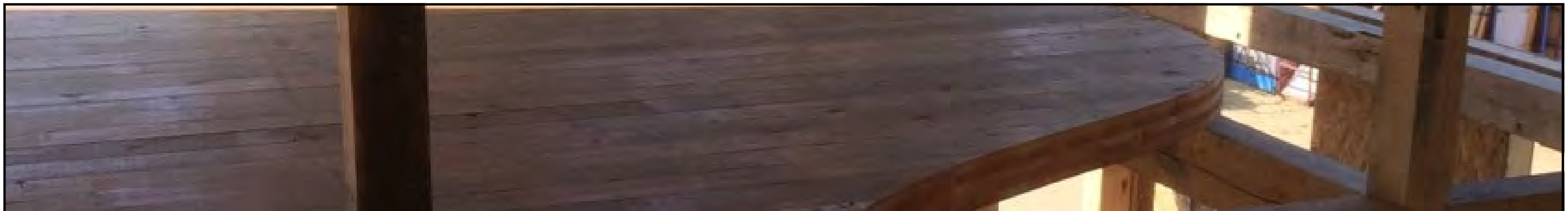

Cross-Laminated Timber (CLT) Shear Wall

Philip Line, M. Omar Amini

Objectives

- Be introduced to design requirements for CLT shear walls appearing in SDPWS-21 Appendix B
- Learn about seismic design coefficients and the associated height limits for the CLT shear wall system appearing in ASCE/SEI Standard 7-22
- Learn about the CLT shear wall design example appearing in the 2020 NEHRP Provisions: Design Examples

Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n, Limits (ft)[d] — Seismic Design Category: B	C	D[e]	E[e]	F[f]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Reinforced concrete ductile coupled walls[q]	14.2	8	2½	8	NL	NL	160	160	100
3. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
4. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
5. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
6. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
7. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
8. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
9. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
10. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
11. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
12. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
13. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
14. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
15. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
16. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
17. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
18. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
19. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
20. Cross-laminated timber shear walls	14.5	3	3	3	65	65	65	65	65
21. Cross-laminated timber shear walls with shear resistance provided by high-aspect-ratio panels only	14.5	4	3	4	65	65	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100

Background

Cross-laminated timber (CLT)

- Stadthaus, London, 2009
- Residential
- 9 stories
- 9 weeks of CLT construction
- 4 laborers
- 1 supervisor

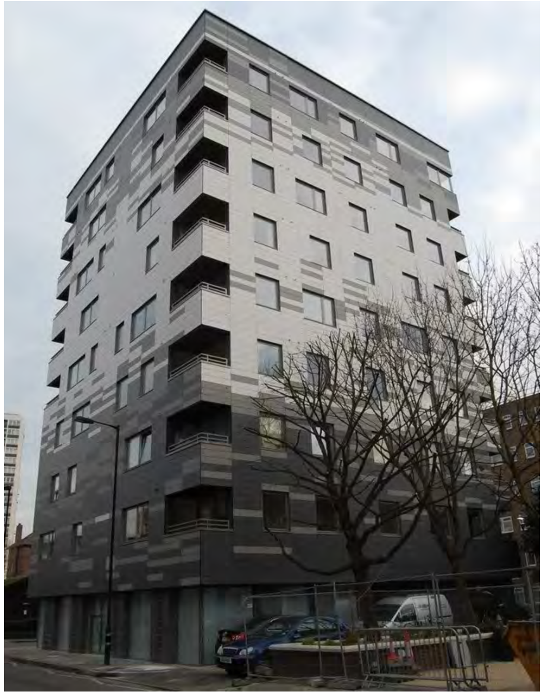

Photo credit: Will Pryce

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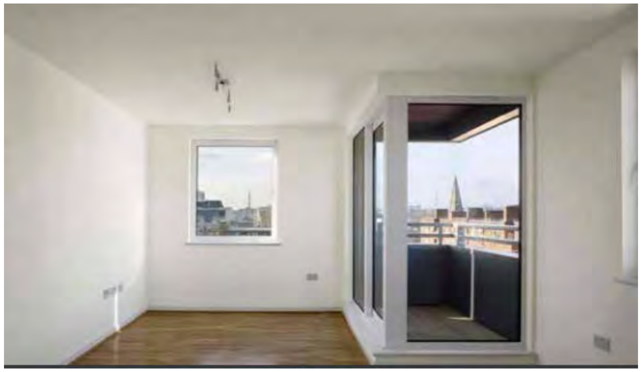
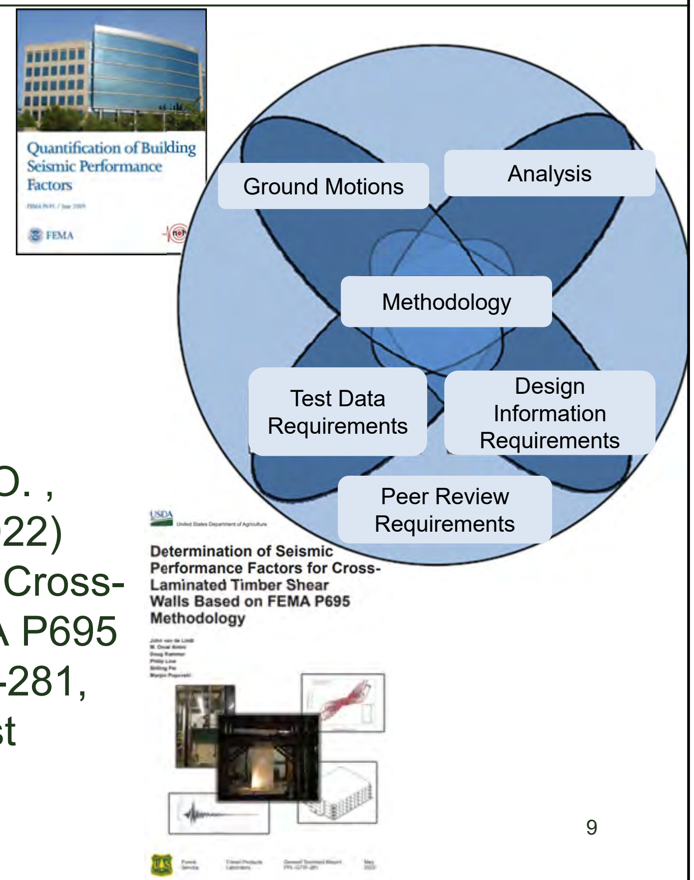


Background

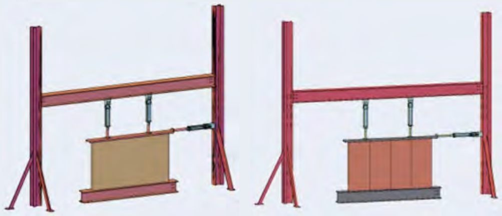
Fort Drum, NY (4-story), 2017; Courtesy Jeff Morrow, Lendlease

Background

- FEMA P-695: Quantification of Building Seismic Performance Factors
- Peer review throughout
- Archetypes
- Design methodology
- Nonlinear time history analysis
- Performance evaluation (CMR & ACMR)
- Project Documentation: van de Lindt, J., Amini, M. O. , Rammer, D., Line, P., Pei, S., and Popovski, M. (2022) “Determination of Seismic Performance Factors for Cross-Laminated Timber Shear Walls Based on the FEMA P695 Methodology.” General Technical Report FPL-GTR-281, Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory.



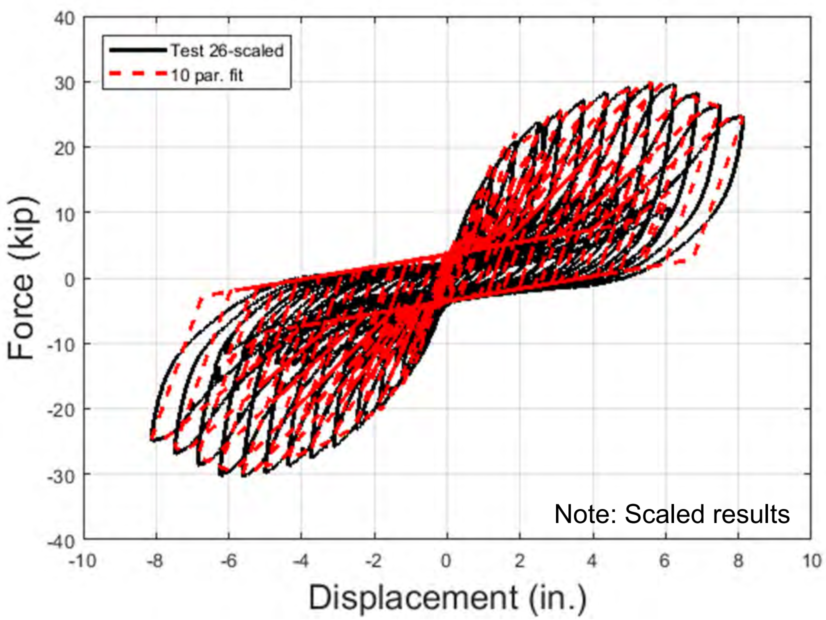
6.2 Background

Test Type	Objective	
Connector tests	• Behavior under cyclic loading • Interpanel connector behavior	
Isolated Wall Tests	• Boundary condition • Gravity loading • Connector type • CLT panel thickness • Connector thickness • CLT panel aspect ratio • Inter-panel connector (vertical joint) • CLT grade	
Full-scale Two-Story Shake Table Test	• Effect of diaphragm on wall behavior • Effect of out-of-plane loading on the connector • Vertical joints between perpendicular walls will also be investigated • Hold-downs in the corners • Stability of the walls • Effect of diaphragm rotation • Combined loading on the connectors	• Three Configurations – 4:1 aspect ratio panels – 2:1 aspect ratio panels – 4:1 aspect ratio panels with transverse walls

6.2 Background

6.2 Background

6.2 Background



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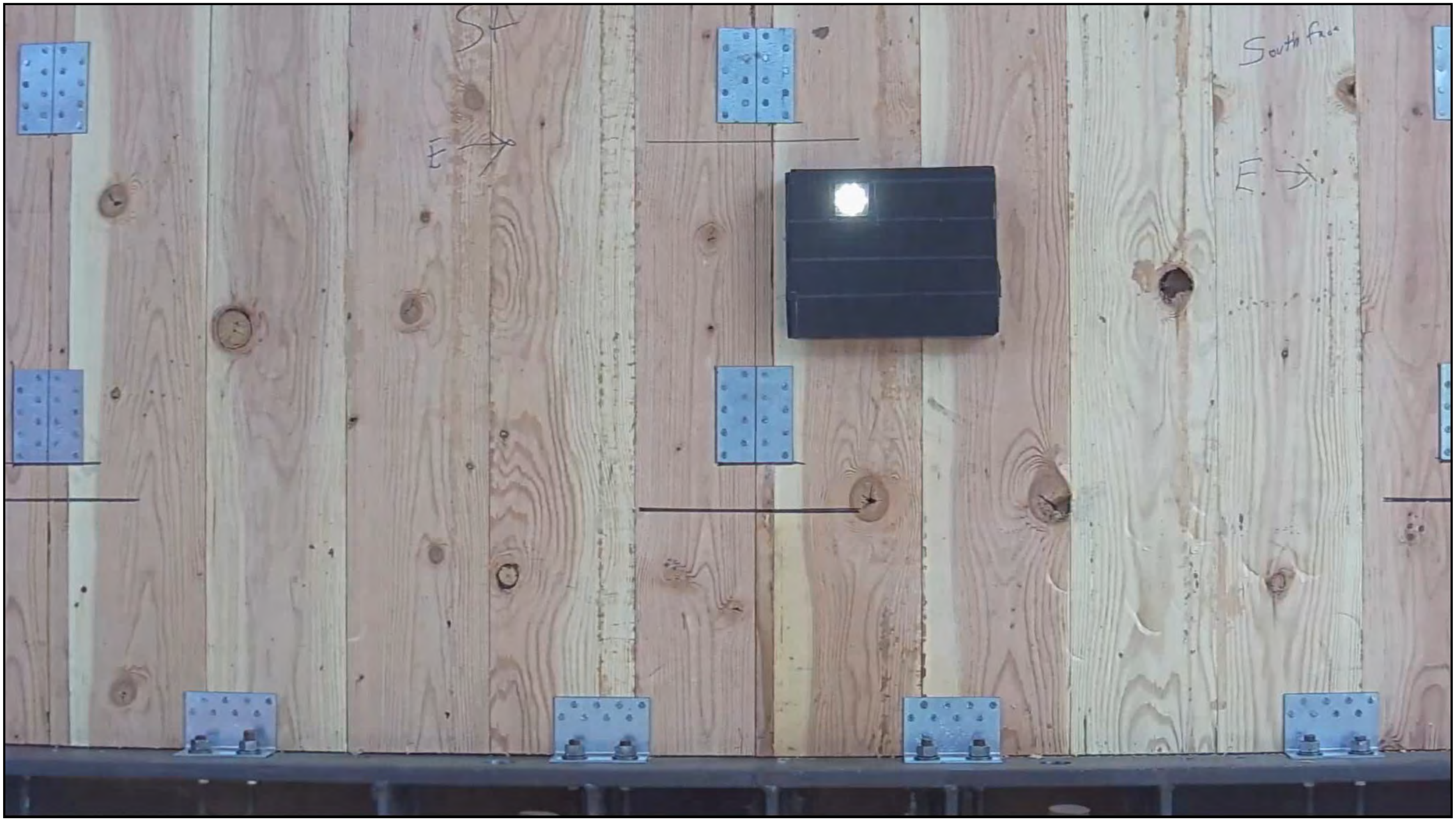

