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.

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, ( B^a )</th>
<th>Overstrength Factor, ( C_{O}^b )</th>
<th>Deflection Amplification Factor, ( C_{D}^b )</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. BEARING WALL SYSTEMS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Special reinforced concrete shear walls(^{a,d} )</td>
<td>14.2</td>
<td>5</td>
<td>( 2\frac{1}{2} )</td>
<td>5</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>2. Reinforced concrete ductile coupled walls(^c )</td>
<td>14.2</td>
<td>8</td>
<td>( 2\frac{1}{2} )</td>
<td>8</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>3. Ordinary reinforced concrete shear walls(^c )</td>
<td>14.2</td>
<td>4</td>
<td>( 2\frac{1}{2} )</td>
<td>4</td>
<td>NL, NL, NP, NP, NP</td>
</tr>
<tr>
<td>4. Detailed plain concrete shear walls(^c )</td>
<td>14.2</td>
<td>2</td>
<td>( 2\frac{1}{2} )</td>
<td>2</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>5. Ordinary plain concrete shear walls(^c )</td>
<td>14.2</td>
<td>1( \frac{1}{2} )</td>
<td>( 2\frac{1}{2} )</td>
<td>( 1\frac{1}{2} )</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>6. Intermediate precast shear walls(^c )</td>
<td>14.2</td>
<td>4</td>
<td>( 2\frac{1}{2} )</td>
<td>4</td>
<td>NL, NL, 40, 40, 40</td>
</tr>
<tr>
<td>7. Ordinary precast shear walls(^c )</td>
<td>14.2</td>
<td>3</td>
<td>( 2\frac{1}{2} )</td>
<td>3</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>8. Special reinforced masonry shear walls</td>
<td>14.4</td>
<td>5</td>
<td>( 2\frac{1}{2} )</td>
<td>3( \frac{1}{2} )</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>9. Intermediate reinforced masonry shear walls</td>
<td>14.4</td>
<td>3( \frac{1}{2} )</td>
<td>( 2\frac{1}{2} )</td>
<td>2( \frac{1}{2} )</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>10. Ordinary reinforced masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>( 2\frac{1}{2} )</td>
<td>1( \frac{1}{2} )</td>
<td>NL, 160, NP, NP, NP</td>
</tr>
<tr>
<td>11. Detailed plain masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>( 2\frac{1}{2} )</td>
<td>1( \frac{1}{2} )</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>12. Ordinary plain masonry shear walls</td>
<td>14.4</td>
<td>1( \frac{1}{2} )</td>
<td>( 2\frac{1}{2} )</td>
<td>1( \frac{1}{2} )</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>13. Prestressed masonry shear walls</td>
<td>14.4</td>
<td>1( \frac{1}{2} )</td>
<td>( 2\frac{1}{2} )</td>
<td>1( \frac{1}{2} )</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>14. Ordinary reinforced AAC masonry shear walls</td>
<td>14.4</td>
<td>2</td>
<td>( 2\frac{1}{2} )</td>
<td>2</td>
<td>NL, 35, NP, NP, NP</td>
</tr>
<tr>
<td>15. Ordinary plain AAC masonry shear walls</td>
<td>14.4</td>
<td>1( \frac{1}{2} )</td>
<td>( 2\frac{1}{2} )</td>
<td>( 1\frac{1}{2} )</td>
<td>NL, NP, NP, NP, NP</td>
</tr>
<tr>
<td>16. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance</td>
<td>14.5</td>
<td>6( \frac{1}{2} )</td>
<td>3</td>
<td>4</td>
<td>NL, NL, 65, 65, 65</td>
</tr>
<tr>
<td>17. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets</td>
<td>14.1</td>
<td>6( \frac{1}{2} )</td>
<td>3</td>
<td>4</td>
<td>NL, NL, 65, 65, 65</td>
</tr>
<tr>
<td>18. Light-frame walls with shear panels of all other materials and with shear bracing</td>
<td>14.1 and 14.5</td>
<td>2</td>
<td>( 2\frac{1}{2} )</td>
<td>2</td>
<td>NL, NL, 35, NP, NP</td>
</tr>
<tr>
<td>19. Light-frame (cold-formed steel) wall systems using flat strap bracing</td>
<td>14.1</td>
<td>4</td>
<td>2</td>
<td>3( \frac{1}{2} )</td>
<td>NL, NL, 65, 65, 65</td>
</tr>
<tr>
<td>21. Cross-laminated timber shear walls with shear resistance provided by high-aspect-ratio panels only</td>
<td>14.5</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td>65, 65, 65, 65, 65</td>
</tr>
</tbody>
</table>

