteen 16d spiral nails for Walls 07 and 08, while Wall 08A used eighteen spiral nails with D=0.13 in. (3.3 mm) and L= 2.48 in. (63 mm) (Figure 12b).

Walls from Configuration III (11, 12 and 12A) consisted of two panels that were connected to the foundation in the same way as walls from Configuration I. The two panels that formed the wall were connected together using a continuous 2.56 in. (65 mm) long half-lapped joint with no gap, and one vertical row of screws. Twelve SFS WT-T type screws with D=0.15 in. (3.8 mm) and L=3.5 in. (89 mm), spaced at 7.9 in. (200 mm), were used in Wall 11 (Figure 12i) to connect panels together, while panels in Walls 12 and 12A were connected to each other using SFS2 screws (Figure 12d). These walls were designed to investigate the effect of gaps in the walls on the overall wall performance under lateral loads.

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Only one CLT panel (Wall 20) was tested from Configuration IV. In addition to four Type A brackets on the front side, this wall had three additional brackets on the back side, spaced right in the middle between the front brackets, for a total of seven brackets. This configuration was representative of an inside wall where both sides of the wall are available for connecting.

Table 3

Test matrix for 7.5 feet (2.3 m) long and 7.5 feet (2.3 m) high walls

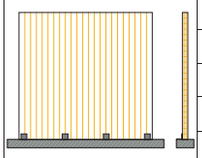
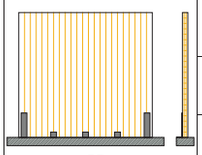
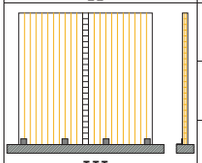
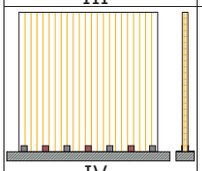
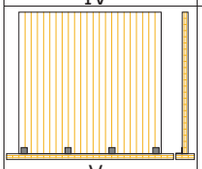
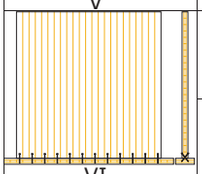
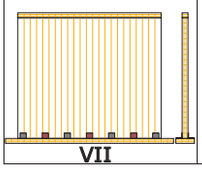
Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kip/ft.]	Test Protocol[1]
 I	00	Bracket A SN 16d, n=18	0	CUREE
	01		0.68	Monotonic
	02		0.68	CUREE
	03		1.37	CUREE
	04	Bracket A, RN, n=12	1.37	CUREE
	05	Bracket A, SFS1, n=18	1.37	CUREE
	06	Bracket A, SFS2, n=10	1.37	CUREE
	09	Bracket C, Rivets L=2.5 in., n=10	1.37	Monotonic
	10A	Bracket C, Rivets L=2.5 in., n=10	1.37	CUREE
 II	07	Bracket A, SN 16d, n=18	1.37	Monotonic
	08	Bracket A, SN 16d, n=18	1.37	CUREE
	08A	Bracket A, SN 16d, n=18 10d, n=18 on Hold-downs	1.37	CUREE
 III	11	Bracket A, SN 16d, n=18 Between panels WT-T, n=12	1.37	CUREE
	12	Bracket A, SN 16d, n=18 Between panels SFS2, n=12	1.37	CUREE
	12A	Bracket A, SN 16d, n=18 Between panels SFS2 , n=12	1.37	ISO
 IV	20	Bracket A SN 16d, n=18 3 brackets on the back side	1.37	CUREE
 V	21	Bracket A SN 16d, n=6	1.37	CUREE
 VI	22	WT-T, n=18	1.37	CUREE
	22B	WT-T, n=34	1.37	CUREE
 VII	23	Bracket A SN 16d, n=6 3 brackets on the back side	1.37	CUREE

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Table 4

Test matrix for 11.3 feet (3.45 m) long and 7.5 feet (2.3 m) high walls

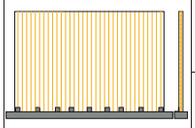
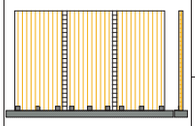
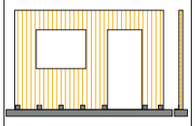
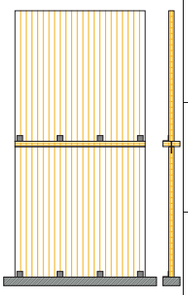
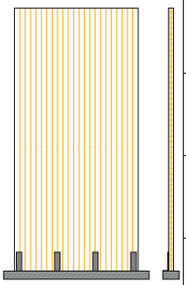
Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kip/ft.]	Test Protocol[1]
 VIII	13	Bracket B (9 brackets) SN 16d, n=10	1.37	Monotonic
	14	Bracket B (9 brackets) SN 16d, n=10	1.37	ISO
 IX	15	Bracket B (9 brackets) SN 16d, n=10 SFS2, n=8	1.37	Monotonic
	16	Bracket B (9 brackets) SN 16d, n=10 SFS2 , n=8	1.37	ISO
 X	19	Bracket B SN 16d, n=10	1.37	ISO

Table 5

Test matrix for two story assemblies and tall walls

Wall Configuration	Test Designation	Brackets and Fasteners	Vertical Load [kip/ft.]	Test Protocol[1]
 XI	28	Bracket A, SN 16d, n=6 Slab-to-wall screws at 7.9 in	1.37	Monotonic
	29B	Bracket A, SN 16d, n=6 on both floors	1.37	ISO
	29C	Bracket A, SN 16d, n=8 at the bottom n=6 on the top storey	1.37	ISO
 XII	24	Bracket D, Rivets L=2.56 in., n=40	1.37	Monotonic
	25	Bracket D, Rivets L=2.56 in., n=40	1.37	ISO
	26	Bracket D, Rivets L=3.54 in., n=40	1.37	ISO
	27	Bracket D, Rivets L=3.54 in., n=20	1.37	ISO

[1]It is noted that test protocol does make a difference in the experimentally measured hysteresis. For more information on these protocols, the interested reader is referred to Krawinkler et al. (2000) and ASTM International (2009).

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To investigate the effect of the foundation stiffness in a real case scenario, walls in Configurations V, VI and VII were placed over a 3.7 in. (94 mm) thick CLT slab with a width of 15.75 in. (400 mm). Wall 21 used four Type A brackets spaced at 28 in. (710 mm) o.c., while Wall 23 used a total of seven brackets (four in front and three on the back) in the same arrangement as in Wall 20. Each of the brackets had six 16d spiral nails. The brackets were connected to the CLT floor slab using three screws with D=0.39 in. (10 mm) and L=3.15 in. (80 mm). Wall 22 used nine pairs of WT-T 0.256 in. x 5.12 in. (6.5 x 130 mm) screws (Figure 12h) placed at an angle of 45 degrees to the slab and spaced at 11 in. (280 mm). Wall 22B used seventeen pairs of the same screws with five pairs being closely grouped near each end of the wall (spaced at 1.57 in. (40 mm)) to simulate a hold-down effect. The rest of the screws were spaced at 12.6 in. (320 mm).

Walls from Configurations VIII, IX, and X were 11.3 ft. (3.45 m) long and 7.5 ft. (2.3 m) high. Walls 13 and 14 (Configuration VIII) were single panel walls that had a total of nine Type B brackets, each with ten 16d spiral nails. Brackets had different spacing, varying from 12.6 in. (320 mm) to 18.1 in. (460 mm). Walls 15 and 16 (Configuration IX) were three-panel walls, with the same number and position of the brackets as the walls of Configuration VIII. The panels were connected to each other using half-lapped joints and fasteners. Walls 15 and 16 used eight SFS2 screws of 0.197 in. x 3.54 in. (5x90 mm) spaced at 11.8 in. (300 mm). Wall 19 of Configuration X was the only wall with openings in the entire research program. The door was 6.2 ft. (1.9 m) high and 2.6 ft. (0.8 m) wide, with the door post being 19.7 in. (500 mm) wide, while the window was 3.8 ft. (1.15 m) wide and 2.6 ft. (0.8 m) high. The wall was connected using seven Type B brackets, each using ten 16d spiral nails.

Configuration XI included three two-story wall assemblies consisting of a lower and upper story wall (7.5 ft. x 7.5 ft. (2.3 m x 2.3 m)) with a 3.7 in. (94 mm) CLT slab in between. Both walls were connected at the bottom using Type A brackets, spaced at 28 in. (710 mm) o.c. Walls 28 and 29B used six 16d spiral nails, while wall 29C used eight such nails. The floor panel was connected to the bottom wall using SFS screws with D=0.315 in. (8 mm) and L=7.9 in. (200 mm), spaced at 7.9 in. (200 mm). Finally, Configuration XII consisted of four single panel tall walls (7.5 ft. x16 ft. (2.3 m x 4.9 m)) that were connected to the steel foundation beam using four Type D brackets spaced at 28 in. (710 mm). Walls 24 and 25 had forty rivets in each bracket (L=2.56 in. (65 mm)), Wall 26 had the same number of 3.5 in (90 mm) long rivets (Figure 12f), while Wall 27 had twenty L=3.5 in. (90 mm) rivets.