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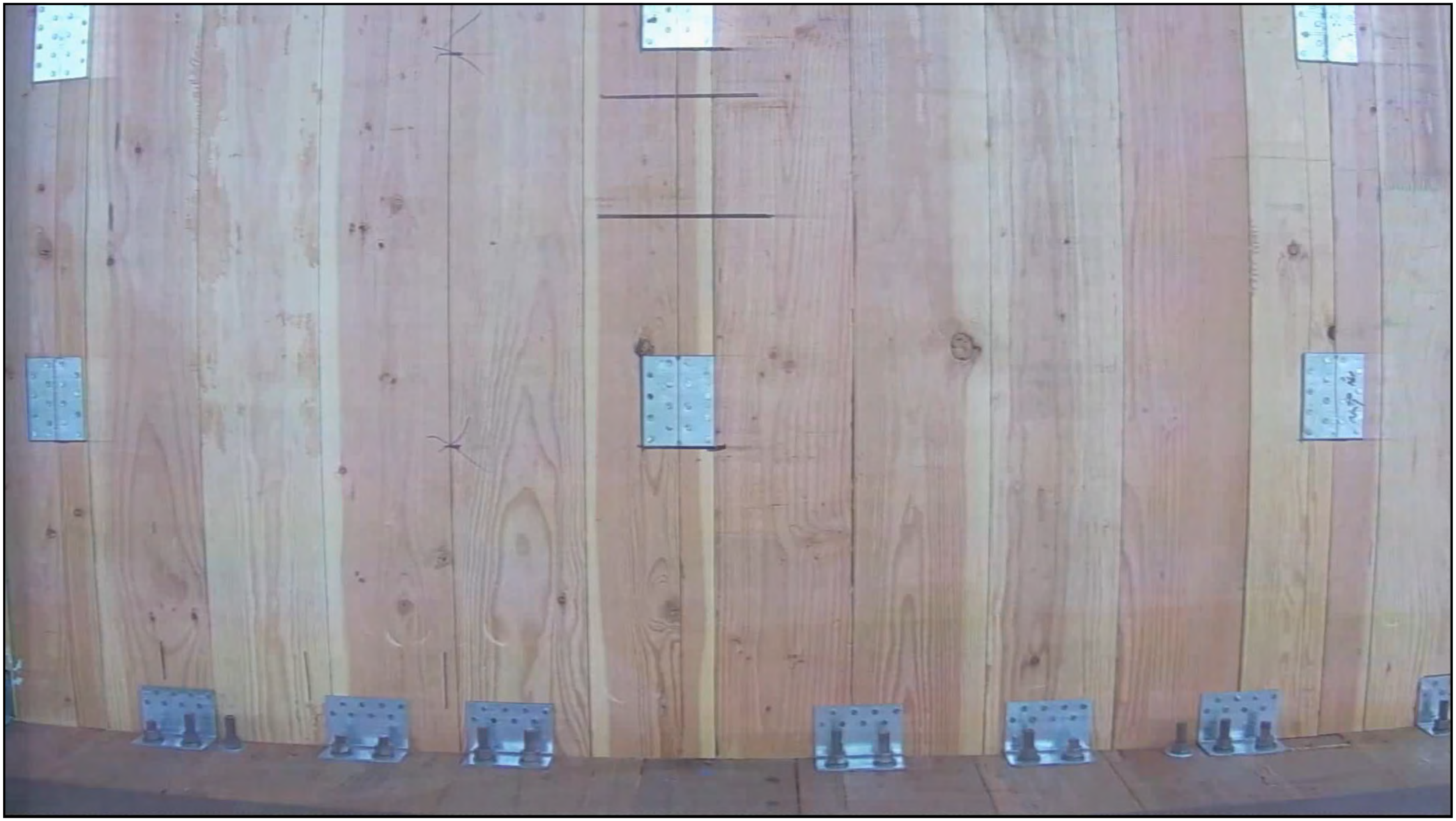

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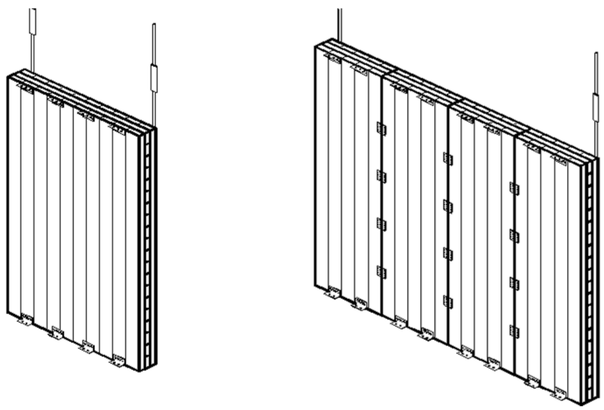
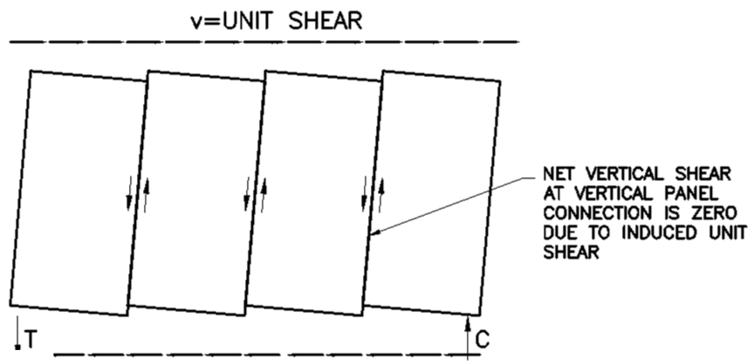
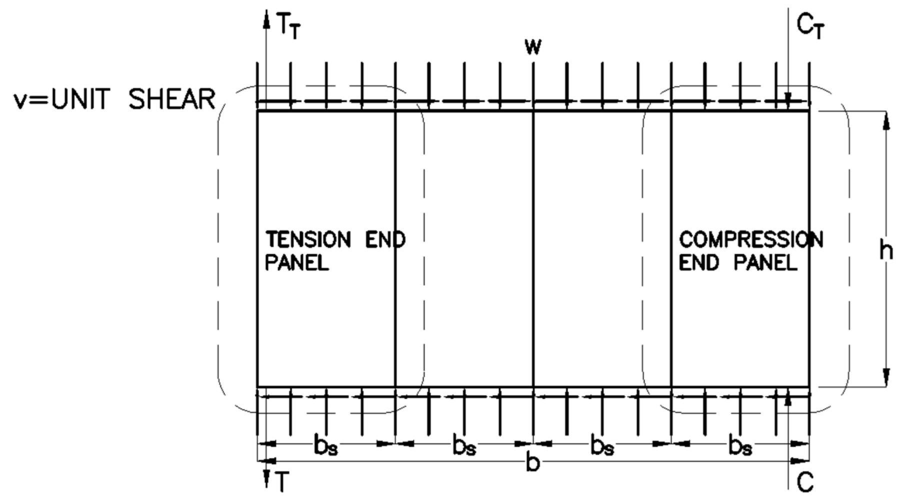



v=UNIT SHEAR
NET VERTICAL SHEAR
AT VERTICAL PANEL
CONNECTION IS ZERO
DUE TO INDUCED UNIT
SHEAR
T
C


v=UNIT SHEAR
T_T
w
C_T
TENSION END
PANEL
COMPRESSION
END PANEL
h
b_s
b_s
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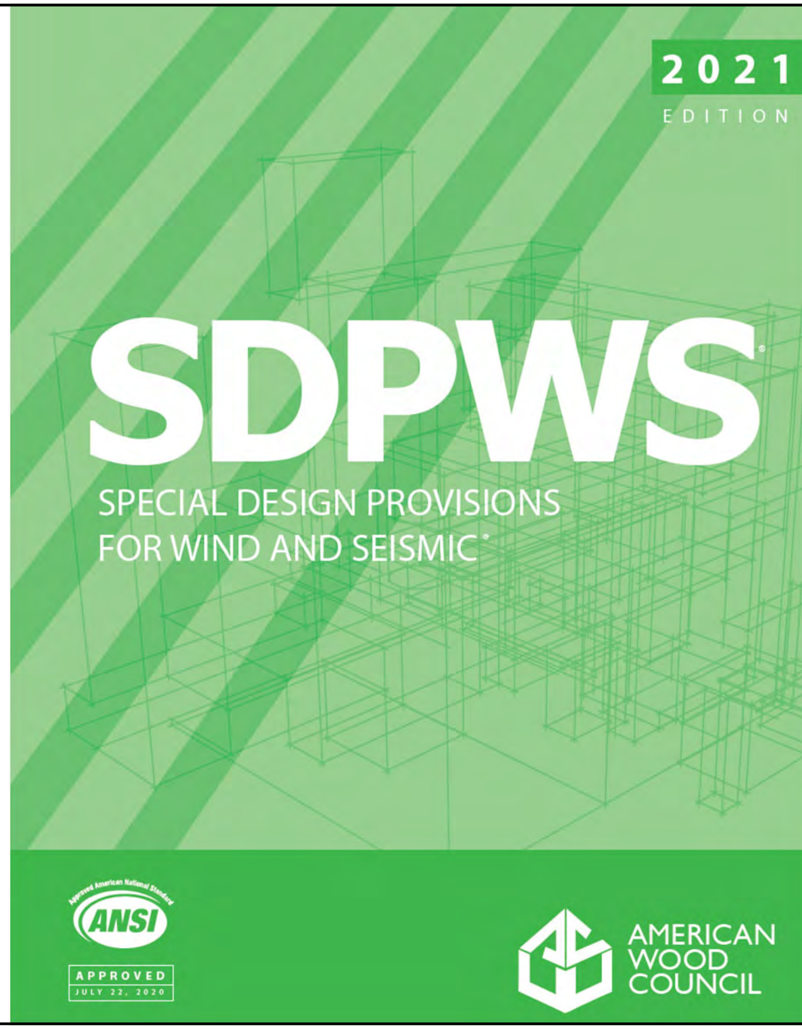
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(Mandatory) Requirements for CLT Shear Walls

B.1 Scope

These provisions shall be used for the design and construction of structural cross-laminated timber (CLT) members and connections that are part of the lateral force-resisting system. Capacity design principles are employed to ensure development of the expected shear capacity of the prescribed nailed connectors of the CLT shear wall. The provisions provided herein shall be applied in combination with the requirements of this Standard, NDS including Appendix E, ASCE 7, and the applicable building code.