**B. BUILDING FRAME SYSTEMS**

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
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<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Steel eccentrically braced frames</td>
<td>14.1</td>
<td>8</td>
<td>2</td>
<td>4</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
<tr>
<td>2. Steel special concentrically braced frames</td>
<td>14.1</td>
<td>6</td>
<td>2</td>
<td>5</td>
<td>NL, NL, 160, 160, 100</td>
</tr>
</tbody>
</table>
Background

Cross-laminated timber (CLT)

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

Photo credit: Will Pryce
Background

Photo credit: Will Pryce
Background

Photo credit: Will Pryce
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)**
## 6.2 Background

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Objective</th>
</tr>
</thead>
</table>
| Connector tests                  | • Behavior under cyclic loading  
• Interpanel connector behavior  | ![Connector test diagram]                                                                                                       |
| 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  | ![Isolated wall test diagram]                                                                                                      |
| 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  | ![Full-scale shake table test diagram]                                                                                           |
|                                 | • 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

Note: Scaled results
Background
Background
6.2 Background

[Shake Table Video]
Background

![Diagram of background information]

- **v=UNIT SHEAR**
- **NET VERTICAL SHEAR AT VERTICAL PANEL CONNECTION IS ZERO DUE TO INDUCED UNIT SHEAR**
- **TENSION END PANEL**
- **COMPRESSION END PANEL**
- **T**
- **C**
(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, SDCs 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-framed structures.
   b. CLT floor panels are designed as floor diaphragms to distribute lateral loads to the CLT shear walls.

2. CLT walls shall be classified as either (1) part of the designated LFRS (e.g., CLT shear walls) 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, b/a, that is not less than the aspect ratio used for CLT shear wall panels in accordance with B.3.1 or B.3.2.
   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 adjacent 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 of failure of these elements will not impair the vertical load and seismic force resisting capability of the designated SFRS.

5. 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.

6. Shear distribution to individual CLT shear walls in a wall line shall provide the same calculated deflection at each shear wall (i.e., distribution of loads based on relative stiffnesses).

7. The nominal nail shear capacity assigned to CLT shear walls shall not include contributions from connections other than shear connections prescribed in B.3.2 and B.3.3.

8. ASCE 7 seismic criteria specific to light-frame construction do 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 SDCs 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, b/a, not greater than 4:1, and individual panel length.

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 4.0 in. thickness so that nails from opposing faces do not overlap.

4. Holes, notches, and other modifications to CLT panels shall be permitted unless the effects of removal of material on load carrying capacity of the CLT panel is determined by an approved rational analysis.

5. Shear distribution to individual CLT shear walls in a wall line shall provide the same calculated deflection at each shear wall (i.e., distribution of loads based on relative stiffnesses).

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 not permitted to be designed as Force-Transfer Around Openings (FTAO) shear walls or as Perforated Shear Walls.

8. The nominal nail shear capacity assigned to CLT shear walls shall not include contributions from connections other than shear connections prescribed in B.3.2 and B.3.3.

9. ASCE 7 seismic criteria specific to light-frame construction do not apply to the design of CLT shear walls and CLT diaphragms.

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 0.150 in. thick, ASTM A572 Grade 50 steel sheet with the geometry illustrated in Figure B.3.2.