A2.3 Test Setup

A solid model of the test setup with a specimen ready for testing is shown in Figure 13. Steel "I" beams with stiffeners provided a foundation to which the specimens were bolted down. Another stiff steel beam that was bolted to the top of CLT walls was used as a spreader bar for the lateral load. The exact influence of the spreader bar is not known for multi panel wall configurations, but it is noted that a gap was provided between the spreader bar and top of the panels. Lateral guides with rollers were also used to ensure a steady and consistent unidirectional movement of the walls. Vertical load was applied using a 3 kip (13.3 kN) hydraulic actuator located in the middle of each side of the wall when testing 7.5 ft. (2.3 m) long walls (Figure 14), or using two such actuators located at third points on each side of the wall for 11.3 ft. (3.45 m) long walls. It is recognized that this would behave slightly different than gravity load, but allows for the restoring force to be determined in order to accurately fit hysteretic models for analyses. Only Wall 00 was tested without any vertical load on it. Walls 01 and 02 were tested with a 685 lb./ft. (10 kN/m) vertical load, which approximately corresponds to a wall located at the bottom of a two-story structure. All other walls were tested using a 1370 lb./ft. (20 kN/m) vertical load, which corresponds to a wall being at the bottom of a four-story structure.

The walls were subjected to either monotonic or cyclic lateral loading using a 24.7 kips (110 kN) hydraulic actuator (Figure 13). Walls 01, 07, 09, 13 and 15 were tested under monotonic (ramp) loads with a displacement rate of 0.008 in./s (0.2 mm/s), while Walls 24 and 28 were tested with a rate of 0.016 in./s (0.4 mm/s). All other walls, as shown in Tables 3, 4 and 5, were tested either using CUREE (Method C) or ISO 16670 cyclic testing protocols (Method B), as specified in ASTM E 2126 (ASTM International, 2009), with a rate of 0.196 in./s (5 mm/s). Instrumentation included lateral displacement at the top and bottom of the wall, uplift at both ends, as well as deformation of the wall along the wall diagonals.

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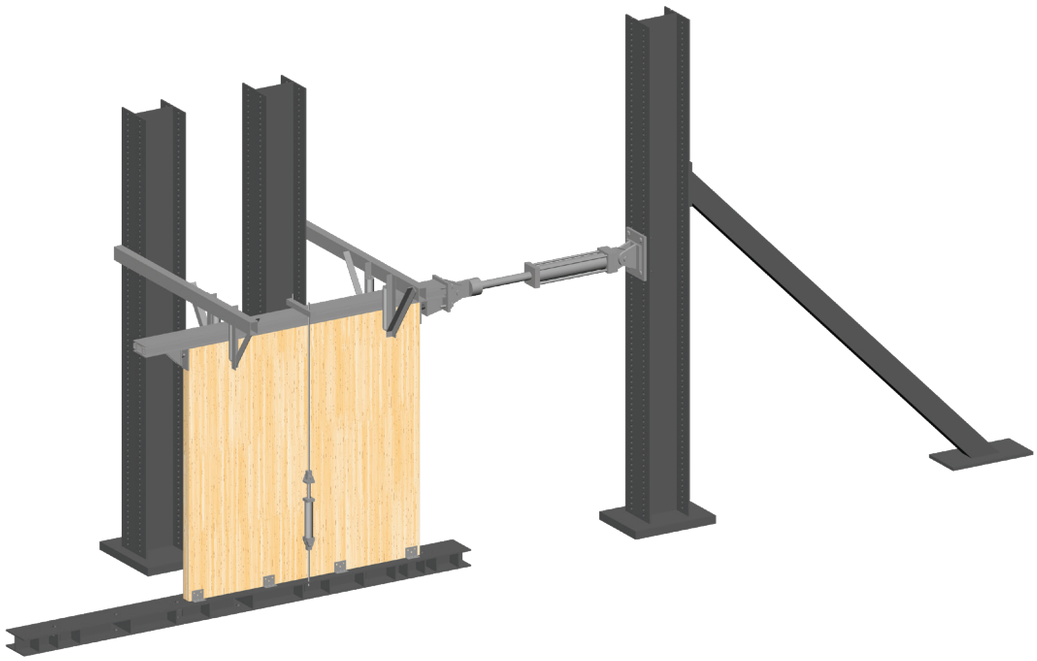

Figure 13
Test setup used for CLT walls

A2.4 Test Observations

As expected, the CLT wall panels behaved almost as rigid bodies during the testing. Although slight shear deformations in the panels were measured, most of the panel deflections occurred as a result of the deformation in the joints connecting the walls to the foundation. In case of multi-panel walls, deformations in the half-lapped joints also had significant contribution to the overall wall deflection. Selected average properties of the CLT walls, based on the envelope curves of both sides of the hysteretic loops obtained from the tests, are given in Table 6. Analysis of the test data was conducted using the procedure specified in ASTM Standard E 2126 (ASTM International, 2009). After determining the envelope curves for the cyclic tests, the Equivalent Energy Elastic Plastic (EEEP) curves were defined and main properties based on these curves were determined. In Table 6, K_y is the initial stiffness, Δ_y the yield displacement, F_{max} the maximum load, Δ_{Fmax} the displacement at maximum load, and Δ_u the ultimate displacement. It should be noted that most findings presented here are based on a single wall test for any different wall arrangement.

Wall 00 with no vertical load had a maximum lateral resistance of 20 kips (88.9 kN) (see Figure 14), while Wall 02 with a 685 lb./ft. (10 kN/m) vertical load had a lateral resistance of 20.3 kips (90.3 kN) (see Figure 16). When the vertical load was increased to 1370 lb./ft. (20 kN/m) (Wall 03), the lateral resistance increased to 22 kips (98.1 kN), an increase of 10% (Figure 15). It seems that the axial load had to be at least 1370 lb./ft. (20 kN/m) or higher to have any significant influence on the lateral load resistance. The amount of vertical load, however, had a higher influence on the wall stiffness. The stiffness of Wall 03 was 28% higher than that of Wall 00. In addition, higher values of vertical load had influence on the shape of the hysteresis loop near the origin. It should be noted that on a system (building) level, the vertical load has relatively significant influence on the seismic performance of CLT buildings, especially at higher deformation levels, when CLT panels basically turn into rocking structural elements.

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Table 6

Average wall properties obtained from tests

Wall	K_y [kip/in./ft.]	Δ_y [in.]	F_{max} [kip]	Δ_{Fmax} [in.]	Δ_u [in.]	Δ_u [% drift]	Ductility [Δ_u / Δ_y]	Energy Dissipation [kip-ft.]‡‡
00	7.3	0.81	20.0	1.77	2.62	2.9	3.5	21.6
01*	8.9	0.72	24.4	1.88	2.16	2.4	3.0	-
02	7.7	0.72	20.3	1.64	2.81	3.1	3.9	24.9
03	9.3	0.67	22.1	1.74	2.50	2.8	3.9	26.5
04	9.6	0.67	23.0	1.54	2.35	2.6	3.6	20.6
05	9.2	0.69	23.1	1.39	2.11	2.3	3.1	23.6
06	8.7	0.69	22.5	1.66	1.97	2.2	2.9	22.3
09*	22.1	0.26	21.0	1.15	1.88	2.1	7.3	-
10A	20.5	0.31	23.0	1.30	1.93	2.1	6.3	25.5
07*	8.9	0.84	27.2	2.07	3.46	3.8	4.1	-
08**	15.5	0.46	26.6	2.09	2.49	2.8	5.4	29.3
08A	16.8	0.45	24.1	1.64	2.27	2.5	5.8	26.6
11	6.3	0.76	17.9	2.13	2.37	2.6	3.2	18.7
12	7.3	0.78	20.8	2.08	2.83	3.1	3.7	25.3
12A	7.4	0.76	20.4	1.61	2.23	2.5	3.0	32.5
20	12.5	0.79	34.2	1.61	2.78	3.1	3.6	42.6
21	9.4	0.34	12.2	1.71	3.34	3.7	9.8	14.7
22	15.6	0.19	9.3	0.48	1.24	1.4	7.5	6.0
22B	15.3	0.20	11.4	0.46	0.98	1.1	5.0	8.7
23	8.7	0.53	16.2	1.83	3.14	3.5	6.1	19.0
13*	31.1	0.40	45.2	1.57	2.77	3.1	6.9	-
14	33.5	0.37	42.9	1.41	2.67	2.9	7.4	67.8
15*	15.4	0.64	35.1	3.42	3.52	3.9	5.5	-
16	11.1	0.72	29.3	3.06	4.22	4.7	5.9	49.2
19‡	15.6	0.56	31.7	1.32	2.54	2.8	4.5	45.7
28*	1.6	1.09	6.5	3.52	6.93	3.8	6.3	-
29B‡	1.9	0.95	6.6	2.91	4.62	2.5	4.9	4.8
29C	2.1	0.92	7.1	2.93	5.60	3.0	6.1	6.0
24*	3.4	1.86	22.8	2.71	2.78	1.5	1.5	-
25	3.8	1.20	16.9	1.75	3.10	1.7	2.8	31.0
26	4.0	1.43	20.9	2.35	2.95	1.6	2.0	26.2
27	3.5	1.70	20.8	3.94	4.48	2.4	2.7	36.1

* Value from a single monotonic test; ** Hold-down fatigue failure observed; ‡ One of the values in the loop for F_u was at 90% of F_{max}; ‡‡ Energy dissipated until the end of the test.

