CLT DIAPHRAGM DESIGN GUIDE

Application 2021 SDPWS Provisions

Disclaimer: This presentation was developed by a third party and is not funded by WoodWorks for the Softwood Lumber Board

Presented by Eric McDonnell
12th July 2023
CLT Diaphragm Components

- **CLT Diaphragm Design Force Recap**
  - Yielding diaphragm components (i.e. shear dowels) are design to the diaphragm design forces AND must be controlled by yield modes IIIs or IV.

\[
\frac{v_n}{\alpha_D} \geq F_{design,ASD} \quad \text{-OR-} \quad \phi_D v_n \geq F_{design,LRFD}
\]

- Remaining components are designed to include a force increase factor, \( I_D \), where \( R'NDS = \) adjusted design capacity.

\[
R'NDS \geq \gamma_D F_{design,ASD} \quad \text{-OR-} \quad R'NDS \geq \gamma_D F_{design,LRFD}
\]

- Combining with ASCE 7-16:

\[
\gamma_D F_{design} = \gamma_D \max (F_{px}, F_x) + \gamma_D \Omega_0 F_{x,\text{transfer}}
\]
4 Diaphragm Shear Components

- Analysis and Design for In-plane Actions

**FIGURE 4.1:** Free body diagram of corner CLT panel
• Panel-to-Panel Connections
• CLT Panel Design

**Figure 4.3:** Edgewise bending in the major (left) and minor (right) CLT strength directions

**Figure 4.4:** Typical half-lap connection
# Diaphragm Boundary Elements

![Image of diaphragm boundary elements with legend and table]

## Table 5.1: Seismic design force requirements for collectors, including their splices and connections to the VLFRS

<table>
<thead>
<tr>
<th>Source of Design Force</th>
<th>Required Design Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDPWS 2021 requirement for all collectors</td>
<td>$\gamma_D \max(F_{px},F_x) + \gamma_D \Omega_0 F_{x,\text{transfer}}$</td>
</tr>
<tr>
<td>SDPA/S force increase</td>
<td>$\Omega_0 F_x + \Omega_0 F_{x,\text{transfer}}$</td>
</tr>
<tr>
<td>ASCE 7-16 §12.10.2.1 seismic design category C through F, when not entirely braced by light-frame wood shear walls</td>
<td>$\Omega_0 F_{px,\text{very 12.10-1}} + \Omega_0 F_{x,\text{transfer}}$</td>
</tr>
<tr>
<td>Item 1</td>
<td>$F_{px,\text{very 12.10-2}} + \Omega_0 F_{x,\text{transfer}}$</td>
</tr>
</tbody>
</table>
• Compression Load Path
• Compression Load Path

**Figure 5.6:** Effective width in compression – discrete bearing

**Figure 6.6:** Example of panel-to-panel over beam connection
Gravity Framing as Lateral Element

• If gravity element is intended to serve as lateral element need to check for combined loading
  • Compression buckling of gravity member, in isolation or combination with CLT panel
  • Tension in gravity member, INCLUDING across beam-to-column connection

• For all timber framed buildings additional checks include:
  • Transfer of diaphragm forces from CLT to supporting gravity element
  • Beam element for tension / compression demand
  • Compression perpendicular to grain at beam-to-column interface
  • Tension load path across beam-to-column joint
    • Maintaining deformation compatibility

For Timber Framed Buildings:
• Recommend keeping diaphragm forces in the CLT plane
• Diaphragm & Collector Connections to Steel VLFRS
• Diaphragm & Collector Connections to Concrete Shear Walls

[Diagram of Diaphragm-to-concrete wall connection]

[Diagram of Collector plate to embed plate in concrete wall connection]
Diaphragm & Collector Connections to Concrete Shear Walls (Continued)

**FIGURE 5.11**: Collector load path to concrete shear wall through CLT panel
• Diaphragm & Collector Connections to Light Framed Shear Walls