2. Vertical legs of angle connectors shall be fastened to wall panel using (8) 16d common steel nails (3.5 in. length x 0.315 in. shank diameter x 0.144 in. head diameter) with bright finish in accordance with ASTM F1667 including Supplementary Requirements of ASTM F1667 and NAIL Bonding 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) 9/32 in. diameter x 4-1/2 in. long (minimum) bolts to provide minimum 4,200 lb. 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 3-5/8 in. unthreaded shank length (minimum) to provide minimum 5,700 lb. bearing length and 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 concrete reaction 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.
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-down systems or conventional hold-down devices that use threaded anchor rods and nuts, screw, or bolt attachment to the CLT panel.

2. Where continuous tie-down systems are used, nuts 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-down 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 5.5 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, group 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 centroidal 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 II or Mode IV yielding, and comply with net section tension rupture and group tear-out resistance in accordance with NDS Appendix E.

B.4 Deflection

The CLT shear wall deflection, \( \delta_{cw} \), shall be calculated by use of the following equation:

\[
\delta_{cw} = \frac{576b_0 h^3}{4E_I (1-v^2)} + \frac{G_{eff} (1-v^2)}{G_I (1-v^2)} \left( \frac{bh}{2} \right) \left( \frac{bh}{2} \right)
\]

where:

- \( b_0 \) = unit shear induced by the design loads, in
- \( E_I \) = effective in-plane bending stiffness of the CLT panel, psi
- \( G_I \) = effective in-plane shear stiffness of the CLT panel, psi/in. of panel length
- \( h \) = CLT panel height, ft
- \( b_w \) = individual CLT panel length, ft
- \( G_{eff} \) = effective in-plane shear stiffness of the CLT panel, psi/in. of panel length
- \( \delta_{cw} \) = wall deflection, in

B.5 Nominal Unit Shear Capacity

Nominal unit shear capacity, \( \tau_{uw} \), 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 treated as the sum of the nominal unit shear capacities of each face.

\[
\tau_{uw} = \frac{nG_{uw}}{2\sqrt{G_I \sqrt{G_{eff}}}}
\]

where:

- \( n \) = number of angle connections at bottom of panel face
- \( G_{uw} \) = connector nominal shear capacity in accordance with NDS (lbs)
- \( G_I \) = CLT panel specific gravity adjustment factor, \( G_I = 0.5 \) for 0.40 < \( G_I < 0.50 \), \( G_I = 0.86 \) for \( G_I < 0.35 \), linear interpolation shall be permitted to be used to determine values of \( G_I \) for values 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.
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 (56, 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 as simple panel or multi-panel configurations. Design unit shear walls 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, torsion, and rotation) and 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 collapse but need not be limited to CLT walls, beams, and columns, and light-framed walls. The required use of the CLT shear wall LFRS is platform construction.

5/20/2023
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 requirement.

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 evaluate 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/torsion 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-down 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 the level of overstrength in the hold-down system in CLT shear wall testing and is intended to ensure shear strength is equal to or greater than the specified nominal strength of the shear connection.

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 Overturing and Opposing Internal Shears at Adjacent Vertical Edges Due to In-plane Unit Shear Loading**
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 considerations 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 as determinants of T and C forces. The contribution of dead load in the overturning design is specifically limited to that of 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 T and C 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 that 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 O is:

\[
\sum M_O = 0
\]

\[
T (b_h - b_v) - w_h b_v h_{eff} = 0
\]

\[
T = \frac{w_h b_v h_{eff}}{b_h - b_v}
\]

where:

v = unit shear, pf
w = dead load including wall panel self-weight, pf
h = CLT panel height, ft
C = CLT panel height, ft
T = compression force, lb
T = length of compression zone, ft
G = compression force at top of compression and panel from story above, lb
m = length of compression zone at top of compression and panel from story above, ft