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The maximum loads obtained from the monotonic tests were greater than the corresponding values obtained from the cyclic tests for each of the two cyclic protocols, while the ultimate deformations and loads at these deformation levels were slightly underestimated. A designer may wish to consider the monotonic behavior if designing a building in a near fault location. An example of the aforementioned wall behavior is given in Figure 16. It was also observed that, during the static tests, more deformation demand was induced on the brackets themselves than on the fasteners used to connect them. It is therefore suggested that cyclic tests be used for determining the properties of CLT wall panels under seismic loads.

Wall 04, with twelve annular ring nails per bracket, exhibited slightly higher resistance than Wall 03 with eighteen 16d spiral nails per bracket. This was mainly due to the higher withdrawal resistance of the annular ring nails. The ductility of Wall 04, however, was slightly lower than that of Wall 03 (Figure 17). The failure mode observed at the brackets of Wall 04 was also slightly different than that of Wall 03. While spiral nails in the brackets exhibited mostly bearing failure combined with nail deformation and withdrawal, annular ring nails in withdrawal had a tendency to pull out small chunks of wood along the way, as shown in Figure 18.

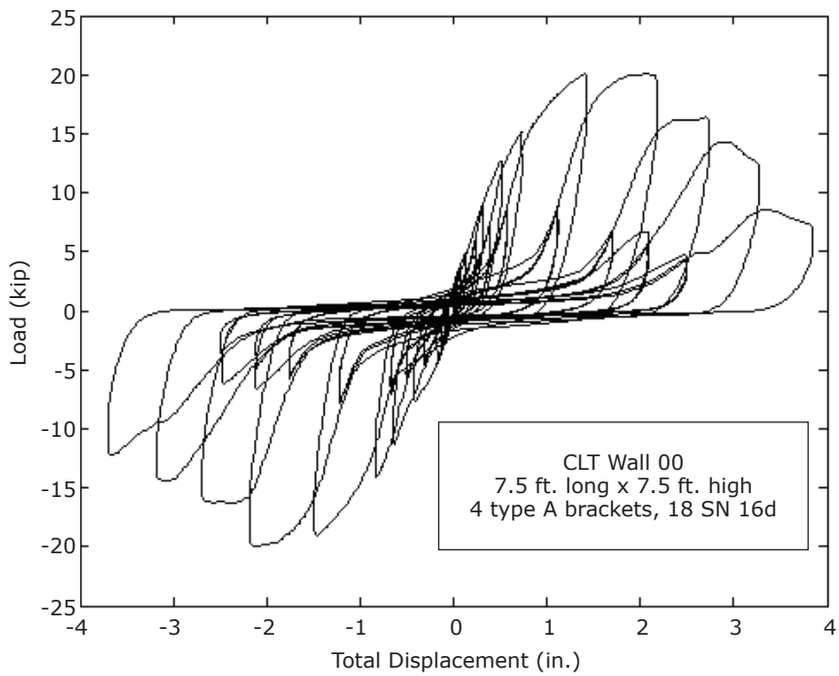

Figure 14
Hysteretic behavior for Wall 00 with no vertical load

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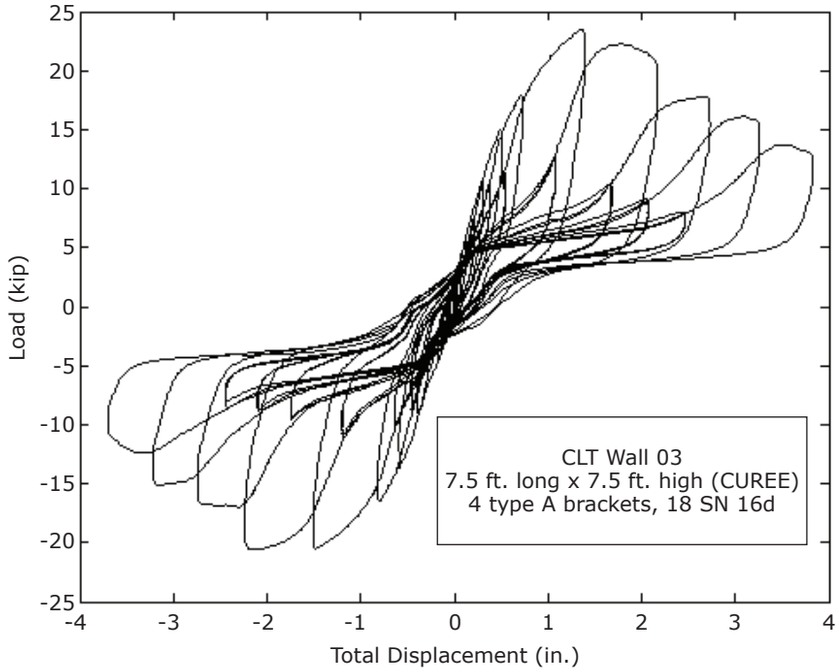

Figure 15
Hysteretic behavior for Wall 03 with 1370 lb./ft. (20 kN/m) vertical load

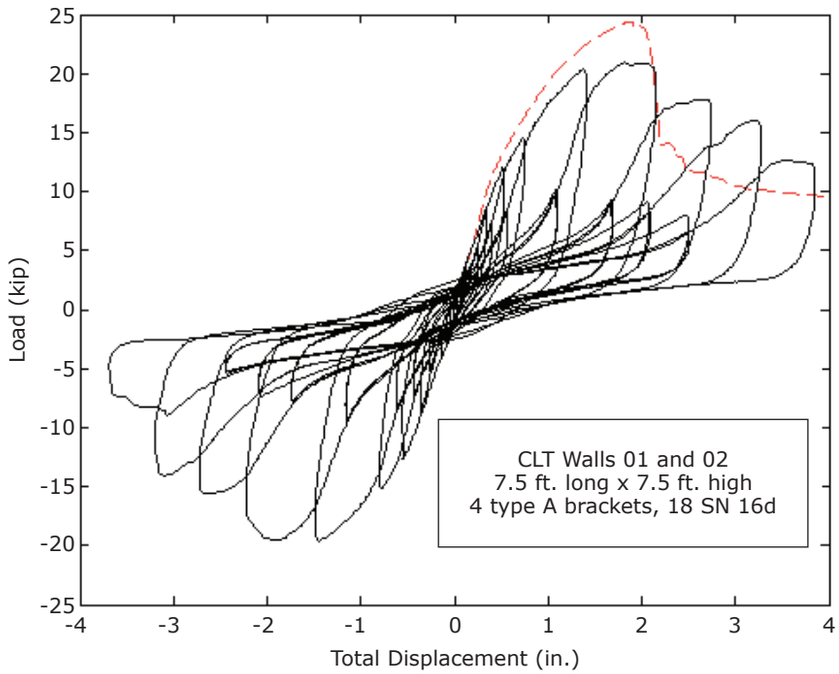

Figure 16
Results from monotonic (dashed line) and cyclic (solid line) tests on CLT walls with Configuration I

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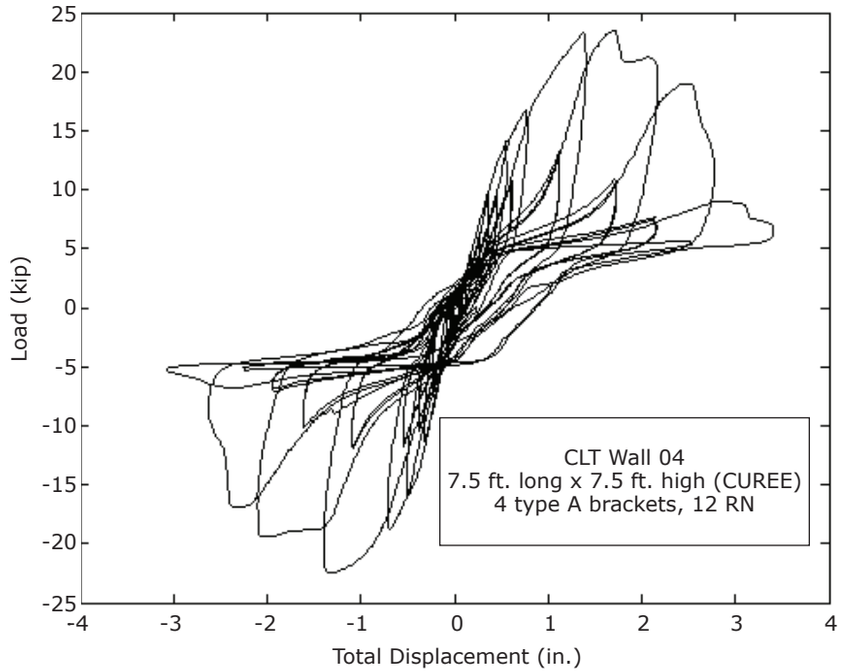

Figure 17
Hysteretic behavior for Wall 04 with annular ring nails

a) b)

Figure 18
Failure modes of the bracket connections at late stages of testing
for: a) Wall 02 with spiral nails; and b) Wall 04 with annular ring nails

The walls with screws (Walls 05 and 06) reached similar maximum loads as the walls with nails. The load carrying capacity for CLT walls with screws (Figures 19 and 20), however, dropped a little bit faster at higher deformation levels than in the case of walls with nails.