**FIGURE 5.15:** CLT to wood sheathed shear wall example detail
## Diaphragm Deflections & Stiffness

- **Classification of Diaphragm as Flexible or Rigid**

<table>
<thead>
<tr>
<th>Category</th>
<th>ASCE 7 §12.3.1</th>
<th>IBC §1604.4</th>
<th>SDPWS §4.1.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexible</td>
<td>Permitted when ( \frac{\delta_{MDD}}{\Delta_{ADVE}} &gt; 2 )</td>
<td>N/A</td>
<td>Per ASCE 7</td>
</tr>
<tr>
<td>Rigid</td>
<td>N/A</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Semi-rigid</td>
<td>When not idealized as flexible or rigid, analysis shall include consideration of diaphragm stiffness</td>
<td>Total lateral force shall be distributed to elements of VLFRS in proportion to their rigidities, considering the rigidity of the diaphragm</td>
<td>Shall consider relative stiffnesses of VLFRS &amp; diaphragms; envelope analysis permitted</td>
</tr>
</tbody>
</table>

### Notes:
- \( \delta_{MDD} \): Maximum in-plane diaphragm deflection (in.)
- \( \Delta_{ADVE} \): Average drift of adjoining vertical elements of the VLFRS over the story below the diaphragm under consideration, under tributary lateral load equivalent to that used in the computation of \( \delta_{MDD} \) (in.)
- **Semi-Rigid Diaphragm Analysis**

<table>
<thead>
<tr>
<th>Bounding Analysis</th>
<th>Stiff Case</th>
<th>Flexible Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Envelope procedure</td>
<td>Rigid diaphragm idealization</td>
<td>Flexible diaphragm idealization</td>
</tr>
<tr>
<td>Semi-rigid bounding</td>
<td>“Stiff” semi-rigid analysis</td>
<td>“Flexible” semi-rigid analysis</td>
</tr>
<tr>
<td>Rigid and semi-rigid bounding</td>
<td>Rigid diaphragm idealization</td>
<td>“Flexible” semi-rigid analysis</td>
</tr>
<tr>
<td>Semi-rigid and flexible bounding</td>
<td>“Stiff” semi-rigid analysis</td>
<td>Flexible diaphragm idealization</td>
</tr>
</tbody>
</table>

- **Diaphragm Deflection Equations**
  - Previously referenced equations
    
    \[
    \delta_{\text{dia}} = \frac{5vL^2}{8EAW} + \frac{vL}{4G_{\text{cf}}t_p} + CLe_n + \frac{\Sigma(x\Delta_c)}{2W}
    \]
  - Proposed CLT specific equation by Lawson, et al
    
    \[
    \delta_{\text{dia}} = \left(12 \text{ in} \right) \frac{5vWL^3}{96EAd^3} + \frac{vL}{4G_{\text{cf}}t_p} \left( \frac{n_1e_{1h}}{P_1} + \frac{n_1e_{1v}}{P_1} \right) + \frac{\Sigma(x\Delta_c)}{2d}
    \]
    
    \[
    \delta_{\text{cante}} = \left(12 \text{ in} \right) \frac{vW'L'^3}{4EAd^2} + \frac{vL'}{2G_{\text{cf}}t_p} + \frac{L'}{2} \left( \frac{n_1e_{1h}}{P_1} + \frac{n_1e_{1v}}{P_1} \right) + \frac{\Sigma(x'\Delta_c)}{d}
    \]

  ![Fig. 1. Deflection parameters of a simple span diaphragm.](image-url)
• Fastener Slip Relationships
  • Limited in the NDS
  • Manufacture Specific Data
  • European Guidance

• Analytical Modeling of CLT Behavior
  • Homogenous Model
  • Discrete Models
  • Component Models

• Estimating Inelastic Seismic Deflections
Special Design Considerations

- Sub-diaphragms
- Staggered CLT Panel Layouts
- Alternate Diaphragm Procedures
- Durability

**Figure 7.1**: Non-staggered and staggered diaphragm conditions
• Bracing of CMU / Concrete Walls w/ CLT Diaphragms

• Acceptable provided walls are self-supporting

4.1.5 Wood Members and Systems Resisting Seismic Forces Contributed by Concrete and Masonry Walls

Wood-frame shear walls, wood-frame diaphragms, trusses, and other wood members and systems shall not be used to resist seismic forces contributed by concrete or masonry walls in structures over one story in height.

Exceptions:
1. Wood floor and roof members shall be permitted to be used in diaphragms and horizontal trusses to resist horizontal seismic forces contributed by masonry or concrete walls provided such forces do not result in torsional force distribution through the diaphragm or truss.