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

\[
\sum M_O = 0
\]

\[
c (b_h - b_v) - w_v h_{eff} = 0
\]

where:

v = unit shear, pf
w = dead load including wall panel self-weight, pf
h = CLT panel height, ft
C = CLT panel height, ft
T = compression force, lb
T = length of compression zone, ft
G = compression force at top of compression and panel from story above, lb
m = length of compression zone at top of compression and panel from story above, ft

C = \frac{F_{tensile}}{Q_{lateral}(x)} \cdot \frac{12bh}{L}

C = \frac{F_{tensile}}{Q_{lateral}(x)} \cdot \frac{12bh}{L}

where:

x = length of compression zone, ft
C = compression force, lb
F_{tensile} = bearing stress perpendicular to grain, psi
Q_{lateral}(x) = design axial stress parallel to grain, psi
G = CLT wall parallel stiffness, in.
K = thickness of CLT wall panel layers oriented parallel to grain for determination of wall axial capacity, in.

The bearing resistance in Equations 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 Equations C-B.5 is based on the design axial stress associated with the CLT wall panel parallel to grain layers and is applicable where designated compression zone elements are used in transferring such forces as opposing 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 both 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, m, is satisfied as static equilibrium is determined by substitution for Equation C-B.4 into Equation C-B.3 and solving for m. The solution for m is limited by bearing stress perpendicular to grain in provided in Equation C-B.5.

\[
x = \frac{h_m}{h_b} \leq \frac{w_h b_v h_{eff}}{h_b}
\]

where:

w = \frac{w_h b_v h_{eff} h_b}{h_m}

Where the length of the compression zone, m, is smaller than the length of the compression end panel, h_c, a positive value under the root in Equation C-B.6 is produced and the resulting value of m 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., C = 0) can be obtained from Equation C-B.7. When Equation C-B.7 is satisfied, a negative root will occur in Equation C-B.6 indicating inadequate compression panel length.

\[
x = \frac{6F_{tensile} h_{eff}}{h_b}
\]

The leading 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 leading and geometry depicted in Figure C-B.3 for the compression end panel will understimate 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 tensile connector spacing in accordance with SDPWS B.3.3 which requires the same average vertical connector spacing with regard to using for the top and bottom edges of the CLT shear wall. The required number of connectors at vertical and horizontal edges (i.e., the same average spacing) provides balanced vertical and horizontal shear and enables interface tension behavior of individual panels at multiple-story shear walls 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 interior force-resisting systems 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 and for out-of-plane forces, interconnection of walls at intersections, and attachment of conventional hold-down devices at wall ends. The combined restraint for fascia members per Mode III, IV, and V and compliance with NDS Appendix E ensures that adding 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 anchor 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 permitting requirements for yielding in Mode III, IV and V are considered beneficial to in-plane shear wall strength and stiffness, which is already governed by nail yielding, and provide a desirable balance of the prescribed shear wall connectors.

To address the potential for excessive stress (e.g., wood scarf and lag screw) attachment to inhibit the rocking mechanisms of the CLT panel due to high axial stresses in screws loaded in withdrawal, screw attachment at 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 detail shown in Table C-B.2. Typical connection details at the top and bottom of wall locations and are subject to approval by the inspector having jurisdiction. Typical boring will use standard smooth Shank nails or pins to resist out-of-plane forces. Details for anchoring the top and bottom of wall for out-of-plane forces are specified in the manufacturer's instructions for out-of-plane anchor.
**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 \( \leq 3.75 \) percent are permissible such that actual panel aspect ratio ranges from 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.