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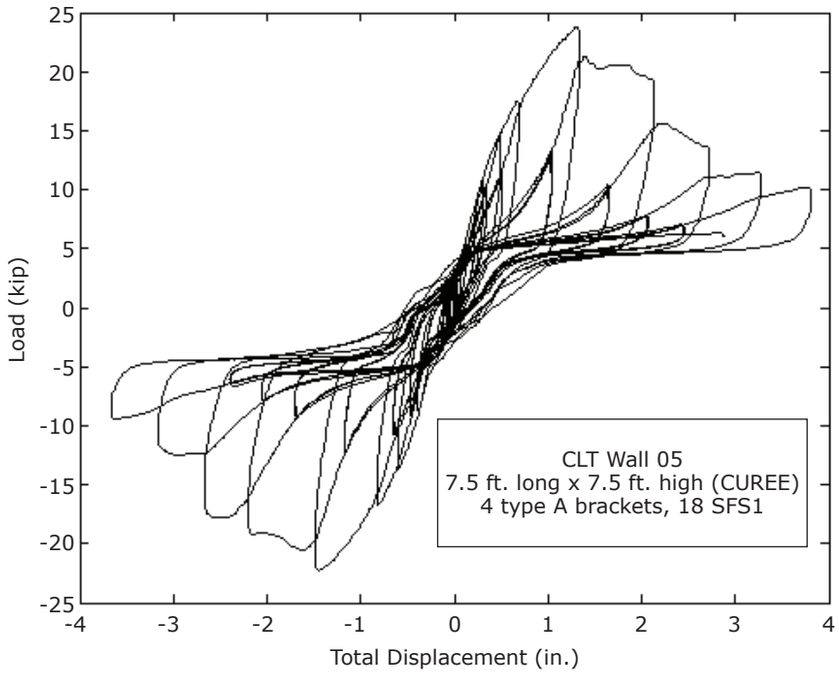

Figure 19
Hysteretic behavior for Wall 05 using 18 SFS1 screws with D=0.16 in. (4.0 mm) and L=2.7 in. (70 mm)

The CLT wall panel with hold-downs (Wall 08A) showed one of the highest stiffness for a wall with a length of 7.5 ft. (2.3 m), its stiffness being 81% higher than Wall 03 with 18 spiral nails per bracket. This CLT wall also showed relatively high ductility capacity (Figure 21). The behavior of one corner of Wall 08A during testing is shown in Figure 22.

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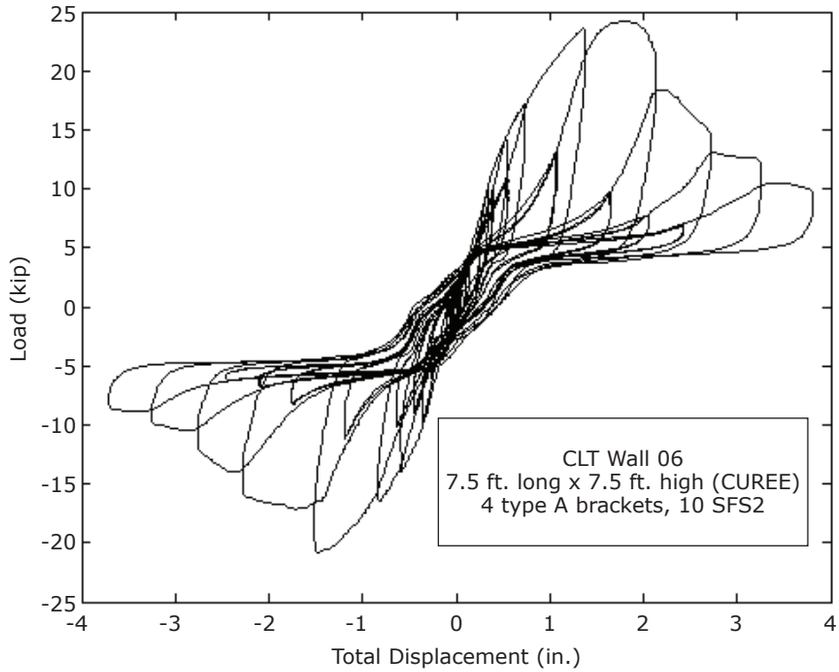

Figure 20
Hysteretic behavior for Wall 06 using 10 SFS2 screws with D=0.20 in. (5.0 mm) and L=3.5 in. (90 mm)

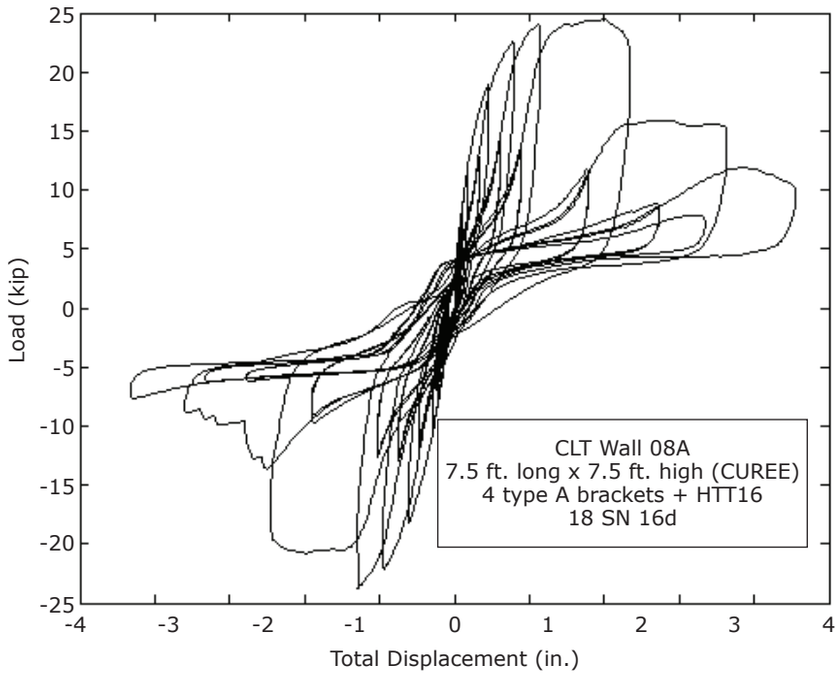

Figure 21
Hysteretic behavior for Wall 08A using brackets and hold-downs

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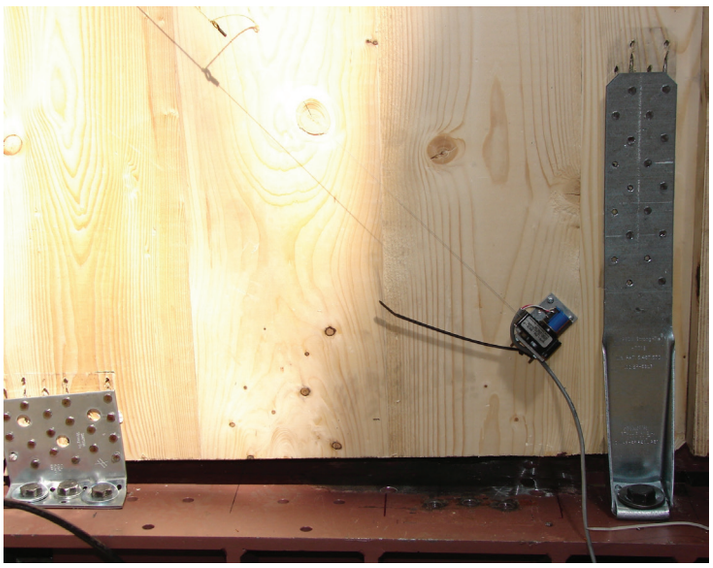

Figure 22
Behavior of one corner of Wall 08A during testing

Although timber rivets were developed to be used with glulam, they have recently been used in many other engineered wood products that have strands or veneers aligned in one direction. During this research program, an attempt was made to use rivets for the first time in CLT, beside the fact that when driven with their flat side along the grains in the outer layers, the rivets will be oriented across the grain in the middle layer. CLT Wall 10A with ten rivets per bracket exhibited a higher stiffness than any other walls tested in Configuration I and had the second highest capacity for walls having a length of 7.5 ft. (2.3 m), with its stiffness being 220% higher than that of Wall 03 with 18 spiral nails per bracket. Rivets were also able to carry more load per single fastener than any other fastener used in the program. In addition, the wall was able to attain relatively high ductility level. The hysteresis loop for Wall 10A with timber rivets is shown in Figure 23.