B.2 Application Requirements

The design and construction of the CLT lateral force-resisting system (LFRS) shall comply with all of the following:

1. The method of construction shall be platform construction in accordance with the following:
   a. CLT floor panels bear on and are supported by CLT shear walls that are part of the designated LFRS. Additional gravity support is permitted to be provided by other gravity framing system elements including but not limited to CLT walls that are not part of the designated LFRS, beams, columns, and light-frame walls.
   b. CLT floor panels are designed as the floor diaphragm to distribute lateral loads to the CLT shear walls.
2. CLT walls shall be classified as either (1) part of the designated LFRS (i.e. CLT shear wall) or (2) not part of the designated LFRS.
3. CLT walls that are not part of the designated LFRS shall meet the following requirements:
   a. CLT panels forming either a single-panel or multi-panel wall shall have an aspect ratio, h/bs, that is not less than the aspect ratio used for CLT shear wall panels in accordance with B.3.1 or B.3.7.
   b. Shear connections at the top and bottom of CLT wall panels shall be in accordance with B.3.2. Where shear connections are provided at adjoining vertical panel edges to form a multi-panel CLT wall, such connections shall be in accordance with B.3.3. Hold-down systems in accordance with B.3.4 are not required.
4. CLT walls that are not part of the designated SFRS shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the designated SFRS.
   a. The design shall provide for the effect of the CLT walls that are not part of the designated SFRS on the structural system at deformations corresponding to the design story drift, the distribution of forces to the structural system, and the corresponding load path to the final point of resistance; this shall be achieved by design for the most critical demands to vertical gravity load supporting elements, vertical SFRS elements, and diaphragms, and their load paths, determined in accordance with both of the following:
      i. Force and deformation demands determined including in the analysis CLT shear walls of the designated SFRS, but neglecting in-plane shear strength and stiffness of CLT walls that are not part of the designated SFRS, and
      ii. Force and deformation demands determined including in the analysis both CLT shear walls of the designated SFRS, and CLT walls that are not part of the designated SFRS.
   b. In addition, the effects of CLT walls classified as not part of the designated SFRS shall be considered where determining whether a structure has one or more of the irregularities defined in ASCE 7 12.3.2 based on the two analysis cases in accordance with item 4a.
   c. Where the fundamental period of the structure is calculated in accordance with ASCE 7 using the structural properties and deformational characteristics, it shall include the effects of stiffness of CLT walls classified as not part of the designated SFRS.
   d. For the purpose of Items 4a, 4b and 4c, CLT wall panels that are classified as not part of the designated SFRS shall be modeled as-

suming they develop in-plane shear strength and stiffness associated with CLT shear walls of the same construction.

5. Shear distribution to individual CLT shear walls in a wall line shall provide the same calculated deflection in each shear wall (i.e. distribution of loads based on relative stiffness).
6. The dead load considered in the overturning design of each individual CLT panel within the CLT shear wall shall be limited to the dead load supported by or directly above the individual CLT panel.
7. CLT shear walls shall be full-height within each story. CLT shear walls are not permitted to be designed as Force-Transfer Around Openings (FTAO) shear walls or as Perforated Shear Walls.
8. The nominal unit shear capacity assigned to CLT shear walls shall not include contributions from connections other than those shear connections prescribed in B.3.2 and B.3.3.
9. ASCE 7 seismic criteria specific to light-frame construction shall not apply to the design of CLT shear walls and CLT diaphragms.

B.3 CLT Shear Wall Requirements

B.3.1 CLT Panels

CLT panels used in CLT shear walls shall be designed in accordance with the NDS and the following requirements:

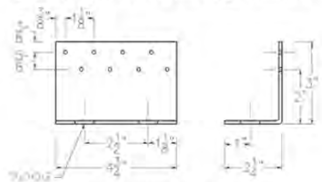
1. CLT in-service moisture content shall be less than 16% and specific gravity, G, shall be 0.35 or greater.
2. CLT panels forming either a single-panel or multi-panel shear wall shall have aspect ratio, h/b_s, not greater than 4 nor less than 2. All CLT panels forming a multi-panel shear wall shall have the same panel height, h, and individual panel length, b_s.
3. CLT panels shall be a minimum of 3.5 in. in thickness. Where angle connectors or vertical edge connectors are installed in both faces of the CLT panel and are directly opposed, CLT panels shall be a minimum of 6.875 in. in thickness so that nails from opposing faces do not overlap.
4. Holes, notches and other modifications to CLT panels shall not be permitted unless the effects of removal of material on load carrying capacity of the CLT panel is determined by an approved rational analysis.

B.3.2 Top and Bottom of Wall Angle Connector

Shear connections at the top and bottom of CLT shear walls shall be composed of prescribed steel angle connectors, nails, and bolts in accordance with the following requirements:

1. Angle connectors shall be fabricated from 0.105 in. thickness, ASTM A653 Grade 33 steel sheet with the geometry as illustrated in Figure B.3.2.
2. Vertical legs of angle connectors shall be fastened to wall panel using (8) 16d carbon steel box nails (3.5 in. length x 0.135 in. shank diameter x 0.344 in. head diameter) with bright finish in accordance with ASTM F1667 including Supplementary Requirements of ASTM F1667 S1 Nail Bending Yield Strength.
3. Horizontal legs of angle connectors shall be fastened to supporting elements (e.g. CLT floor or roof panels, concrete foundation elements, or roof framing elements other than CLT) with (2) 5/8in. diameter x 4-1/2 in. long (minimum) bolts to provide minimum 4-1/2in. bearing length with washer per ASME B18.21.1, or (2) 5/8 in. full-body diameter lag screws with 2-3/4 in. thread penetration (minimum) excluding tapered tip and 3-1/8 in. unthreaded shank length (minimum) to provide minimum 5-7/8 in. bearing length and with washer per ASME B18.21.1. Bolts and lag screws shall be ASTM A307 Grade A or higher. The anchorage, foundation, and other supporting elements shall be capable of resisting a concurrent tension force and shear force transmitted through horizontal leg fasteners, with force in each direction equal to the connector nominal shear capacity. The design of the 5/8 in. diameter anchor to concrete foundation or other concrete elements shall be in accordance with ACI 318 and the prescribed loading above shall be considered to meet the ductile yield mechanism requirement of ACI 318.
4. Angle connectors at the bottom and top of wall panels shall extend to within 12 in. of each end of each panel of a single or multi-panel shear wall.
5. Each wall panel shall have at least two angle connectors at the top and bottom. The same number of angle connectors shall be provided at the top and bottom of each wall panel.

Figure B.3.2 Top and Bottom of Wall Angle Connector

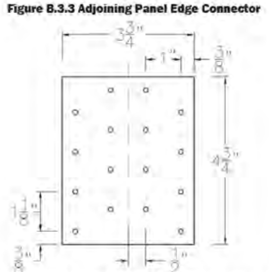


B.3.3 Adjoining Panel Edge Connector

In CLT multi-panel shear walls, shear connections at adjoining vertical panel edges of CLT wall panels shall be composed of prescribed steel plate connectors and nails in accordance with the following requirements:

1. Plate connector shall be fabricated from 0.105 in. thickness, ASTM A653 Grade 33 steel sheet with the geometry as illustrated in Figure B.3.3.
2. Plate connectors shall be fastened to each wall panel using (8) 16d carbon steel box nails (3.5 in. length x 0.135 in. shank diameter x 0.344 in. head diameter) with bright finish in accordance with ASTM F1667 including Supplementary Requirements of ASTM F1667 S1 Nail Bending Yield Strength.
3. The number of plate connectors required at adjoining vertical edges of CLT panels shall equal the number of angle connectors along the bottom edge of the wall panel times the CLT panel aspect ratio, h/b_s, rounded up to the next whole number.

Figure B.3.3 Adjoining Panel Edge Connector



B.3.4 Hold-down System

Each end of each shear wall shall be provided with a hold-down system. Hold-down systems shall comply with the following:

1. Hold-down systems shall consist of continuous tie-rod systems or conventional hold-down devices that use threaded anchor rods and nail, screw or bolt attachment to the CLT panel.
2. Where continuous tie-rod systems are used, rods at each level shall be designed for cumulative overturning tensile forces and bearing plates shall be provided at the floor level above each story. Tie-rod elongation or conventional hold-down device deformation for each story shall not exceed 0.185 in. when applying strength design load combinations of ASCE 7.
3. The hold-down system including anchorage to the foundation shall be designed to resist not less than 2.0 times the forces associated with the design shear capacity of the CLT shear wall determined in accordance with 4.1.4.1 for seismic and 1.5 times the forces associated with the design shear capacity of the CLT shear wall in accordance with 4.1.4.2 for wind. Connections between the hold-down device and CLT panel shall comply with net section tension rupture, row tear-out, group tear-out in accordance with NDS Appendix E. The anchorage to foundation shall be designed in accordance with ACI 318.

B.3.5 Compression Zone

The length of the compression zone for overturning compression forces induced by the design loads shall be determined to satisfy static equilibrium assuming a uniform distribution of bearing stress in the compression zone. For multi-panel shear walls, the compression zone shall be contained within the outermost wall panel. CLT wall panel resistance to induced axial compression forces shall be determined using cross section dimensions associated with the compression zone.

B.3.6 Other Load Path Connections to CLT Wall Panels

Connections to CLT wall panels in addition to those connections prescribed in B.3.2 and B.3.3 shall be provided in accordance with 4.1.1 to provide a continuous load path. Load path connections to CLT wall panels shall be with dowel-type fasteners designed to develop Mode IIIs or Mode IV yielding, and com-

ply with net section tension rupture, row tear-out, group tear-out in accordance with NDS Appendix E. Screws (e.g. wood screws and lag screws) shall not be used in supplemental top and bottom of wall connections to supporting elements. Angle connectors prescribed in B.3.2 shall not be considered in design to resist out-of-plane forces.

B.3.7 CLT Shear Walls with Shear Resistance Provided by High Aspect Ratio Panels Only

CLT shear walls with shear resistance provided by high aspect ratio panels only shall meet all requirements applicable for CLT shear walls and the following:

a. All CLT wall panels used as part of the designated lateral force-resisting system shall have aspect ratio, h/b_s, equal to 4, and
b. All CLT wall panels that are not part of the designated lateral force-resisting system shall have aspect ratio, h/b_s, not less than 4.

B.4 Deflection

CLT shear wall deflection, δ_{SW}, shall be calculated by use of the following equation:

$$\delta_{SW} = \frac{576 v b_s h^3}{EI_{eff\,(in-plane)}} + \frac{vh}{GA_{eff\,(in-plane)}} + 3\Delta_{nail\,slip,h} + 2\Delta_{nail\,slip,v}\frac{h}{b_s} + \frac{h\Delta_a}{\sum b_s} \quad \text{(B-1)}$$

where:

v = unit shear induced by the design loads, plf

$EI_{eff\,(in-plane)}$ = effective in-plane bending stiffness of the CLT panel, lbs-in.^{2}

$GA_{eff\,(in-plane)}$ = effective in-plane shear stiffness of the CLT panel, lbs/in. of panel length

h = CLT panel height, ft

b_s = individual CLT panel length, ft

$\sum b_s$ = sum of individual CLT panel lengths, ft

$\Delta_{nail\,slip,h}$ = $V_{nail\,load}/6700$, in.

$\Delta_{nail\,slip,v}$ = $\Delta_{nail\,slip,h}$ in. (=0 for a single panel shear wall)

$V_{nail\,load}$ = load per nail, lbs (calculated as total shear load at base of wall divided by total number of nails in base connectors)

Δ_a = vertical deformation of the wall overturning anchorage system (including but not limited to fastener slip, device elongation, rod elongation, and uncompensated shrinkage) plus the vertical compression deformation, the effects of which are measured at the ends of the shear wall and associated with the unit shear force induced by the design load in the shear wall, in.