**C-B.4 Shear Wall Deflection**

The CLT shear wall deflection equation incorporates five 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 change in observed stiffness of angle 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 deflections 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**

The bending term in the deflection equation is simplified from \( V_b/b^2/2 \times (E/L)_{tg} \) for a centerline with point load to \( 576k_a b_i^2 (E/L)_{tg} \) in account for the unit conversion so that \( (E/L)_{tg} \) can be in lb/in² and other units can be in ft. \( (E/L)_{tg} \) is the effective in-plane stiffness for bending to account for composite behavior between adjacent parallel laminations where transverse \( E \) is approximated as longitudinal E/30. \( (E/L)_{tg} \) can be calculated directly using transformed section properties in accordance with Equation C-B.8.

\[
(E/L)_{tg} = \frac{(b_i)^4}{12} \left( E_{LZ} \right) \Delta_L + E_{LZ2} \Delta C \quad (C-B.8)
\]

where:
- \( b_i \) = CLT panel length
- \( E_{LZ} \) = modulus of elasticity parallel to the grain for longitudinal layers (i.e., longitudinal E)
- \( E_{LZ2} \) = modulus of elasticity perpendicular to the grain for transverse layers (i.e., transverse E taken as longitudinal E/30)
- \( \Delta_L \) = individual longitudinal lamination thickness
- \( \Delta C \) = individual transverse lamination thickness

An alternative formulation for \( (E/L)_{tg} \) for a 5-layer panel with equal thicknesses layers is adapted from Bliss and Fonesco (63), see Equation C-B.9:

\[
(E/L)_{tg} = E_L \left( \frac{(b_i)^4}{12} \right) \left[ 1 + \frac{1}{E_{LZ} E_{LZ2}} \right] \quad (C-B.9)
\]

where:
- \( b_1, b_2, b_3 \) = thickness as shown in the Figure C-B.4 for a 5-layer panel (i.e., 3 longitudinal layers and 2 transverse layers).

**Figure C-B.5 Illustration of \( b_1, b_2, b_3 \) for a 5-layer panel**
The sliding term, \( V_\text{sliding} \), addresses sources of deformation in the connector including nails and bolts. The slip constant takes into account loading perpendicular to the grains in the nailed connection. The single nail diameter of 0.335 in. used for all the connections in this study allows for a simplified nail slip term, \( V_\text{nail} \). 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 fixed 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, \( \Delta_z \), is based on the induced overturning forces and includes sources of deflection such as frame, slip, device elongation, rod elongation, uncompensated shrinkage, and vertical compression deformation.

### Table C-8.1: Example \( \Delta_{\text{in-plane}} \) for in-plane shear

<table>
<thead>
<tr>
<th>Number of Layers</th>
<th>( \Delta_{\text{in-plane}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>28,700 psi</td>
</tr>
<tr>
<td>5</td>
<td>32,300 psi</td>
</tr>
<tr>
<td>7</td>
<td>33,500 psi</td>
</tr>
<tr>
<td>9</td>
<td>34,500 psi</td>
</tr>
</tbody>
</table>

1. Calculated values of \( \Delta_0 \) and \( \Delta_{\text{in-plane}} \) are based on the following inputs: \( E = 400000 \, \text{psi}, \text{lamella} = \frac{143.8}{6} = 23.9 \, \text{in}, \text{number of lamellae} = 1, \text{lamella thickness} = 0.5 \, \text{in}, \) and lamella thickness = 0.25 in.
<table>
<thead>
<tr>
<th>Table C-B.2</th>
<th>Typical Connection Details (continued)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior wall to floor connection (example with interior framed wall)</td>
<td></td>
</tr>
<tr>
<td>Interior wall to floor connection</td>
<td></td>
</tr>
<tr>
<td>Exterior wall to roof</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table C-B.2</th>
<th>Typical Connection Details (continued)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior wall to roof</td>
<td></td>
</tr>
<tr>
<td>Interior wall to roof</td>
<td></td>
</tr>
<tr>
<td>Wall panel to wall panel connection, square edge configuration (i.e., vertical edge connection)</td>
<td></td>
</tr>
<tr>
<td>Wall panel to wall panel connection, lap configuration (i.e., vertical edge connection)</td>
<td></td>
</tr>
</tbody>
</table>
6.3 CLT Shear Wall Example Description

- 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

Figure 6-2. Elevation

Figure 6-3. Typical Floor Plan (first story openings shown)
### 6.3 CLT Shear Wall Example Description