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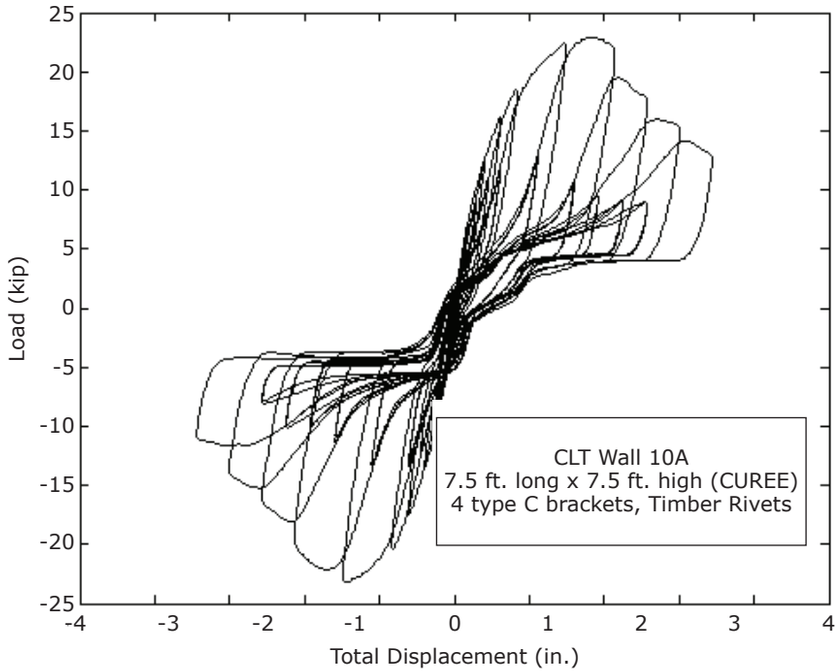

Figure 23
Hysteretic behavior for Wall 10A using timber rivets

By introducing a half-lapped joint in the wall, thus creating a wall made of two separate panels, the behavior of the wall was not only influenced by the types of fasteners in the bottom brackets, but also by the types of fasteners used in the half-lapped joint. These walls (Walls 11 and 12) showed stiffness reduced by 32% and 22%, respectively, and a slightly reduced strength, with respect to the reference Wall 03. Both walls shifted the occurrence of the yield load F_y and ultimate load F_u to higher deflection levels, while only Wall 12 was able to show an increase in its ultimate deflection.

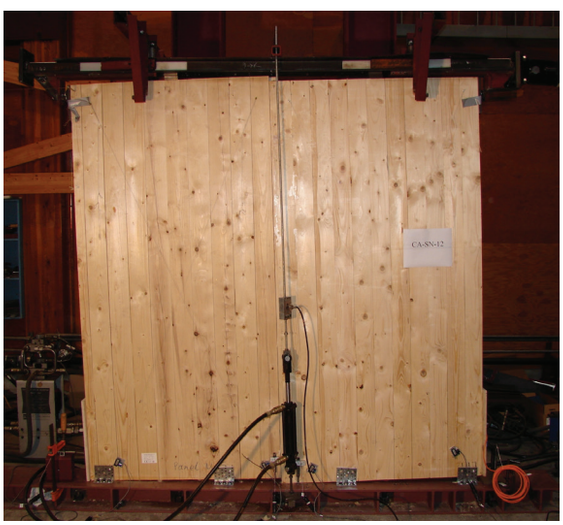

Figure 24
Behavior of Wall 12 using two panels during testing

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Wall 11 with WT-T screws in the half-lapped joint showed ultimate load reduced by 19%, while Wall 12 with regular 0.20 in. (5.0 mm) x 3.5 in. (90 mm) screws showed a reduction of only 5%. In addition, Wall 11 showed higher reduction of ductility compared to the reference Wall 03, while the ductility for Wall 12 was only slightly lower than that of the reference wall. Based on the results, in the case of multi-panel walls with half-lapped joints, the use of regular screws is recommended in high seismic zones. A photo of Wall 12 during the testing is shown in Figure 24, while the behavior of Walls 11 and 12 are shown in Figures 25 and 26, respectively.

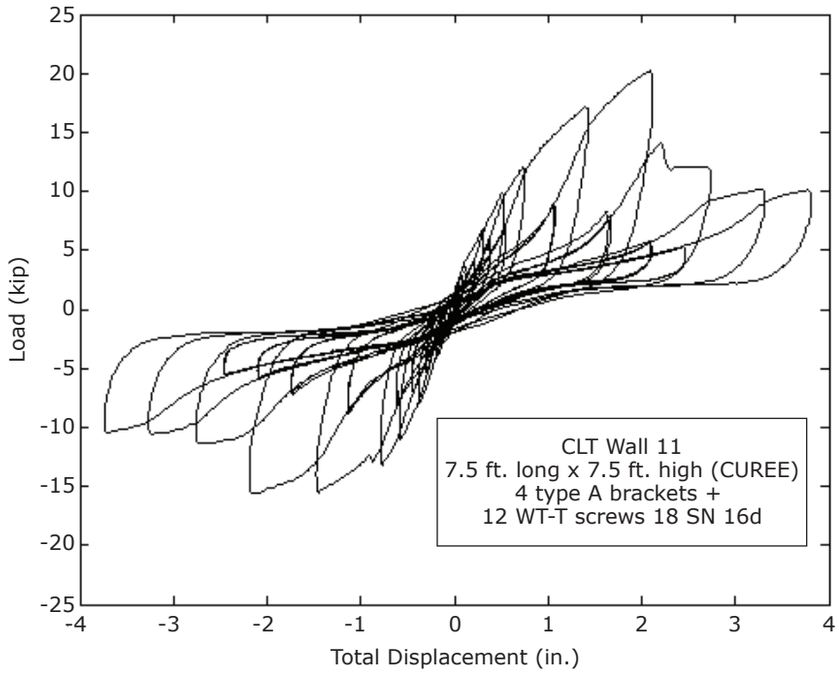

Figure 25
Hysteretic behavior for Wall 11 with 0.15 in. (3.8 mm) x 3.5 in. (89 mm) WT-T screws used in the half-lapped joints

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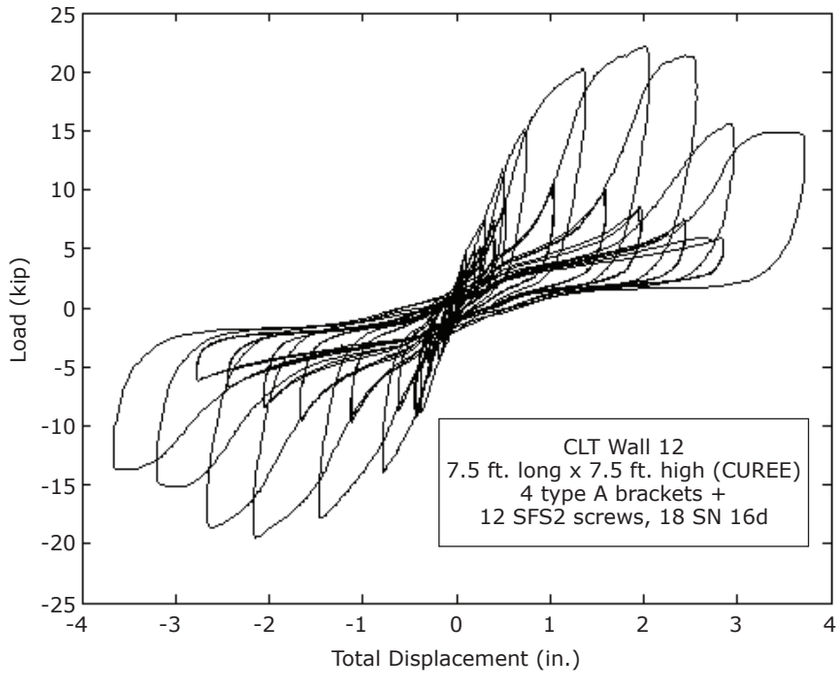

Figure 26
Hysteretic behavior for the two-panel Wall 12 with 0.2 in. (5.0 mm) x 3.5 in. (90 mm) SFS2 screws used in the half-lapped joints under CUREE cycling protocol

The presence of the half-lapped joints and the type of fasteners used to connect them was found to have more significance on the overall wall behavior as the length of the wall increases. For example, results from Walls 14 and 16, which had lengths of 11.3 ft. (3.45 m), show a significant change in stiffness and strength for Wall 16 with half-lapped joints (Figure 28) compared to Wall 14, which had no half-lapped joints (Figure 27). The half-lapped joints enabled Wall 16 to carry a significant portion of the maximum load at higher deformation levels, but at a considerable (25%) reduction in maximum strength.