B.5 Nominal Unit Shear Capacity

Nominal unit shear capacity, v_s, shall be in accordance with Equation B-2. Where both faces of a panel are provided with angle connectors in accordance with B.3.2, the nominal unit shear capacity shall be permitted to be taken as the sum of the nominal unit shear capacities of each face.

$$v_s = n\left(\frac{2605}{b_s}\right)C_G \quad \text{(B-2)}$$

where:

n = number of angle connectors along bottom of panel face

2605 = connector nominal shear capacity in accordance with NDS (lbs)

b_s = individual CLT panel length (ft)

C_G = CLT panel specific gravity adjustment factor. C_G = 1.0 for $G \geq 0.42$. C_G = 0.86 for G = 0.35. Linear interpolation shall be permitted to be used to determine values of C_G for G between 0.35 and 0.42.

B.6 ASD and LRFD Design Unit Shear Capacities

The LRFD factored unit shear resistance and the ASD allowable unit shear capacity shall be determined in accordance with 4.1.1.

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Commentary to Appendix B

C-B.1 Scope

Requirements for CLT shear walls are based on research that demonstrates adequate adjusted collapse margin ratios using the FEMA P695 methodology (66, 79). CLT shear wall design requirements are intended to produce yielding of nails and metal connectors at CLT panel edges, and combined rocking and sliding behavior of individual wall panels prior to occurrence of the ultimate shear wall strength limit state associated with nailed connection failure. CLT shear walls can be in single panel or multi-panel configurations. Design unit shears are associated with uniform spacing of prescribed connectors at the bottom of the panel, top of the panel and at vertical edges of multi-panel shear walls. Typical single panel and multi-panel wall configurations are shown in Figure C-B.1 and examples of typical connection details are depicted in Table C-B.2. While angle connectors at top and bottom are shown on one face of the CLT wall panel only, it is permissible to place connectors on both faces and for the minimum requirement of two connectors per panel to be on opposite faces of the CLT wall panel. Multi-panel shear walls are formed by individual panels having the same aspect ratio to promote deformation compatibility within the shear wall. The design and detailing requirements produce yielding of the prescribed nailed connections and rocking behavior in the shear wall as depicted in Figure C-B.2 when subjected to in-plane shear forces. Details of Table C-B.2 do not incorporate concrete floor toppings for clarity of illustrating connector requirements for in-plane shear loading and added fastening for out-of-plane loads. A clear space should be provided between such toppings and the vertical and horizontal legs of the connector to avoid inhibiting connector deformation (e.g., bending, tension and rotation) under in-plane shear loading of the shear wall.

C-B.2 Application Requirements

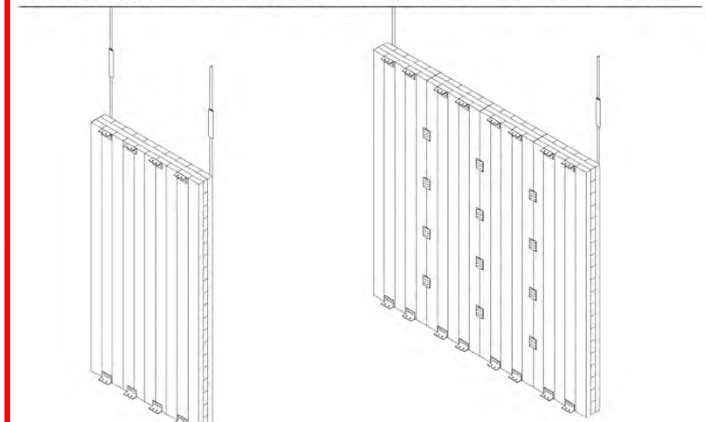
The CLT shear wall lateral force-resisting system (LFRS) is intended for use in platform construction where all individual wall panels are single-story clear-height panels and the CLT floor panels are designed as the floor diaphragm. Elements of the gravity framing system can include but need not be limited to CLT walls, beams and columns, and light-frame walls. The required use of the CLT shear wall LFRS in platform construction precludes application for balloon frame construction associated with multi-story clear-height wall panels. For gable end wall conditions, the requirement for wall panels of the same height necessitates a configuration of wall panels of the same height in the story below the gable end while the "triangular" gable end wall portion can be composed of CLT wall panels or other elements designed as the collector.

The design method requires similar detailing (i.e., minimum panel aspect ratio and shear connections) for all CLT wall panels, whether part of the designated lateral force-resisting system or not, to promote deformation compatibility with the CLT shear wall system up to the point of failure. CLT wall panels that are not part of the designated lateral force-resisting system are expected to be present in addition to the CLT shear walls designated as the vertical elements of the lateral force-resisting system. In general, such added wall elements are considered to reduce in-plane shear demands on the LFRS and improve the strength and stiffness of the building as a whole, much like the presence of sheathed walls in excess of the designed shear walls in a sheathed wood-frame shear wall structure. However, such added wall elements may produce adverse effects on the structural system response that must also be considered in design of the structural system, including but not limited to the distribution of forces and load path to elements of the structural system, which may require strengthening relative to a design that does not account for the interaction with CLT wall panels that are not designated as shear walls. In addition, consideration must be given to the potential for CLT wall panels not designated as shear walls to create structural irregularities such as a weak story irregularity, torsional irregularity, in-plane discontinuity in vertical lateral force-resisting element irregularity, and out-of-plane offset irregularity.

A suggested method for evaluating the structure for the presence of ASCE 7 structural irregularities involves consideration of two separate cases representing bounding values of strength and stiffness contributed by the CLT panels that are not part of the designated seismic force resisting system (SFRS): (1) considering elements that are part of the designated SFRS alone, and (2) considering elements not part of the designated SFRS in combination with the elements that are part of the designated SFRS. Per this method, the bounding values of strength for CLT panels that are designated as not part

of the SFRS range from a minimum of zero in Case 1 to a maximum equal to shear wall strength associated with full overturning restraint in Case 2. For some structural irregularities, placement criteria, rather than structural distribution of strength and stiffness, can trigger an irregularity (e.g., out-of-plane offset, in-plane discontinuity and non-parallel system irregularity). Such irregularities can be triggered for CLT wall panels whether part of the designated SFRS or not part of the designated SFRS.

Figure C-B.1 Typical Shear Wall for a) Single Panel Configuration, and b) Multi-panel Configuration



CLT wall panels with the prescribed nailed connectors are expected to contribute strength and stiffness over the full range of displacement expected of the CLT shear walls as seen in testing results of CLT shear walls with similar nailed connectors with and without hold-downs (65). The extent to which CLT wall panels that are not part of the designated LFRS add strength and stiffness depends on the level of overturning restraint provided to the individual wall panels through dead load and overturning restraint by surrounding elements. The design requirements conservatively prescribe that the strength and stiffness contribution of such walls, for purposes of determining their adverse effect on the structural system, be taken as equal to a shear wall with full overturning restraint provided at wall ends. It is recognized that alternative design approaches may entail detailing of CLT wall panels to either isolate them from resisting in-plane shear forces or to minimize their resistance to in-plane shear forces (such as through use of slotted holes to promote sliding, rocking, or both) while also providing equivalent deformation capacity to CLT wall panels with the prescribed in-plane shear connections. Where such alternative approaches are used, effects of the alternative design for CLT wall panels that are not part of the LFRS (including those associated with strength and stiffness of the alternative design elements) must be considered in the design of the structural system.

Loads are distributed to shear walls within the wall line based on the stiffness determined using SDPWS Equation B-1 for each shear wall within the wall line. For distribution of shear to vertical elements of the LFRS, a diaphragm can be idealized as flexible, idealized as rigid, or modeled as semi-rigid in accordance with the requirements of the ASCE 7 12.3 and SDPWS 4.1.7.

C-B.3 CLT Shear Wall Requirements

CLT shear wall requirements include use of CLT panels of prescribed aspect ratios; use of prescribed nailed connectors at bottoms of panels, tops of panels, and adjoining vertical edge(s) of multi-panel shear walls; a minimum required capacity for overturning tension devices; and compression zone length requirements.

The prescribed angle connectors at the top and bottom of panels and nailed plate connectors at the adjoining vertical edge of multi-panel shear walls have been evaluated under fully-reversed cyclic testing of shear walls and should not be modified or substituted without verification of equivalent shear wall performance by cyclic testing of shear walls that evaluates simultaneous uplift and shear loading of the connectors. For the prescribed angle connector, observed failure from shear wall testing was due to combined nail bending, nail withdrawal from the wood, and limited occurrence of combined bending/tension failure of the nail, without metal connector failure. The prescribed connectors for the vertical edge provide equivalent shear capacity to that of the angle connectors at the top and bottom of the CLT shear wall. The prescribed connectors have been tested both as part of a shear wall and as individual components under uplift loading and shear loading. Testing employed bolts in the horizontal leg of the connectors. Lag screws are prescribed as an alternative with strength in shear and uplift capable of developing the tested strength of the nailed connector.

Design of CLT shear walls and associated load paths is in accordance with the basic load combinations of ASCE 7 (load combinations without overstrength) except where otherwise required by this standard. Hold-down requirements are intended for two common hold-down systems – continuous tie-rod systems and conventional hold-down devices. For both, the required design for 2 times the forces associated with the design unit shear capacity of the CLT shear wall is consistent with level of overstrength in the hold-down system in CLT shear wall testing and is intended to ensure shear strength in excess of the specified nominal strength of the shear connections can be developed. A device elongation limit of 0.185 inches for strength design is required to be met at each story level to avoid concentration of device elongation in one level and is based on consideration of ICC-ES evaluation criteria limits for conventional hold-down devices attached to wood members and continuous rod tie-down systems used to resist wind uplift in light-frame wood walls (82, 83).

Figure C-B.2 Rotation of Individual Panels in a CLT Shear Wall

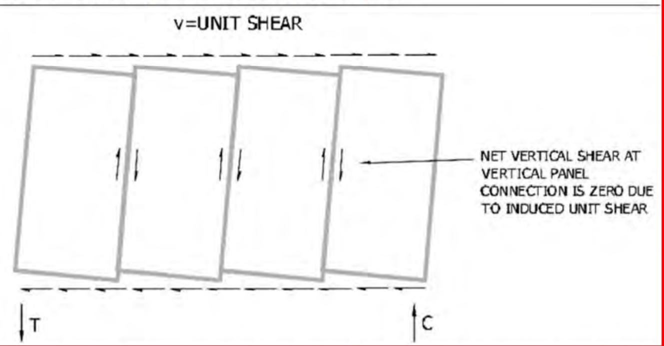
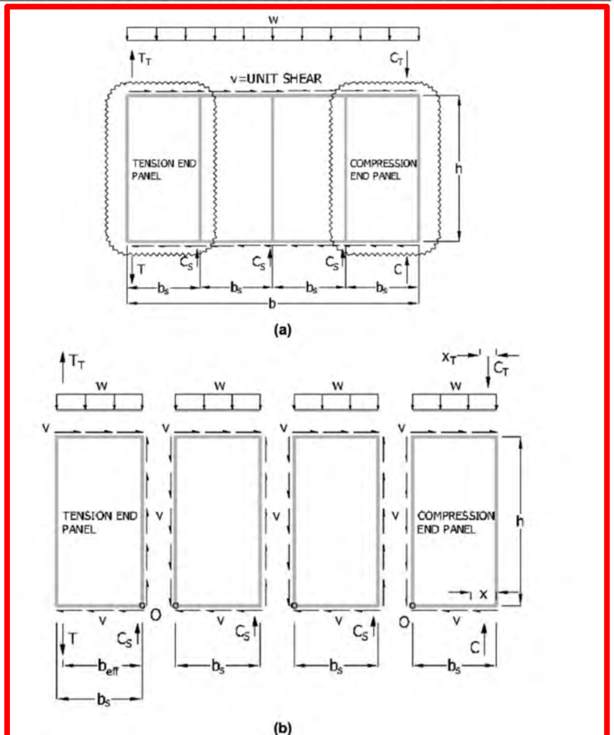


Figure C-B.3 Combined Shear and Gravity Loading and Geometry for CLT Shear Wall Composed of Multiple CLT Panels (e.g., Multi-panel Shear Wall), a) Compression End and Tension End Panel Circled, and b) Illustration of Individual CLT Panel Overturning and Opposing Internal Shears at Adjacent Vertical Edges Due to In-plane Unit Shear Loading



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Under in-plane unit shear loading, individual CLT panels within a CLT shear wall designed and detailed in accordance with Appendix B will rotate as shown in Figure C-B.2. For purposes of determining tension force, T, and compression force, C, static equilibrium is based on consideration of the tension end panel and compression end panel depicted in Figure C-B.3. Consistent with individual panel rotation behavior as opposed to overturning as a rigid body whole, the static analysis of individual panels is employed in determination of T and C forces. The contribution of dead load in the overturning design is specifically limited to only that dead load tributary to the individual panel and to elements aligned directly above the panel of interest per Figure C-B.3. The dead load includes reactions from headers, beams, and similar elements when they are supported by the panel of interest. Vertical load reactions from floors above are to be applied to each panel of interest as C and T reactions for end panels and C reactions for interior panels, as applicable. As a result of this assumption of individual panel overturning, the overturning induced tension force is larger and overturning induced compression force is smaller than T and C forces associated with overturning the wall as a rigid monolith, primarily because the static analysis does not assume distributed gravity loading over the length of the wall can be mobilized via a whole-wall rigid body assumption to reduce the T force or increase the C force.