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

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof/Ceiling</td>
<td>Light-frame roof, gypsum board ceiling, roofing, insulation</td>
<td>25 psf</td>
</tr>
<tr>
<td>Floor</td>
<td>5-layer CLT (6.875 in. thick), gypsum board ceiling, flooring. Includes 8 psf of floor area for wall partitions</td>
<td>35 psf</td>
</tr>
<tr>
<td>Interior Walls</td>
<td>3-layer CLT (4.125 in. thick), light-frame wall, gypsum board finish, sound insulation</td>
<td>20 psf</td>
</tr>
<tr>
<td>Exterior Walls</td>
<td>3-layer CLT (4.125 in. thick), light-frame wall, gypsum board interior finish, stucco exterior, insulation</td>
<td>30 psf</td>
</tr>
</tbody>
</table>
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):
\[
V = C_s W = \frac{S_{DS}}{(R/I)} W = \frac{1.0}{(3.0/1.0)} W = 0.333 W \text{ kips}
\]

The portion of base shear tributary to the CLT shear walls of interest is:
\[
V_{(\text{Line 4})} = 42.3 \text{ kips}
\]
6.4 Seismic Forces

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

<table>
<thead>
<tr>
<th>Story</th>
<th>Lateral force, $V_x$ (kips)</th>
<th>Unit Shear Force per Foot of Shear Wall Length (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>15.9</td>
<td>477</td>
</tr>
<tr>
<td>2</td>
<td>33.5</td>
<td>1,009</td>
</tr>
<tr>
<td>1</td>
<td>42.3</td>
<td>1,273</td>
</tr>
</tbody>
</table>

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

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

LRFD design unit shear capacity for seismic:

\[ v_{s(\text{seismic})} = \phi(n) \left( \frac{2605}{b_s} \right) C_G \]  

(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
- \( \phi \) = 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

<table>
<thead>
<tr>
<th>Story</th>
<th>Panel thickness</th>
<th>Panel length, $b_s$</th>
<th>Panel height, $h$</th>
<th>Number of connectors per panel at top and bottom panel edge</th>
<th>Number of connectors at each adjoining vertical panel edge</th>
<th>$V_{n1}$ Nominal unit shear capacity, $(n \cdot 2605)/b_s$ (plf)</th>
<th>$V_{s(seismic)}$, LRFD design unit shear capacity, $(\phi = 0.5)$ (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>(in.)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(n)</td>
<td>(n x h/b_s)</td>
<td>1,096</td>
<td>548</td>
</tr>
<tr>
<td>2</td>
<td>4.125</td>
<td>4.75</td>
<td>9.5</td>
<td>2</td>
<td>4</td>
<td>2,193</td>
<td>1,097</td>
</tr>
<tr>
<td>1</td>
<td>4.125</td>
<td>4.75</td>
<td>9.5</td>
<td>5</td>
<td>10</td>
<td>2,742</td>
<td>1,371</td>
</tr>
</tbody>
</table>
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:

\[ v' = \phi \lambda K F_v(t_v) = 0.75(1.0)(2.88)(9,700) = 20,849 \text{ plf} \]

where:

\[ F_v(t_v) = 9,700 \text{ plf} \] (ASD value from CLT panel manufacturer’s evaluation report)

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

\[ 20,849 \text{ plf} \gg 1,273 \text{ plf} \]

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

The sum of the moments is 0:

\[ \sum M_o = 0 \]

\[ T (b_{eff}) - \nu b_s h + w b_s \left( \frac{b_z}{2} \right) - T_T(b_{eff}) = 0 \]  
(SDPWS-21 Eq. C-B.1)