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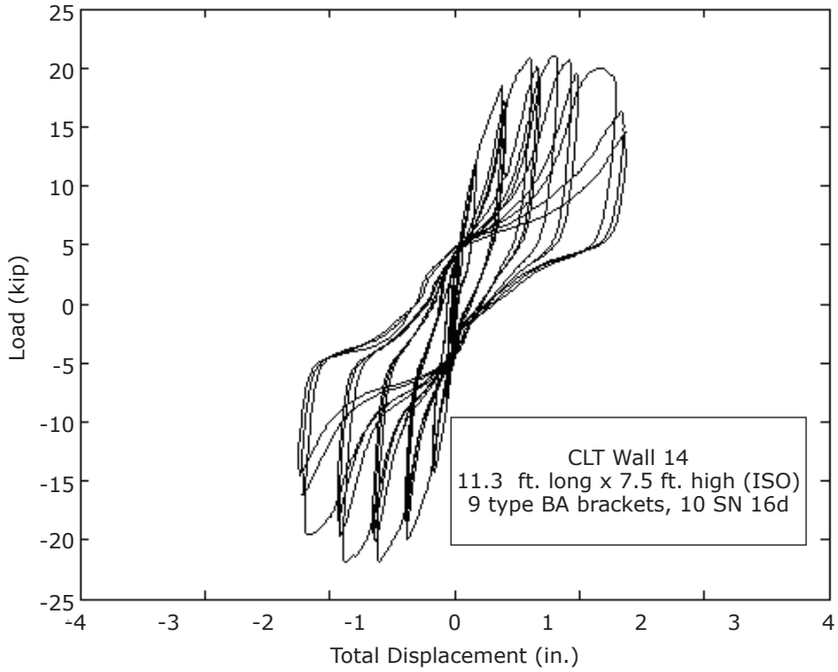

Figure 27
Hysteretic behavior for Wall 14 consisting of one 11.3 ft. (3.45 m) long panel

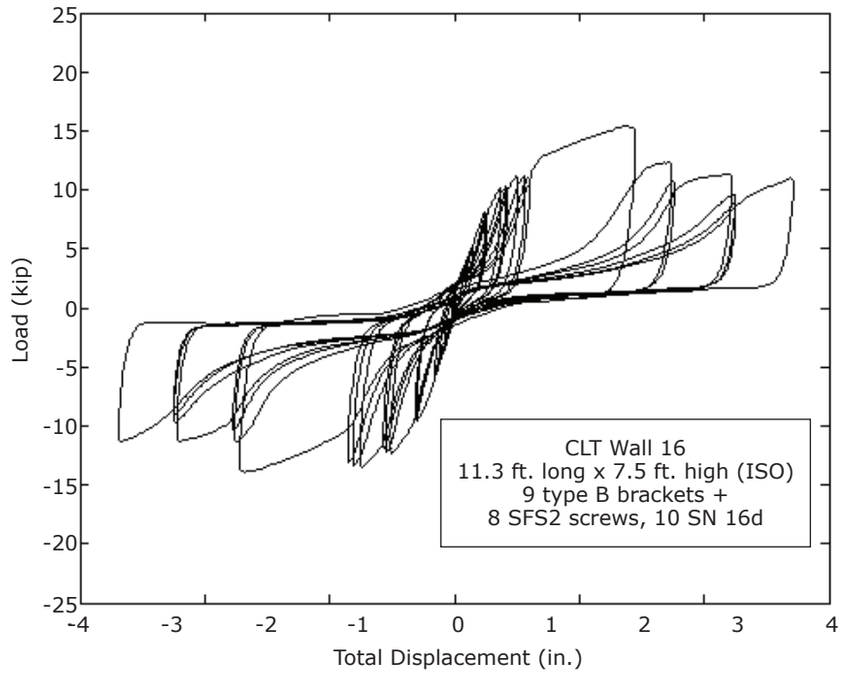

Figure 28
Hysteretic behavior for the three-panel Wall 16 where panels were connected with regular 0.2 in. (5.0 mm) x 3.5 in. (90 mm) SFS2 screws

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It is a well-known fact that the protocol used for cyclic testing of wood-based connections or structural assemblies has an influence on the test results. By comparing results for Walls 12 and 12A (Figures 26 and 29), it can be seen that the choice of the protocol had very little influence on the stiffness of the wall, the yield deflection (both determined using the EEEP method), and the maximum load (Table 6). However, there was significant difference in the deflection at which the maximum load was reached (1.6 in. (41 mm) with ISO vs. 2.1 in. (53 mm) with CUREE), and in the ultimate deflection, which was 2.8 in. (72 mm) using the CUREE protocol vs. 2.2 in. (57 mm) using the ISO protocol.

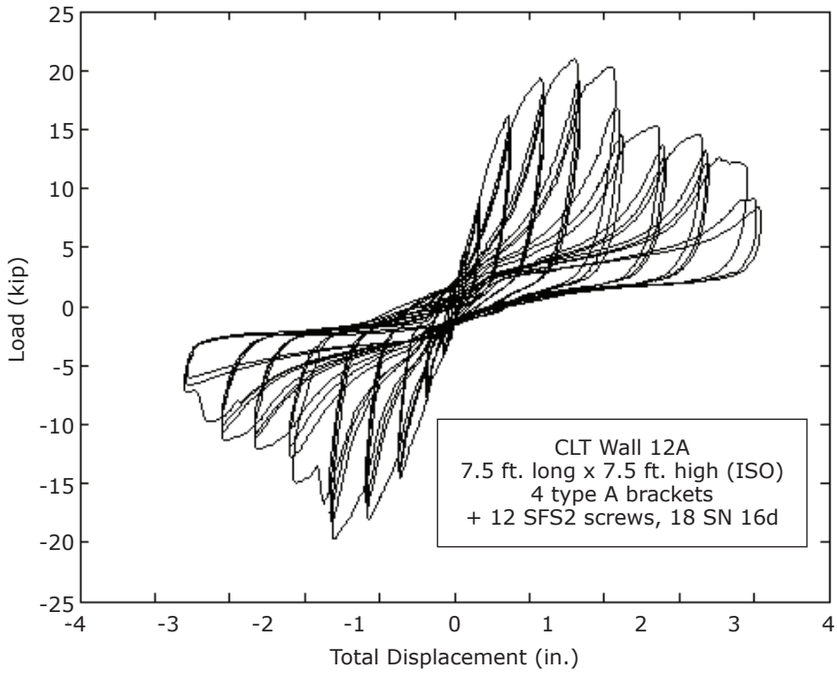

Figure 29
Hysteretic behavior for the two-panel Wall 12A tested under ISO cycling protocol

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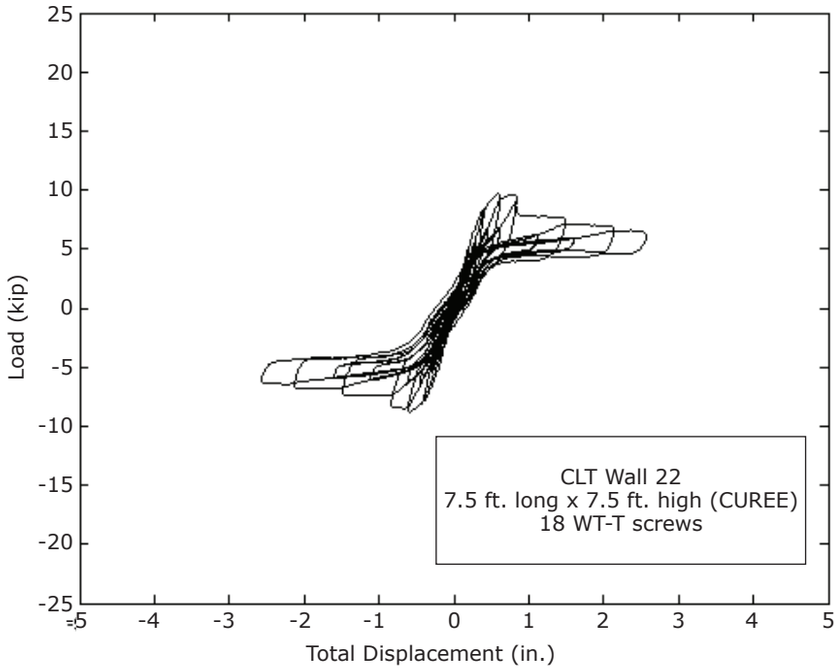

Figure 30
Hysteretic behavior for Wall 22 with WT-T screws

Walls 22 (Figure 30) and 22B that were connected to the base CLT panel with WT-T type screws placed at 45° showed lower resistance than any single story wall in the program. Grouping the screws at the ends of the panels (Wall 22B) created a hold-down effect and helped increasing the wall capacity by about 30% compared to that of Wall 22. Based on the test results, the use of screws at an angle as a primary connector for wall-to-floor connections is not recommended for structures in seismic regions due to reduced capability for energy dissipation (Figure 30) and the sudden pull-out failure of screws in tension.