For the tension end panel depicted in Figure C-B.3, the tension force from summation of moment about point O is:

$$\sum M_o = 0$$

$$T\,(b_{eff}) - vb_sh + wb_s\left(\frac{b_s}{2}\right) - T_T(b_{eff}) = 0 \qquad \text{(C-B.1)}$$

$$T = \frac{vb_sh - wb_s\left(\frac{b_s}{2}\right)}{b_{eff}} + T_T \qquad \text{(C-B.2)}$$

where:

- v = unit shear, plf
- w = dead load including wall panel self-weight, plf
- b_s = CLT panel length, ft
- b_{eff} = moment arm for calculation of T force, ft
- h = CLT panel height, ft
- T = tension force, lb
- T_T = tension force at top of tension end panel from story above, lb

For the compression end panel depicted in Figure C-B.3, the compression force from summation of moment about O is:

$$\sum M_o = 0$$

$$C\left(b_s - \frac{x}{2}\right) - vb_sh - wb_s\left(\frac{b_s}{2}\right) - C_T\left(b_s - \frac{x_T}{2}\right) = 0 \qquad \text{(C-B.3)}$$

where:

- v = unit shear, plf
- w = dead load including wall panel self-weight, plf
- b_s = CLT panel length, ft
- h = CLT panel height, ft
- C = compression force, lb
- x = length of compression zone, ft
- C_T = compression force at top of compression end panel from story above, lb
- x_T = length of compression zone at top of compression end panel from story above, ft

The compression force associated with the length of the compression zone, assuming a uniform stress distribution (i.e., rectangular stress distribution) in the compression zone, is determined by the following equations:

$$C = F_{c\perp}'\,(t)(x)\left(\frac{12\text{in.}}{\text{ft}}\right) \qquad \text{(C-B.4)}$$

$$C = F_c'(t_{parallel})(x)\left(\frac{12\text{in.}}{\text{ft}}\right) \qquad \text{(C-B.5)}$$

where:

- x = length of compression zone, ft
- C = compression force, lb
- $F_{c\perp}'$ = bearing stress perpendicular to grain, psi
- F_c' = design axial stress parallel to grain, psi
- t = CLT wall panel thickness, in.
- $t_{parallel}$ = thickness of CLT wall panel layers oriented parallel to grain for determination of wall axial capacity, in.
- 12in./ft = conversion of compression zone length in feet to inches

The bearing resistance in Equation C-B.4 is based on the compression perpendicular to grain stress associated with the CLT floor panel or wood bottom plate supporting the wall. The bearing resistance in Equation C-B.5 is based on the design axial stress associated with the CLT wall panel parallel to grain layers and is applicable where designed compression zone elements are used to transfer

such forces as opposed to being limited by compression perpendicular to grain bearing stress in the floor panel. The designed compression zone force transfer detail through the floor panel is likely to be used in cases with a combination of high axial compression loads and high aspect ratio panels for the purpose of limiting wall thickness increases associated with meeting compression zone length requirements.

The length of the uniform stress compression zone, x, to satisfy static equilibrium is determined by substitution of Equation C-B.4 or C-B.5 into Equation C-B.3 and solving for x. The solution for x limited by bearing stress perpendicular to grain is provided in Equation C-B.6.

$$x = \frac{-ab_s + \sqrt{(ab_s)^2 - 2a\left[vb_sh + \frac{wb_s^2}{2} + C_T\left(b_s - \frac{x_T}{2}\right)\right]}}{-a} \qquad \text{(C-B.6)}$$

where:

$$a = 12F_{c\perp}'\,t$$

Where the length of the compression zone, x, is smaller than the length of the compression end panel, b_s, a positive value under the root in Equation C-B.6 is produced and the resulting value of x can be used to determine a precise value of compression force, C, in accordance with Equation C-B.3. A negative value under the root of Equation C-B.6 signifies the compression zone is not contained within the compression end panel. A preliminary check for whether adequate compression panel length is provided under unit shear loading alone (e.g., w = C_T = 0) can be obtained from Equation C-B.7. When Equation C-B.7 is not satisfied, a negative root will occur in Equation C-B.6 indicating inadequate compression panel length.

$$v \le \frac{6F_{c\perp}'t}{h/b_s} \qquad \text{(C-B.7)}$$

The loading and geometry depicted in Figure C-B.3 for tension end and compression end panels are for purposes of illustrating a method to calculate an appropriate T and C force for the system. Testing shows rotation of the compression end panel is primarily about the outermost edge of the compression end panel – not about the centroid of the calculated compression zone. As such, using the loading and geometry depicted in Figure C-B.3 for the compression end panel will underestimate the moment arm and overestimate vertical edge forces when summing forces vertically at that location. Results of such analysis should not be used to modify required vertical connector spacing in accordance with SDPWS B.3.3 which requires the same average vertical connector spacing with rounding as used for the top and bottom edges of the CLT shear wall. The required number of connections at vertical and horizontal edges (i.e., the same average spacing) provides balanced vertical and horizontal shear and enables the intended rotation behavior of individual panels of a multi-panel shear wall when subjected to in-plane unit shear loading.

C-B.3.6 Other Load Path Connections to CLT

Load path connections to CLT wall panels occur in addition to those of the designated lateral force-resisting system for in-plane shear resistance and include connections for out-of-plane wind and seismic forces and general structural integrity. These additional load path connections include attachment of wall panels to elements above and below for out-of-plane forces, interconnection of walls at intersections, and attachment of conventional hold-down devices at wall ends.

The combined requirement for fastener yielding per Mode III_s or IV and compliance with NDS Appendix E ensures that added fastening provides a predictable yielding mechanism with levels of overstrength similar to that provided by the prescribed connections for in-plane shear resistance. Connections at the top and bottom of wall for resistance to out-of-plane forces are in addition to prescribed angle connectors which do not have an established design value for loads perpendicular to the plane of the wall. Connections at the top and bottom of wall meeting requirements for yielding in Mode III_s or IV are considered beneficial to in-plane shear wall strength and stiffness, which is already governed by nail yielding, without degrading peak load and post-peak response of the prescribed shear wall connectors.

To address the potential for excessive screw (e.g., wood screw and lag screw) attachment to inhibit the rocking mechanism of the CLT panel due to high axial stiffness and strength of screws loaded in withdrawal, screw attachment of top and bottom of wall connections to supporting elements is not permitted. Screws used in these locations are considered an alternative to the prescribed smooth shank dowel fasteners (see Table C-B.2 Typical connection details) at the top and bottom of wall locations and are subject to approval by the authority having jurisdiction. Typical fastening will employ smooth shank nails or pins to resist out-of-plane forces. Details for anchoring the top and bottom of walls for out-of-plane forces are not specifically prescribed to enable varying design options for meeting out-of-plane anchor-

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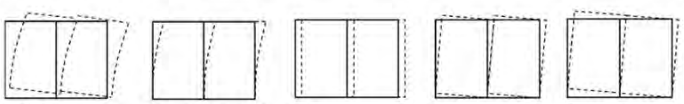
C-B.3.7 CLT Shear Walls with Shear Resistance Provided by High Aspect Ratio Panels Only

CLT shear walls with shear resistance provided by high aspect ratio panels only is a specific configuration of the CLT shear wall system where high aspect ratio is defined as wall panel height to wall panel length ratio of 4. Minor variations in actual panel aspect ratio of +/- 2.5 percent are permissible such that actual panel aspect ratio range is 3.9 to 4.1. All requirements applicable for CLT shear walls are also applicable and additionally only CLT panels with aspect ratio of 4 are permissible as part of the designated shear wall system. CLT wall panels of equal or greater aspect ratio are permissible when not used as part of the designated shear wall system to promote deformation compatibility of CLT wall panels that are not designated as shear walls. Where the system used is "CLT shear walls with shear resistance provided by high aspect ratio panels only", it is required that the aspect ratio requirement be met in all shear walls. While this limitation can be accommodated in single story and multi-story construction with equal story height it may not be practical to implement where story height varies. In such cases, use of CLT shear walls with a permissible range in aspect ratio from 2 to 4 should be considered.

C-B.4 Shear Wall Deflection

The CLT shear wall deflection equation incorporates four primary components: individual wall panel bending, individual wall panel shear, sliding, and rigid body overturning. Individual panel rotation is included for multi-panel configurations. The deflection method accounts for the difference in observed stiffness of single and multi-panel CLT shear walls tested as well as influence of individual panel aspect ratio on shear wall deflection. The equation does not account for potential stiffening effects of boundary elements such as intersecting wall, floor, and roof elements. Components of shear wall deflection are depicted in Figure C-B.4.

Figure C-B.4 Shear Wall Deflection Components Due to Panel Bending and Shear, Sliding Due to Fastener Slip, Rotation due to Fastener Slip at Vertical Edge Connections, and Rigid Body Rotation



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COMMENTARY TO APPENDIX B

The bending term in the deflection equation is simplified from $vb_sh^3/(3 * (EI)_{eff})$ for a cantilever with point load to $576vb_sh^3/(EI)_{eff}$ to account for the unit conversion so that $(EI)_{eff}$ can be in lb-in^2 and other units can be in feet. $(EI)_{eff}$ is the effective in-plane panel stiffness for bending to account for composite behavior between adjacent parallel laminations where transverse E is approximated as longitudinal E/30. $(EI)_{eff}$ can be calculated directly using transformed section properties in accordance with Equation C-B.8.

$$(EI)_{eff} = \left(\frac{{b_s}^3}{12}\right)\left(E_{0,L}\,\Sigma t_{L,i} + E_{90,T}\Sigma t_{T,i}\right) \quad \text{(C-B.8)}$$

where:

b_s = CLT panel length
$E_{0,L}$ = modulus of elasticity parallel to the grain for longitudinal layers (i.e., longitudinal E)
$E_{90,T}$ = modulus of elasticity perpendicular to the grain for transverse layers (i.e., transverse E taken as longitudinal E/30)
$t_{L,i}$ = individual longitudinal lamination thickness
$t_{T,i}$ = individual transverse lamination thickness

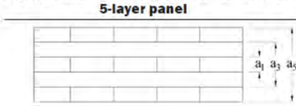
An alternative formulation for $(EI)_{eff}$ for a 5-layer panel with equal thickness layers is adapted from Blass and Fellmoser (63), see Equation C-B.9:

$$(EI)_{eff} = E_{0,L}\left(\frac{{b_s}^3 a_5}{12}\right)\left[1-\left(1-\frac{E_{90,T}}{E_{0,L}}\right)\left(\frac{a_3 a_1}{a_5}\right)\right] \quad \text{(C-B.9)}$$

where:

a_1, a_3, a_5 = thickness as shown in the Figure C-B.5 for a 5-layer panel (i.e., 3 longitudinal layers and 2 transverse layers).