When SDPWS 4.15 is met, other requirements such as wall anchorage in ASCE 7 12.11, drift limits in ASCE 7 12.11, and any out-of-plane deflection limits of the wall material may apply.
Example 1: 12-Story Office w/ Distributed Frames

- Savannah, Georgia: Wind vs Seismic Checks

8.2.3 Wind vs. Seismic Comparison

The building seismic and wind base shears are:

- $V_{\text{base,seismic}} = 470$ kips
- $V_{\text{base,NS,wind}} = 1,900$ kips
- $V_{\text{base,EW,wind}} = 680$ kips

Level 11 horizontal force comparisons:

- $F_{d,\text{seismic}} / \phi_D = 74$ kips/0.5 = 148 kips
- $F_{d,\text{NS,wind}} / \phi_D = 124$ kips/0.8 = 166 kips
- $F_{d,EW,wind} / \phi_D = 54$ kips/0.8 = 86 kips
• **Diaphragm Design Checks**

**FIGURE 8.3:** Typical spline connection

**FIGURE 8.4:** Effective lamella locally transferring shear in spline connection

**FIGURE 8.5:** Typical panel to glulam beam
• Collector Design Checks
Collector Design Checks

- Typical Collector in Tension
- Collector to Braced Frame Design
- CLT Collector in Compression
- Combined Comp + Bending

**FIGURE 8.7:** Typical steel collector

**FIGURE 8.8:** Typical steel collector plate detail
Chord Design Checks

- Chord plate in tension
- Tension Splice plate
- Local wood tear-out checks
- CLT tension + out-of-plane bending
• Verify Diaphragm Rigidity

\[ \delta_{\text{MDD}} = \delta_{\text{bending}} + \delta_{\text{shear}} + \delta_{\text{slip}} + \delta_{\text{splice}} \]

**Fig. 1.** Deflection parameters of a simple span diaphragm.

\[ \delta_{\text{dia}} = \left( \frac{12 \text{ in}}{1 \text{ ft}} \right) \frac{5\nu WL^3}{96EAd^2} + \frac{vL}{4G_{\text{c,}}\tau^2} + \frac{L}{4} \left( \frac{n_{\text{eff}}}{P_1} + \frac{n_{\text{eff}}}{P_1} \right) + \frac{\Sigma(x \Delta)}{2d} \]

\[ \frac{\delta_{\text{MDD}}}{\Delta_{\text{ADVE}}} = 0.14 < 2.0 \]
Example 2: 12-Story Office w/ Concrete Cores

- San Francisco, California

4.2 Sheathed Wood-Frame Diaphragms

4.2.1 Application Requirements

Sheathed wood-frame diaphragms shall be permitted to be used to resist wind forces provided the deflection is the plane of the diaphragm, as determined by calculations, tests, or analyses drawn therefrom, does not exceed the maximum permissible deflection limit of attached load distributing or resisting elements. Permissible deflection shall be that deflection that will permit the diaphragm and any attached elements to maintain their structural integrity and continue to support their prescribed loads as determined by the applicable building code or standard. Framing members, blocking, and connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

4.2.2 Diaphragm Aspect Ratios

Size and shape of diaphragms shall be limited to the aspect ratios in Table 4.2.2.

<table>
<thead>
<tr>
<th>Table 4.2.2 Maximum Diaphragm Aspect Ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Flat or Sloped Diaphragms)</td>
</tr>
<tr>
<td>Sheathed Wood-Frame Diaphragm Assemblies</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Maximum L/W Ratio</td>
</tr>
<tr>
<td>Wood structural panel, unblocked</td>
</tr>
<tr>
<td>Wood structural panel, blocked</td>
</tr>
<tr>
<td>Single-layer horizontally-sheathed lumber</td>
</tr>
<tr>
<td>Single-layer diagonally-sheathed lumber</td>
</tr>
<tr>
<td>Double-layer diagonally-sheathed lumber</td>
</tr>
</tbody>
</table>

4.2.3 Deflection

Calculations of diaphragm deflection shall account for bending and shear deflections, fastener deformation, chord splice slip, and other contributing sources of deflection. The diaphragm deflection, $\delta_{dp}$, shall be permitted to be calculated by use of the equations in Table 4.2.2.
• Collector Design Checks
• Chord Design Checks
Example 3: 5-Story Residential w/ Sheathed SW’s