The behavior of the tall walls specimens with riveted connections was highly influenced by the number of rivets used in each bracket. Although the number and spacing of the rivets in the brackets for Walls 24, 25 and 26 were chosen to satisfy the rivet yielding failure mode according to the existing Canadian code specifications for sawn lumber and glulam, they did not yield but experienced fastener pull-out combined with a wood shear plug failure mode (Figure 31a). This failure mode is not ductile and should be avoided in practice. By increasing the spacing between the rivets in Wall 27, the failure mode was changed to the desired rivet yielding mode (Figure 31b).

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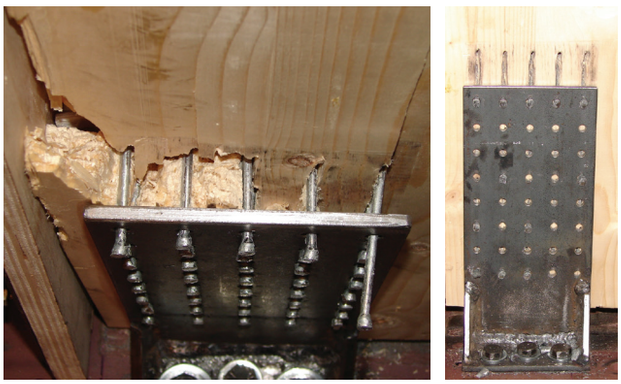

a) b)

Figure 31
Bracket failure modes for a) Wall 25 with 40 L=2.56 in. (65 mm) rivets, and b) Wall 27 with 20 L=3.5 in. (90 mm) rivets

Finally, several two-story wall tests have been conducted by Popovski et al. (2010) and interested readers are referred to that paper for further information on test setup, protocol, and results.

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APPENDIX B
DESIGN EXAMPLE

B1 Performance of a Multi-story CLT Building Designed Using Force-based Design

Utilizing the results of the numerical model and shear wall tests (FPInnovations) introduced earlier in this Chapter, a multi-story CLT building assumed to be located at a generic California site was designed using the Equivalent Lateral Force Procedure (ELFP) outlined in ASCE 7-10. The performance of the as-designed structure was then assessed numerically under a suite of earthquake ground motions scaled to both the design basis earthquake (DBE) and maximum considered earthquake (MCE) intensity.

The floor plan of the NEESWood Capstone building (Pei et al., 2009; van de Lindt et al., 2010) was used to illustrate the design process and quantitatively present the performance of a mid-rise (6-story) CLT building. In the Equivalent Lateral Force Procedure outlined in ASCE 7-10, a response modification factor (R) must be selected based on the lateral force resisting system being utilized in the design. However, as explained earlier in this Chapter, seismic response modification factors are not yet available for CLT systems in the United States. Therefore, the design was repeated for four different R-factors over a range, namely R=2, 3, 4, and 6. The basic building properties used in the ELFP are listed in Table 7.

Table 7
Building properties used in the illustrative design examples

Story	1	2	3	4	5	6
Concentrated Story Weight (kips)	95.7	95.7	95.7	95.7	88.7	62.6
Cumulative Weight (kips)	534.2	438.5	342.8	247.1	151.4	62.6
Height from Ground (ft.)	9	18	27	36	45	54

The design spectral values used for this example are S_{DS}=1.0 g and S_{D1}=0.6 g (corresponding to MCE level S_s=1.5 g and S1=0.9 g). The base shear coefficients based on the ELFP were calculated to be 0.50, 0.33, 0.25, and 0.17 for R=2, 3, 4, and 6, respectively. The design of the CLT building was conducted by selecting a CLT wall configuration for each story based on the LRFD wall resistance per Table 2 in Section 5. The wall selection in this design example is constrained by the architectural floor plan (shown in Figure 32) in that only a limited amount of wall segments can be placed in each story. The list of available wall segments and their lengths for each story is summarized in Table 8. Note that the X-direction is the long direction of the floor plan; the Y-direction is the short direction. This notation will be used throughout this illustrative example.

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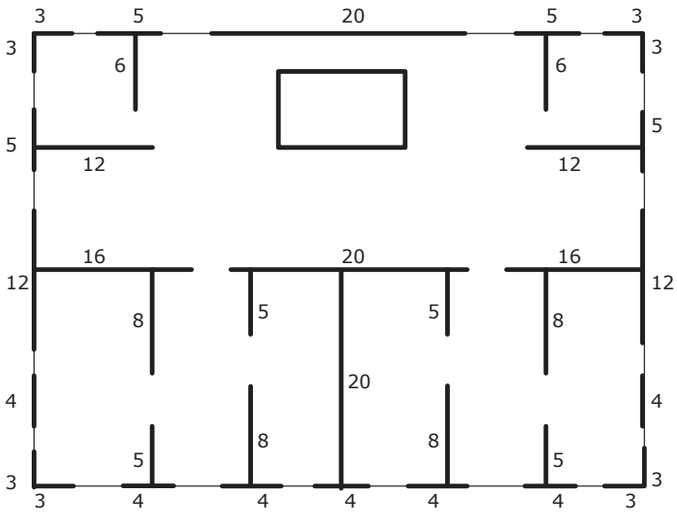
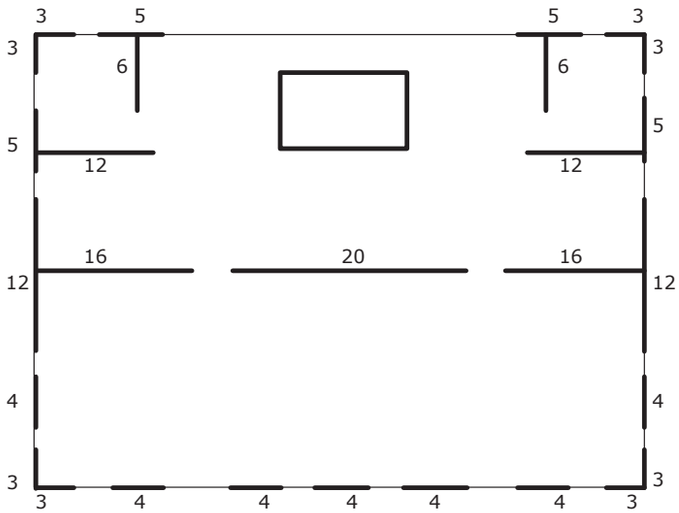
Table 8
Number of various length wall segments for each story at each direction

Segment Length (ft.)	3	4	5	6	8	12	16	20
Story 1-5: X	4	5	2	0	0	2	2	2
Story 1-5: Y	4	2	6	2	2	2	0	1
Story 6: X	4	5	2	0	0	0	2	1
Story 6: Y	4	2	2	2	0	2	0	0



(a)



(b)

Figure 32
Wall segments (lengths in ft.) allowed for example building floor plans (a) stories 1-5 and (b) story 6.

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The resulting designs are presented in Table 9. Because the 2S configuration is the weakest configuration available, some of the available wall segments were not designated as structural walls if the story shear demand was exceeded using 2S configuration on part of the available wall segments. The nonlinear time history analysis conducted later in this example considered only the structural wall segments, i.e. the lateral resistance contribution from other panels was neglected.

Table 9
Wall design configurations and total length selected to satisfy shear demands

Story	R=2			R=3			R=4			R=6		
	Config.	X[1]	Y[1]	Config.	X[1]	Y[1]	Config.	X[1]	Y[1]	Config.	X[1]	Y[1]
1	4DE	138	122	2DE	118	102	2S	138	122	2S	98	78
2	4DE	138	122	2DE	118	102	2S	138	122	2S	98	78
3	3DE	138	122	2DE	98	78	2S	138	122	2S	90	78
4	2DE	138	122	2S	138	122	2S	98	78	2S	58	50
5	2S	138	122	2S	118	102	2S	66	62	2S	34	40
6	2S	94	66	2S	42	30	2S	42	30	2S	26	20

[1]Total wall length (in feet) selected to satisfy shear demands

The performance of the 6-story building was assessed using nonlinear time history analysis with a suite of ground motions. The ground motion suite included 22 bi-axial far-field ground motions developed during the ATC-63 research project (ATC, 2008; FEMA, 2009). These ground motions were scaled so that their averaged response spectrum approximately matches the design response spectra at DE and MCE levels. The bi-axial ground motions were also rotated by 90 degrees and thus, at each hazard level, each design was analyzed using each of the 22 record pairs for a total of 44 analyses.

The maximum story drift experienced by the building was obtained from each nonlinear time history simulation, resulting in 44 maximum story drift values for each building at each hazard level. These maximum drift values were rank-ordered and plotted as empirical cumulative distribution functions (CDF) as shown in Figures 33 and 34. Story drift levels of 3.5% during CLT wall tests have been observed, during which the structure remains stable. The analysis presented herein utilizes 3.5% as a story drift limit for purposes of the nonlinear time history analyses in this Appendix. It is noted that this drift limit is still considered preliminary because full CLT systems have not yet been tested to failure to verify the suitability of this story drift limit.