Figure C-B.5 Illustration of a_1, a_3, and a_5 for a 5-layer panel



The panel shear deformation term utilizes inplane shear stiffness, $GA_{eff\,(in\text{-}plane)}$, in units of pounds per inch (lb/in). Example values of $GA_{eff\,(in\text{-}plane)}$ provided in Table C-B.1 are calculated in accordance with Flaig M. and Blass H. (64) per Equation C-B.10 and Equation C-B.11.

$$G_{eff,CA} = \frac{Kb^2}{5}\cdot\frac{n_{CA}}{t_{gross}}\cdot\frac{m^2}{(m^2+1)} \quad \text{(C-B.10)}$$

$$G_{eff,CLT} = \left(\frac{1}{G_{lam}}+\frac{1}{G_{eff,CA}}\right)^{-1} \quad \text{(C-B.11)}$$

where:

K = slip modulus of crossing areas (Use K = 4.0 N/mm^3 = 14,735 lb/in.^{3})
b = width of lamella
m = number of longitudinal lamellae
n_{CA} = number of glue lines within CLT cross section thickness
t_{gross} = CLT cross section thickness
G_{lam} = individual lamination shear modulus, psi (Use longitudinal E/16)

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Table C-B.1 Example $GA_{eff\,(in\text{-}plane)}$ for in-plane shear[1]

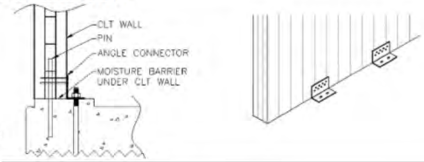
	Number of Layers			
	3	5	7	9
G_{eff}	28,700 psi	32,300 psi	33,700 psi	34,500 psi
$GA_{eff\,(in\text{-}plane)}$	118,300 lb/in.	222,000 lb/in.	324,500 lb/in.	426,600 lb/in.

1. Calculated values of G_{eff} and $GA_{eff\,(in\text{-}plane)}$ are based on use of Equations C-B.10 and C-B.11 with the following inputs: E=1,400000 psi, lamella width b = 5.5 in., number of longitudinal lamellae based on a 48 in. panel width, slip modulus of crossing areas K = 14,735 lb/in.^{3} (4.0 N/mm^3) and lamella thickness =1.375 in.

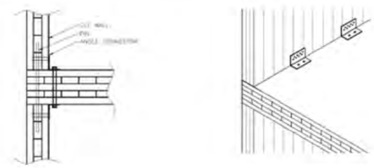
The sliding term, $V_{(nail\ load)}/(135{,}000\ D^{1.5})$, addresses sources of deformation in the connector including nails and bolts. The slip constant takes into account loading perpendicular to the grain in the nailed connection. The single nail diameter of 0.135 in. used for all the connectors in this study, allows the use of a simplified nail slip term, $V_{(nail\ load)}/(6700)$. The deflection equation also explicitly breaks out sliding from multi-panel rotation due to vertical connection slip. If there is no vertical edge connection, then vertical connection slip equals 0 inches. The final term in the deflection equation represents rigid body rotation about the compression toe of the shear wall and is the same as used for sheathed wood-frame shear walls. Vertical deformation of the wall hold-down system, Δ_a, is based on the induced overturning forces and includes sources of deflection such as fastener slip, device elongation, rod elongation, uncompensated shrinkage, and vertical compression deformation.

Table C-B.2 Typical Connection Details

Wall panel to foundation connection



Exterior wall to floor connection



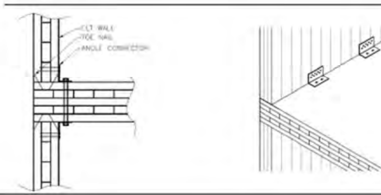


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Table C-B.2 Typical Connection Details (continued)

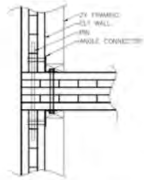
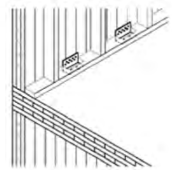
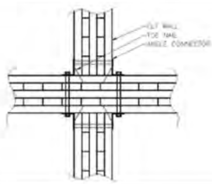
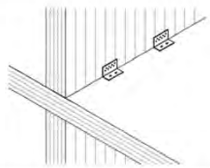
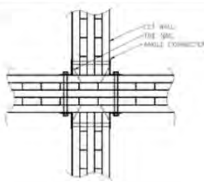
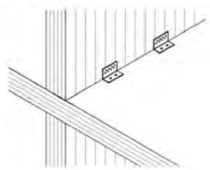
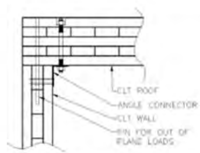
Exterior wall to floor connection (example with interior framed wall)		
Interior wall to floor connection		
		
Exterior wall to roof		

Table C-B.2 Typical Connection Details (continued)

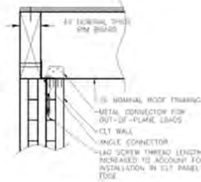
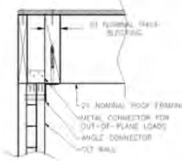
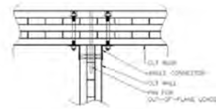
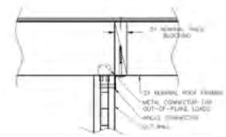
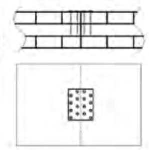
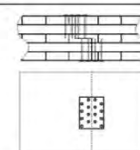
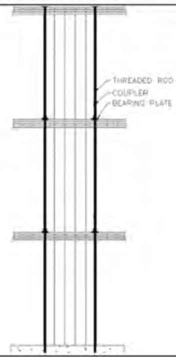
Exterior wall to roof		
Interior wall to roof		
Wall panel to wall panel connection, square edge configuration (i.e., vertical edge connection)		
Wall panel to wall panel connection, lap configuration (i.e., vertical edge connection)		

Table C-B.2 Typical Connection Details (continued)

Multi-story overturning restraint



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6.3 CLT Shear Wall Example Description

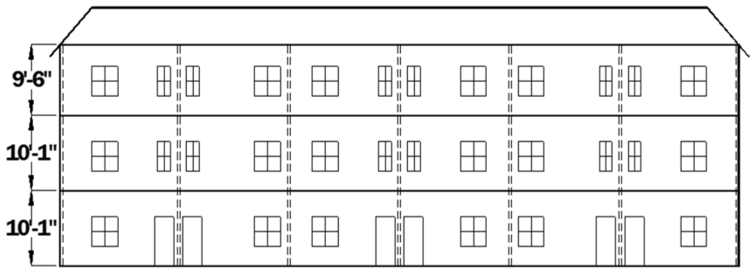


Figure 6-2. Elevation

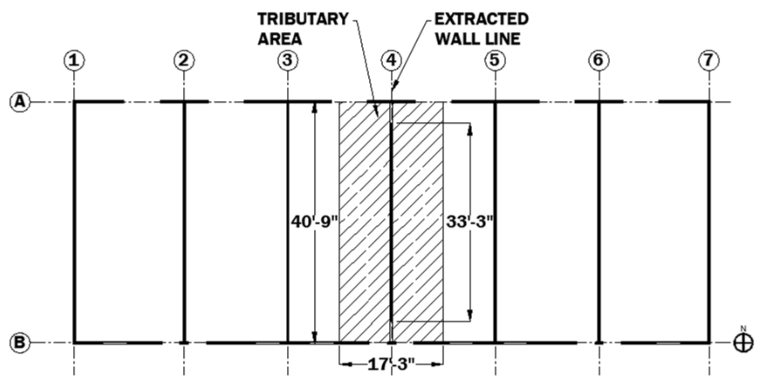


Figure 6-3. Typical Floor Plan (first story openings shown)

- A three-story, six-unit townhouse cross-laminated timber building of platform construction
- The CLT shear wall design in this example includes:
  - Check of CLT shear wall shear strength
  - Check of CLT shear wall hold-down size and compression zone length for overturning
  - Check of CLT shear wall deflection for conformance to seismic drift

6.3 CLT Shear Wall Example Description

Table 6-1: Weights of Roof/Ceiling, Floors, and Walls

Item	Description	Weight
Roof/Ceiling	Light-frame roof, gypsum board ceiling, roofing, insulation	25 psf
Floor	5-layer CLT (6.875 in. thick), gypsum board ceiling, flooring. Includes 8 psf of floor area for wall partitions	35 psf
Interior Walls	3-layer CLT (4.125 in. thick), light-frame wall, gypsum board finish, sound insulation	20 psf
Exterior Walls	3-layer CLT (4.125 in. thick), light-frame wall, gypsum board interior finish, stucco exterior, insulation	30 psf

6.4 Seismic Forces

Seismic base shear calculation assumptions:

- S_{DS} = 1.0
- I_e = 1.0
- R = 3 (for *CLT shear walls)*

Seismic base shear, V, per ASCE/SEI 7-22 Equation 12.8-2 (for short-period structures):

$$V = C_s W = \frac{S_{DS}}{(R/I)} W = \frac{1.0}{(3.0/1.0)} W = 0.333\, W\ kips$$

The portion of base shear tributary to the CLT shear walls of interest is:

$V_{(Line\ 4)}$ = 42.3 kips

6.4 Seismic Forces

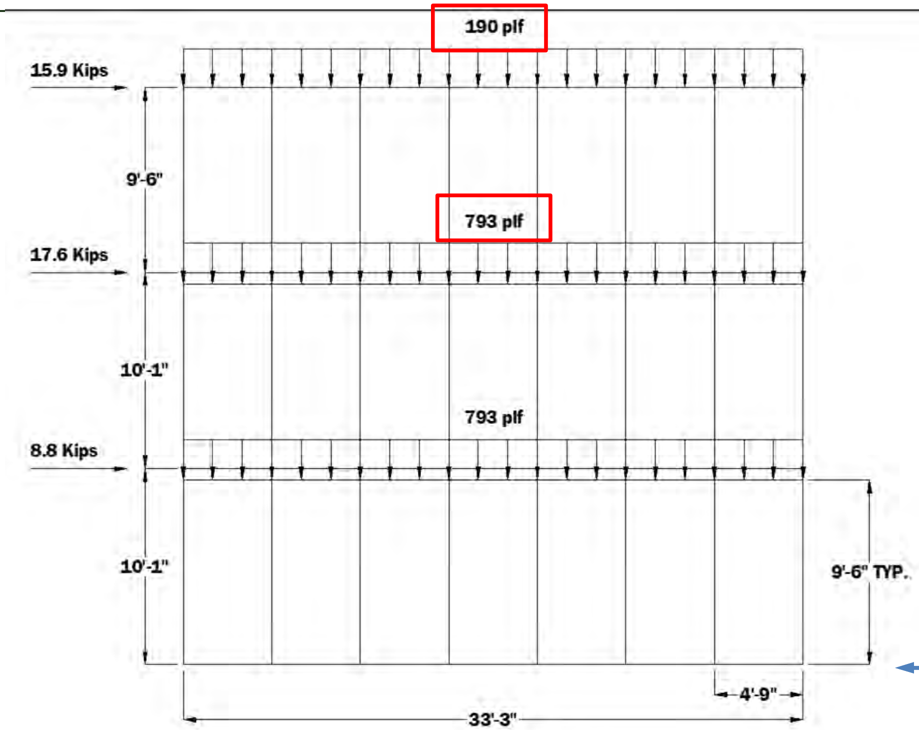


Figure 6-4. Vertical Distribution of Seismic Force and Dead Load Tributary to the CLT Shear Walls Located Along Line 4

$V_{(Line\ 4)}$ = 42.3 kips

Table 6-4: Summary of Cumulative Lateral Seismic Force and Unit Shear Force per Story (Along Line 4)

Story	Lateral force, V_x	Unit Shear Force per Foot of Shear Wall Length
	(kips)	(plf)
3	15.9	477
2	33.5	1,009
1	42.3	1,273

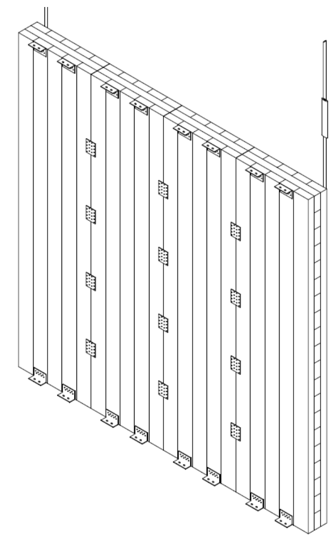
6.5.1 Shear Capacity of Prescribed Connectors