- Portland, OR
• Collector Design Checks
CLT Diaphragms Designed to 2021 SDPWS

- **COMPLETE/PERMITTED PROJECTS**
  - 4 states
  - 13 projects
  - 20 buildings

- **IN DESIGN/PERMITTING**
## Precalculated Design Capacities

- Individual Fasteners
- Spline Capacities
- Fasteners / Spline Capacities
- Steel Strap Capacities

### Table A.4.4: Nominal diaphragm shear capacity for spaced fastener in spline continued

<table>
<thead>
<tr>
<th>Spline Material</th>
<th>Fastener</th>
<th>Nominal Diaphragm Shear Capacity of Fasteners, $V_{n} = 4.5Z_{n}Y_{n}$, @ Spacing, 5 Sym pt (pf)</th>
<th>Reference Spline Shear Capacity, $V_{R}^{*}$ (pf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>12-in. o.c.</td>
<td>6-in. o.c.</td>
</tr>
<tr>
<td>CLT 5G = 0.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>General sheeting (2X/12)</td>
<td>8d common nail</td>
<td>330</td>
<td>659</td>
</tr>
<tr>
<td>General sheeting (2X/12)</td>
<td>10d common nail</td>
<td>386</td>
<td>718</td>
</tr>
<tr>
<td>General sheeting (2X/12)</td>
<td>Example screw 1</td>
<td>363</td>
<td>726</td>
</tr>
<tr>
<td>General sheeting (2X/12)</td>
<td>Example screw 2</td>
<td>336</td>
<td>667</td>
</tr>
<tr>
<td>Structural 1-sheeting (2X/22)</td>
<td>8d common nail</td>
<td>397</td>
<td>732</td>
</tr>
<tr>
<td>Structural 1-sheeting (2X/22)</td>
<td>10d common nail</td>
<td>463</td>
<td>926</td>
</tr>
<tr>
<td>Structural 1-sheeting (2X/22)</td>
<td>Example screw 1</td>
<td>423</td>
<td>847</td>
</tr>
<tr>
<td>Structural 1-sheeting (2X/22)</td>
<td>Example screw 2</td>
<td>466</td>
<td>921</td>
</tr>
<tr>
<td>General sheeting (7/8)</td>
<td>10d common nail</td>
<td>423</td>
<td>841</td>
</tr>
<tr>
<td>General sheeting (7/8)</td>
<td>10d common nail</td>
<td>486</td>
<td>972</td>
</tr>
<tr>
<td>General sheeting (7/8)</td>
<td>Example screw 1</td>
<td>386</td>
<td>773</td>
</tr>
<tr>
<td>General sheeting (7/8)</td>
<td>Example screw 2</td>
<td>462</td>
<td>927</td>
</tr>
<tr>
<td>Structural 1-sheeting (7/8)</td>
<td>10d common nail</td>
<td>517</td>
<td>1,083</td>
</tr>
<tr>
<td>Structural 1-sheeting (7/8)</td>
<td>10d common nail</td>
<td>537</td>
<td>1,174</td>
</tr>
<tr>
<td>Structural 1-sheeting (7/8)</td>
<td>Example screw 1</td>
<td>461</td>
<td>923</td>
</tr>
<tr>
<td>Structural 1-sheeting (7/8)</td>
<td>Example screw 2</td>
<td>539</td>
<td>1,177</td>
</tr>
<tr>
<td>General sheeting (1X/8)</td>
<td>10d common nail</td>
<td>484</td>
<td>968</td>
</tr>
<tr>
<td>General sheeting (1X/8)</td>
<td>10d common nail</td>
<td>555</td>
<td>1,099</td>
</tr>
<tr>
<td>General sheeting (1X/8)</td>
<td>Example screw 1</td>
<td>434</td>
<td>868</td>
</tr>
<tr>
<td>General sheeting (1X/8)</td>
<td>Example screw 2</td>
<td>528</td>
<td>1,095</td>
</tr>
<tr>
<td>Structural 1-sheeting (1X/8)</td>
<td>10d common nail</td>
<td>529</td>
<td>1,098</td>
</tr>
<tr>
<td>Structural 1-sheeting (1X/8)</td>
<td>Example screw 1</td>
<td>527</td>
<td>1,094</td>
</tr>
<tr>
<td>Structural 1-sheeting (1X/8)</td>
<td>Example screw 2</td>
<td>603</td>
<td>1,265</td>
</tr>
</tbody>
</table>

*a*  
- Tabulated values based on all adjustment factors applicable to $Z_{n}$ in NDS Table 11.3.1 equal to 11.0. ($Z = 2$). Designer to verify applicability.  
- All fastener capacity values provided are controlled by Mode I, or I/II fastener yielding.