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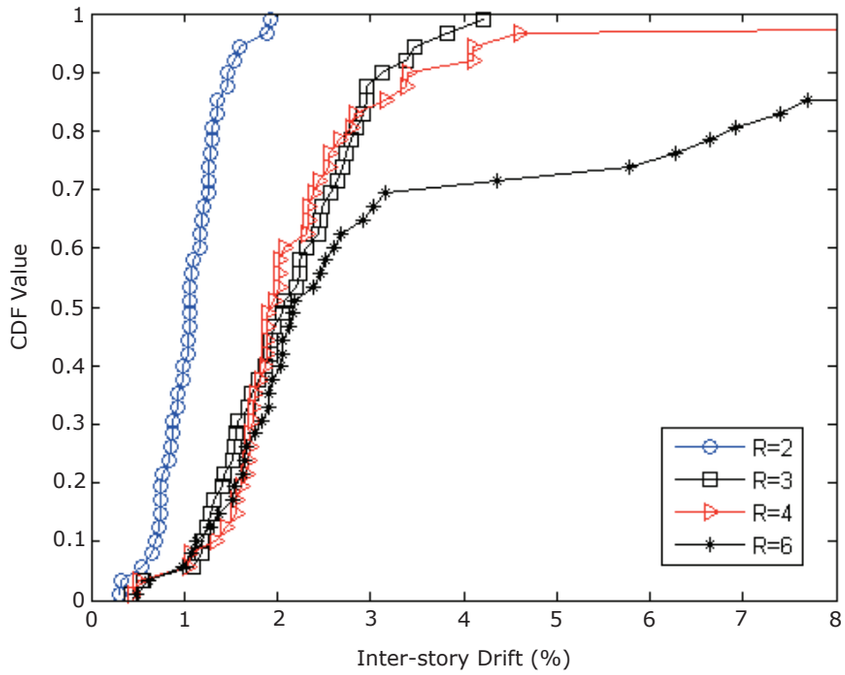

Figure 33
Performance of as-designed CLT building under DE level

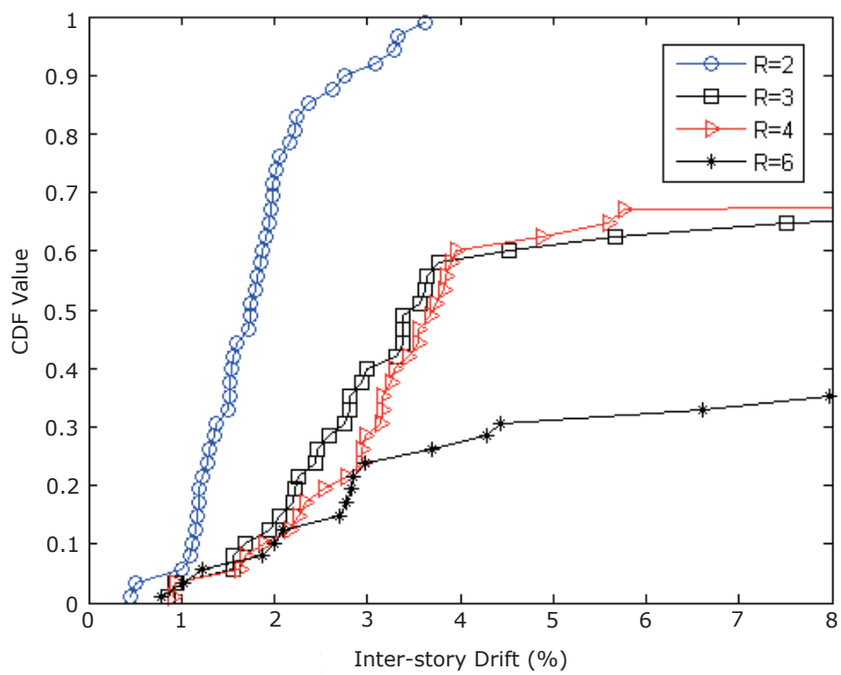

Figure 34
Performance of as-designed CLT building under MCE level

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It can be seen that, when subjected to the suite of DBE level ground motions, there was greater than a 90% non-exceedance for R less than 4. At R=6, the probability of not exceeding 3.5% story drift reduces to about 70%. At the MCE intensity, a maximum story drift of 3.5% corresponds to approximately less than a 10% exceedance probability using R equal to 2.

While discussing Figures 33 and 34, there are a few important assumptions that should be kept in mind. The numerical model used in this analysis ignored the contribution from all non-structural walls and did not account for boundary conditions resulting from other transverse wall sections. In typical CLT construction, the panels that are not selected in the design as structural will also be connected minimally to the floor diaphragm, thus contributing to building lateral resistance. Additionally, collapse is judged to occur at 3.5% story drift but the collapse mechanism is not specifically modeled. Determination of simulated collapse mechanisms and story drift levels associated with collapse are subject to future refinement of CLT modeling and evaluation.

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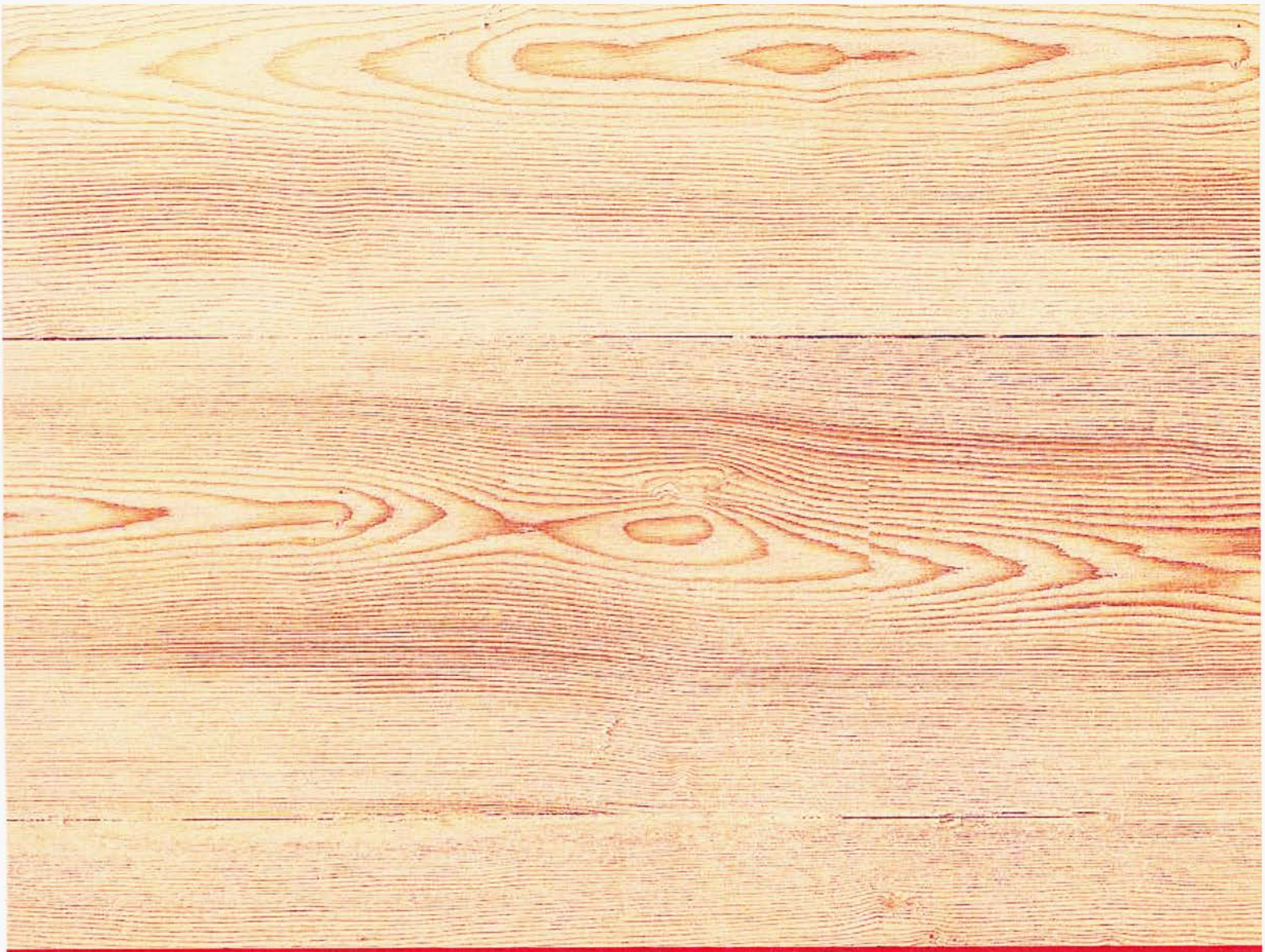

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CLT Handbook

CROSS-LAMINATED TIMBER

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