LRFD design unit shear capacity for seismic:

$$v_{\mathrm{s(seismic)}} = \phi(n)\left(\frac{2605}{b_s}\right)C_G \qquad \text{(SDPWS-21 Eq. B-2)}$$

where:

- n = number of angle connectors along bottom of panel face
- 2,605 = connector nominal shear capacity (lb)
- b_s = individual CLT panel length (ft)
- C_G = CLT panel specific gravity factor which equals 1.0 for $G \geq 0.42$ specific gravity panels used in this example, and
- ϕ = resistance factor equal to 0.5 for seismic design



From SDPWS Figure C-B.1

6.5.1 Shear Capacity of Prescribed Connectors

Table 6-5: CLT Shear Wall Connectors and LRFD Design Unit Shear Capacity

Story	Panel thick-ness	Panel length, b_s	Panel height, h	Number of connectors per panel at top and bottom panel edge	Number of connectors at each adjoining vertical panel edge	v_n, Nominal unit shear capacity, $(n \cdot 2605)/b_s$	$v_{s(seismic)}$, LRFD design unit shear capacity, (ϕ= 0.5)
	(in.)	(ft)	(ft)	(n)	(n x h/b_s)	(plf)	(plf)
3	4.125	4.75	9.5	2	4	1,096	548
2	4.125	4.75	9.5	4	8	2,193	1,097
1	4.125	4.75	9.5	5	10	2,742	1,371

6.5.1 Shear Capacity of Prescribed Connectors

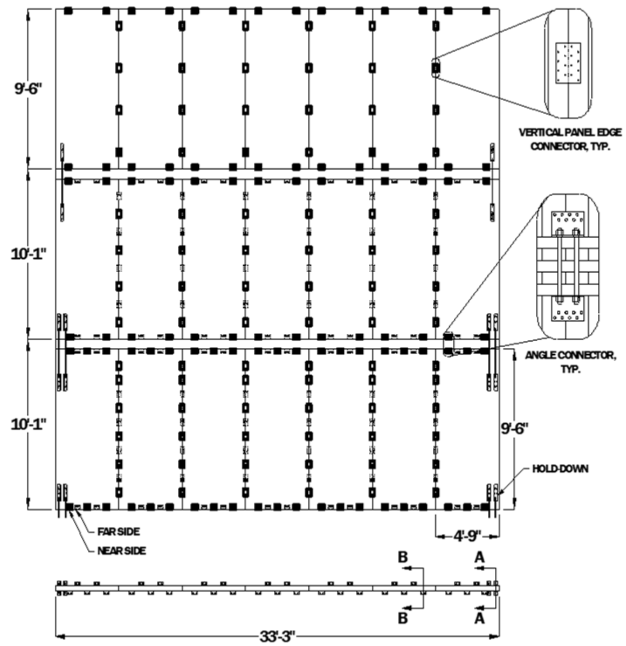


Figure 6-5. CLT Shear Walls at 1st, 2nd, and 3rd Story with Connector And Hold-down Location

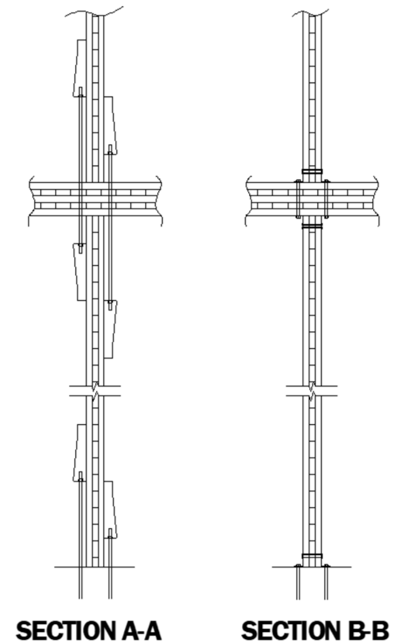


Figure 6-6. Wall-floor Intersections

6.5.2 Shear Capacity of CLT Panel

For this 3-layer E1 grade CLT panel, the allowable stress design (ASD) in-plane shear unit shear capacity is converted to LRFD using NDS-2018 Table 10.3.1:

$$v_r' = \phi\lambda K_F F_v(t_v) = 0.75(1.0)(2.88)(9,700) = 20,849\ plf$$

where:

$F_v(t_v)$= 9,700 plf (ASD value from CLT panel manufacturer's evaluation report)

CLT panel in-plane unit shear capacity, $v_r' =$ 20,849 plf is greater than the largest unit shear force story demand of 1,273 plf (from Table 6-4)

$$20,849\ plf \gg 1,273\ plf$$

In-plane unit shear capacity value does not to account for holes, cuts or other modifications

6.6.1 CLT Shear Wall Hold-down Design

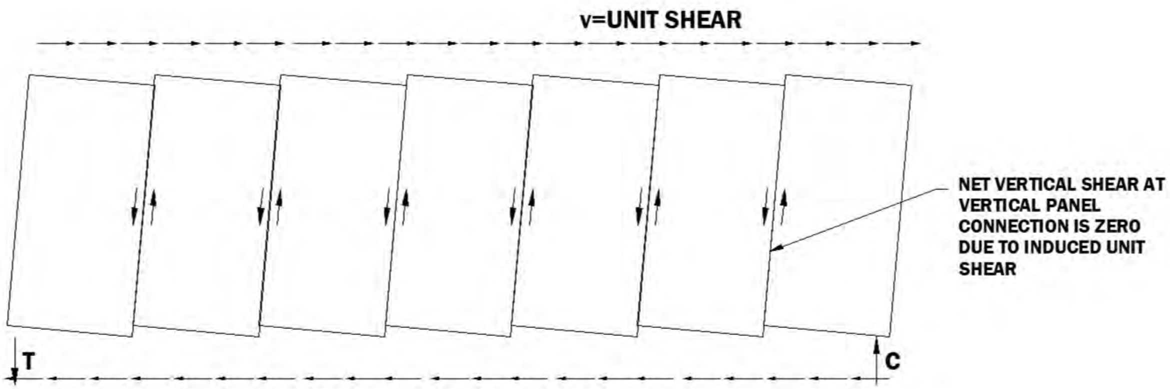


Figure 6-1. Illustration of Rocking Behavior of Seven Individual Panels in A Multi-panel CLT Shear Wall Designed in Accordance with SDPWS-21 Appendix B

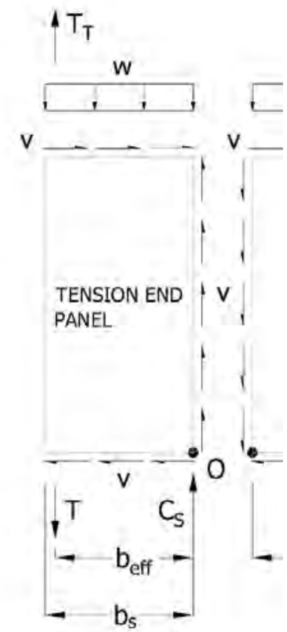


Figure 6-7. Free-body Diagram for the Tension End Panel of the CLT Multi-panel Shear Wall

6.6.1 CLT Shear Wall Hold-down Design

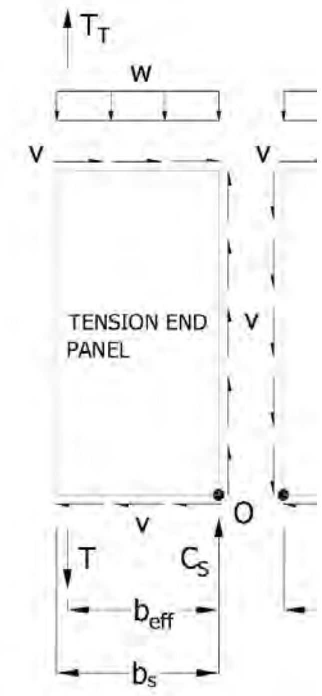
$$\sum M_o = 0$$

$$T\,(b_{eff}) - v b_s h + w b_s \left(\frac{b_s}{2}\right) - T_T(b_{eff}) = 0$$ (SDPWS-21 Eq. C-B.1)

$$T = \frac{v b_s h - w b_s\left(\frac{b_s}{2}\right)}{b_{eff}} + T_T$$ (SDPWS-21 Eq. C-B.2)

Table 6-6: Solution for Tension Force, T, for Hold-down Strength Requirement

Story	Unit shear force per foot of shear wall length	$v_{s(seismic)}$, LRFD design unit shear capacity, (ϕ= 0.5)	2 x $v_{s(seismic)}$	T_T from story above	T for 2 x $v_{s(seismic)}$ requirement for load combination 1.0E - 0.7D
	(plf)	(plf)	(plf)	(lb)	(lb)
3	477	548	1,097	0	11,293
2	1,009	1,097	2,194	11,293	34,540
1	1,273	1,371	2,742	34,540	63,968



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6.6.1 CLT Shear Wall Hold-down Design

The same screw attached hold-down is used for all locations with each having an LRFD design tension capacity of 17,678 lb and associated deflection of 0.253 in.

- **1st story walls to foundation, four hold-downs**

  4 x 17,687 lb = 70,748 lb > 63,968 lbs

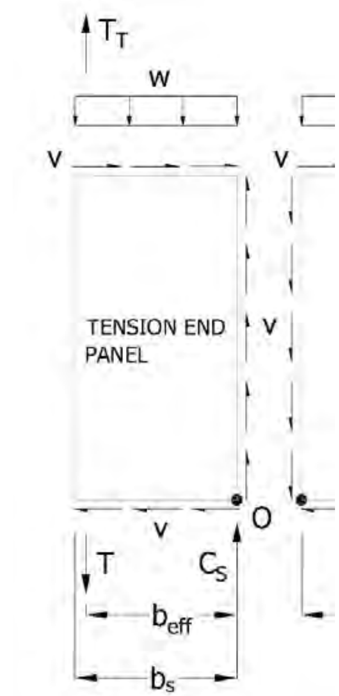
- **2nd story to top of 1st story, four hold-downs**

  4 x 17,687 lb = 70,748 lb > 34,540 lbs

- **3rd story to top of 2nd story, two hold-downs**

  2 x 17,687 lb = 37,374 lb > 11,293 lbs

Check CLT panel for tension, row and group tear out



6.6.1 CLT Shear Wall Hold-down Design

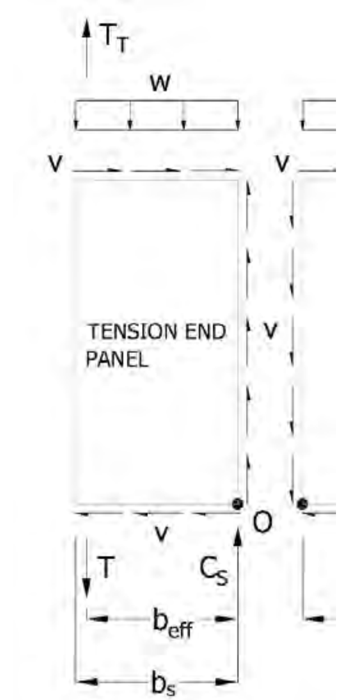
From **SDPWS-21 Section B.3.4**, hold-down device deformation for each story shall not exceed 0.185 in. for T forces from strength design load combinations (see Table 6-7).