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

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

<table>
<thead>
<tr>
<th>Story</th>
<th>Unit shear force per foot of shear wall length</th>
<th>( \nu_s(\text{seismic}) )</th>
<th>( 2 \times \nu_s(\text{seismic}) )</th>
<th>( T_T ) from story above</th>
<th>T for 2 x ( \nu_s(\text{seismic}) ) requirement for load combination: 1.0E - 0.7D</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>(plf)</td>
<td>548</td>
<td>1,097</td>
<td>0</td>
<td>11,293</td>
</tr>
<tr>
<td>2</td>
<td>1,009</td>
<td>1,097</td>
<td>2,194</td>
<td>11,293</td>
<td>34,540</td>
</tr>
<tr>
<td>1</td>
<td>1,273</td>
<td>1,371</td>
<td>2,742</td>
<td>34,540</td>
<td>63,968</td>
</tr>
</tbody>
</table>
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**
  
  \[4 \times 17,687 \text{ lb} = 70,748 \text{ lb} > 63,968 \text{ lbs}\]

- **2nd story to top of 1st story, four hold-downs**
  
  \[4 \times 17,687 \text{ lb} = 70,748 \text{ lb} > 34,540 \text{ lbs}\]

- **3rd story to top of 2nd story, two hold-downs**
  
  \[2 \times 17,687 \text{ lb} = 37,374 \text{ lb} > 11,293 \text{ lbs}\]

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>
<thead>
<tr>
<th>Story</th>
<th>Unit shear force per foot of shear wall length (plf)</th>
<th>T for load combination 1.0E - 0.7D (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>477</td>
<td>4,714</td>
</tr>
<tr>
<td>2</td>
<td>1,009</td>
<td>14,604</td>
</tr>
<tr>
<td>1</td>
<td>1,273</td>
<td>27,472</td>
</tr>
</tbody>
</table>

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

\[
\Delta_{\text{hold-down}} = \frac{27,472 \text{ lb}}{4(17,678 \text{ lb})} (0.253 \text{ in.}) = 0.098 \text{ in.} < 0.185 \text{ in.}
\]
6.6.2 CLT Shear Wall Compression Zone

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

\[
\sum M_o = 0
\]

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

\[
C = F_{c_{\perp}}'(t)(x)\left( \frac{12\text{in.}}{\text{ft}} \right) \quad \text{(SDPWS-21 Eq. C-B.4)}
\]

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

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

<table>
<thead>
<tr>
<th>Story</th>
<th>Unit shear force per foot of shear wall length</th>
<th>Dead load, (w_{DL})</th>
<th>Live load, (w_{LL})</th>
<th>C(_T), Compression from top</th>
<th>Compression zone length, x</th>
<th>C, for load combination (1.0E + 1.4D + 0.5L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>(plf)</td>
<td>(plf)</td>
<td>(plf)</td>
<td>(lb)</td>
<td>(in.)</td>
<td>(lb)</td>
</tr>
<tr>
<td>3</td>
<td>477</td>
<td>190</td>
<td>0</td>
<td>0</td>
<td>2.00</td>
<td>5,257</td>
</tr>
<tr>
<td>2</td>
<td>1,009</td>
<td>793</td>
<td>690</td>
<td>5,257</td>
<td>7.64</td>
<td>20,144</td>
</tr>
<tr>
<td>1</td>
<td>1,273</td>
<td>793</td>
<td>690</td>
<td>20,144</td>
<td>4.56</td>
<td>36,545</td>
</tr>
</tbody>
</table>

Check of CLT wall panel axial capacity is required
6.7 CLT Shear Wall Deflection

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

(SDPWS-21 Eq. B-1)

<table>
<thead>
<tr>
<th>Total shear wall = deflection, ( \delta_{SW} )</th>
<th>Panel bending and shear +</th>
<th>Sliding +</th>
<th>Panel rotation +</th>
<th>Rigid body rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Dotted Illustration" /></td>
<td><img src="image2.png" alt="Solid Illustration" /></td>
<td><img src="image3.png" alt="Dotted Illustration" /></td>
<td><img src="image4.png" alt="Solid Illustration" /></td>
<td><img src="image5.png" alt="Dotted Illustration" /></td>
</tr>
</tbody>
</table>
6.7 CLT Shear Wall Deflection

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

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

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:

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

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

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

Volume 1: Design Examples

Questions?
Thank you!