*b*  
- Adjusted design spline capacity to be calculated from reference spline capacity using NDS Table 9.3.1.  
- Before using highlighted fastener capacity values, verify the adjusted design spline capacity is greater than the combined demands per SDPWS 4.6.4.6:

- Verify adjusted spline capacity is greater than SDPWS 4.6.4.3 Except 1 for wind design.
- Verify adjusted spline capacity is greater than SDPWS 4.6.4.3 for seismic design and SDPWS 4.6.4.3 Except 1 for wind design.
## Literature Review

- Component Level Testing
- Full Scale Diaphragm Testing
- Diaphragm Design Literature
- Other References

### 8.5 Summary of Significant Tests and Related References

<table>
<thead>
<tr>
<th>Article Title</th>
<th>Connection Type</th>
<th>Fastener &amp; Loaded</th>
<th>Type of Loading</th>
<th>Loading Direction</th>
<th>Reported Results</th>
<th>Additional Notes</th>
<th>Reported Fastener Slip Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brown et al., 2008</td>
<td>L-socket in place</td>
<td>Steel plate</td>
<td>Monotonic and cyclic</td>
<td>Parallel to sheave plane</td>
<td>Load displacement, stress, and hysteretic loops</td>
<td>Test results compared with EC8 strength and stiffness prediction equations</td>
<td>$X_{max}$ and $Y_b$ (peak yielding force)</td>
</tr>
<tr>
<td>Figueiredo et al., 2016</td>
<td>Internal socket, bolt, and nut</td>
<td>Steel plate</td>
<td>Monotonic and cyclic</td>
<td>Parallel to sheave plane</td>
<td>Load-carrying capacity and stiffness</td>
<td>Authors observed significant difference between load stiffness and Hooke's law method</td>
<td>$X_{max}$ and $Y_b$ (ultimate load stiffness)</td>
</tr>
<tr>
<td>Luna et al., 2018</td>
<td>Double socket and bolts</td>
<td>Steel plate</td>
<td>Monotonic and cyclic</td>
<td>Parallel to sheave plane</td>
<td>Load-carrying capacity and stiffness</td>
<td>H-bolt tested analytically; none of the options provided a significantly higher stiffness</td>
<td>$K_{elastic}$</td>
</tr>
<tr>
<td>Mewett, 2012</td>
<td>In-plane stiffness of cross-laminar timber beams</td>
<td>Steel plate</td>
<td>Monotonic and cyclic</td>
<td>Parallel to sheave plane</td>
<td>Peak and ultimate load, stiffness</td>
<td>Stress-strain curve from cross-laminar timber beam</td>
<td>$X_{max}$ to $Y_{last}$ (ultimate load stiffness)</td>
</tr>
<tr>
<td>Kostka et al., 2012</td>
<td>Lap and overlap joints</td>
<td>Steel plate</td>
<td>Monotonic and cyclic</td>
<td>Parallel to sheave plane</td>
<td>12 tested configurations; $Y_b$ represented statistical average for each configuration</td>
<td>Each configuration includes $X_{max}$, monotonic, and fatigue test data.</td>
<td>$X_{max}$ and $Y_b$ (peak yielding force)</td>
</tr>
<tr>
<td>[This work, 2014]</td>
<td>Steel plate</td>
<td>Steel plate</td>
<td>Monotonic and cyclic</td>
<td>Parallel to sheave plane</td>
<td>Load-carrying capacity and stiffness</td>
<td>Lap shear test results also available</td>
<td>$K_{max}$ and $Y_b$ (peak yielding force)</td>
</tr>
</tbody>
</table>

*Note: The table lists various test configurations and their corresponding results and additional notes.*
Thank you

CONTACT INFORMATION:  scott.breneman@woodworks.org
                       eric.mcdonnell@holmes.us