Table 6-7: Solution for Tension Force, T, for Hold-down Deflection Requirement

Story	Unit shear force per foot of shear wall length	T for load combination 1.0E - 0.7D
	(plf)	(lb)
3	477	4,714
2	1,009	14,604
1	1,273	27,472

Deflection of most highly loaded hold-down is less than 0.185 in. The SDPWS-21 deflection limit is satisfied

$$\Delta_{hold-down} = \frac{27{,}472\ lb}{4(17{,}678\ lb)}(0.253\ in.) = 0.098\ in. < 0.185\ in.$$



6.6.2 CLT Shear Wall Compression Zone

Compression force, C, and length of compression zone, x, from compression end panel moment equilibrium

$$\sum M_o = 0$$

$$C\left(b_s - \frac{x}{2}\right) - vb_s h - wb_s\left(\frac{b_s}{2}\right) - C_T\left(b_s - \frac{x_T}{2}\right) = 0$$ (SDPWS-21 Eq. C-B.3)

$$C = F_{C\perp}{}'\,(t)(\,x)(\frac{12\text{in.}}{\text{ft}})$$ (SDPWS-21 Eq. C-B.4)

$$C = Fc'\left(t_{parallel}\right)(x)(\frac{12\text{in.}}{\text{ft}})$$ (SDPWS-21 Eq. C-B.5)

C and x summarized in Table 6-8

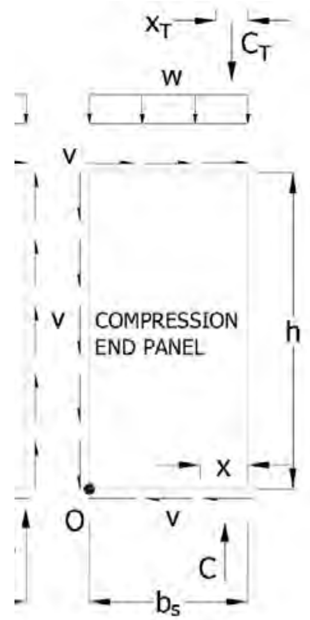


Figure 6-8. Free-body Diagram for the Compression End Panel of the CLT Multi-panel Shear Wall

6.6.2 CLT Shear Wall Compression Zone

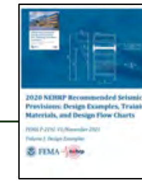
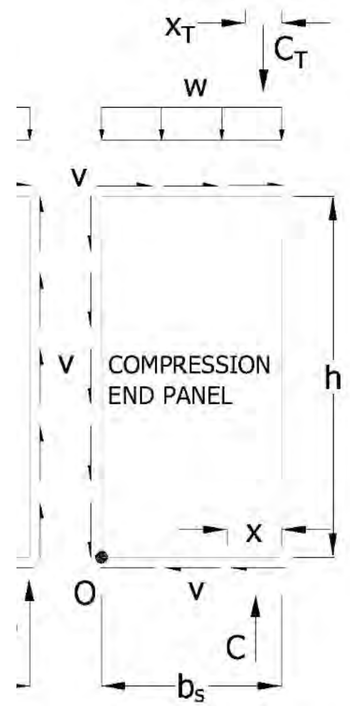


Table 6-8: Solution for Compression Zone Length, x, and Force C

Story	Unit shear force per foot of shear wall length	Dead load, w_{DL}	Live load, w_{LL}	C_T, Compres -sion from top	Compres -sion zone length, x	C, for load combination 1.0E + 1.4D +0.5L
	(plf)	(plf)	(plf)	(lb)	(in.)	(lb)
3	477	190	0	0	2.00	5,257
2	1,009	793	690	5,257	7.64	20,144
1	1,273	793	690	20,144	4.56	36,545



Check of CLT wall panel axial capacity is required

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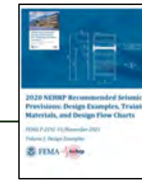
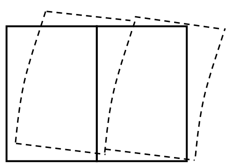
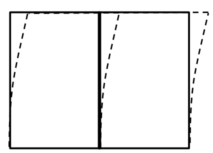
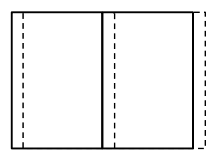
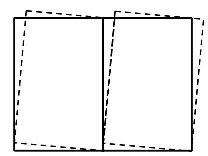
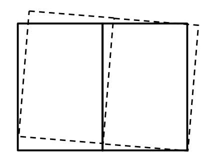
6.7 CLT Shear Wall Deflection



$$\delta_{SW} = \frac{576 v b_s h^3}{EI_{eff(in-plane)}} + \frac{vh}{GA_{eff(in-plane)}} + 3\Delta_{nail\ slip,h} + 2\Delta_{nail\ slip,v}\left(\frac{h}{b_s}\right) + \Delta_a \frac{h}{\sum b_s}$$

(SDPWS-21 Eq. B-1)

Total shear wall = deflection, δ_{SW}	Panel bending and shear +	Sliding +	Panel rotation +	Rigid body rotation
				

6.7 CLT Shear Wall Deflection

Table 6-9: CLT Shear Wall Deflection Components and Total Shear Wall Deflection, δ_{SW}

Story	$\frac{576 v b_s h^3}{EI_{eff(in-plane)}}$	$\frac{vh}{GA_{eff(in-plane)}}$	$3\Delta_{nail\ slip,h} + 2\Delta_{nail\ slip,v}\left(\frac{h}{b_s}\right)$	$\Delta_a \frac{h}{\sum b_s}$	δ_{SW}, shear wall deflection
	(in.)	(in.)	(in.)	(in.)	(in.)
3	0.02	0.04	0.15	0.04	0.24
2	0.04	0.09	0.16	0.04	0.33
1	0.05	0.11	0.16	0.03	0.35

For allowable story drift limit is $2.5\%h$ from ASCE/SEI 7-22 Table 12.12-1, corresponding allowable deflection calculated using, C_d, equal to 3 for cross-laminated timber shear walls:

$$\delta_e = \frac{0.025(h)}{C_d} = \frac{0.025(114\text{ in.})}{3.0} = 0.95\text{ in.} > \delta_{SW}$$

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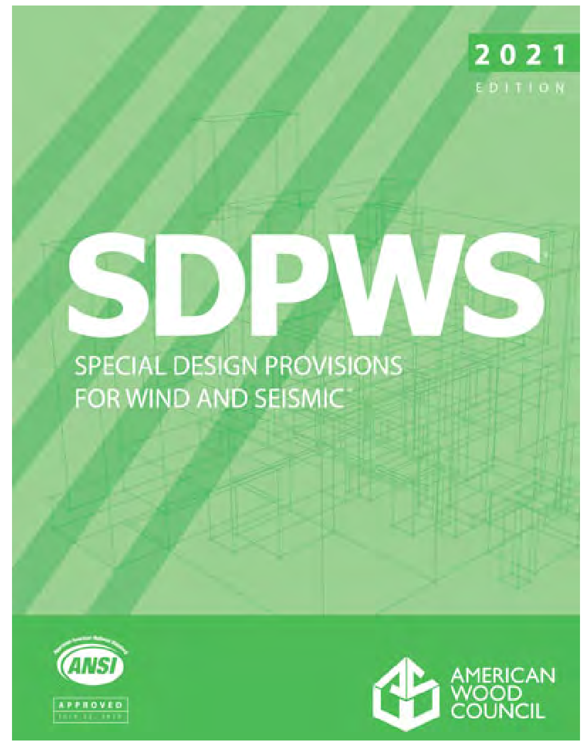
Building Code Reference of SDPWS Standard

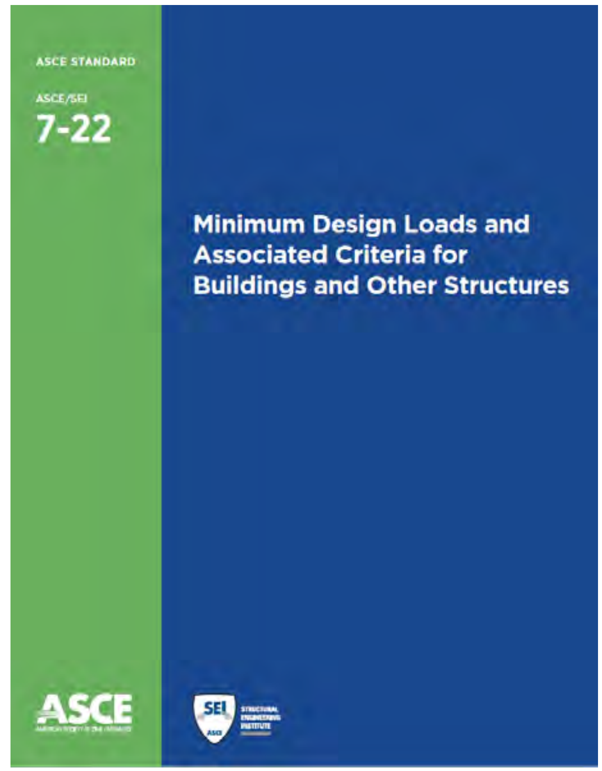
2021 IBC

- References 2021 SDPWS in Section 2305 for lateral design and construction



Building Code Reference of ASCE 7-22 Standard

Referenced in 2024 IBC



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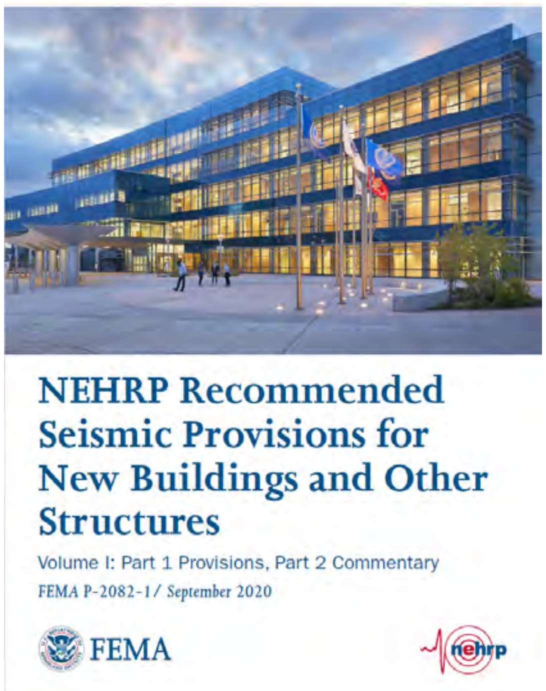
NEHRP Provisions



2020 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA P-2082)

https://www.fema.gov/sites/default/files/2020-10/fema_2020-nehrp-provisions_part-1-and-part-2.pdf

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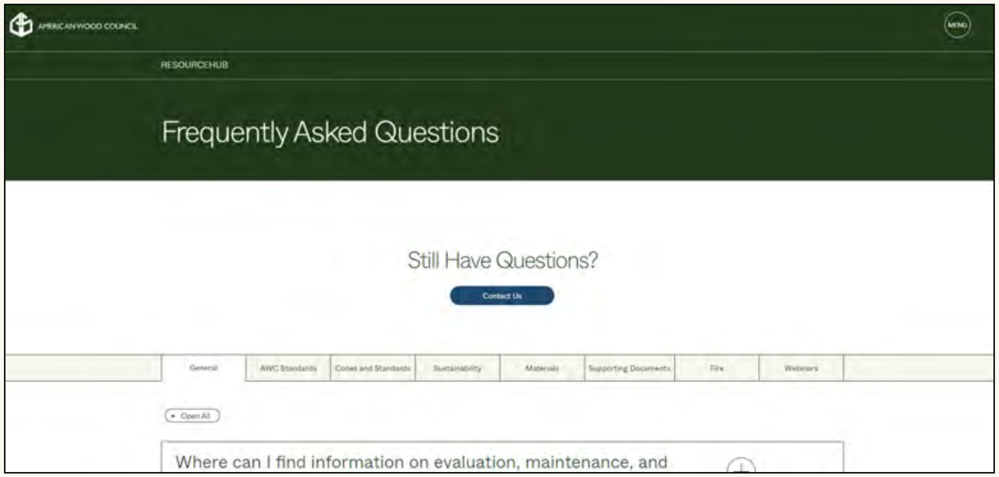
NEHRP Provisions Design Examples

2020 NEHRP Recommended Seismic Provisions: Design Examples, Training Materials, and Design Flow Charts, FEMA P-2192

Volume 1: Design Examples

https://www.fema.gov/sites/default/files/documents/fema_nehrp_design-examples-training-materials_volume-1.pdf

Questions?



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Thank you!

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