Design and Performance of CLT Structures under Lateral Loads

by

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Lateral Loads

- Generated by action of earthquakes or strong winds
- Wind loads are due to air pressure or suction
- Earthquake loads are related to mass and acceleration

CLT and Lateral Loads

- CLT originated as a system in Europe 20 years ago
- Most initial buildings located in low seismic or low wind areas
- Situation is different in many parts of the USA

CLT Construction and Details

- Usually used as platform type of construction
- Floor panels connected to the walls underneath with long self-tapping screws
- Walls connected to floors with brackets and screws or nails
CLT Platform Type Construction

Lateral Load Design Requirements for CLT

- Design requirements in early stages of standardization and recognition by the model building codes in the U.S.
- **Wind Design**
  - Linear-elastic analysis under wind loads
  - Resistances for members and connections to be derived from applicable design standards (NDS, PRG-320) and as presented in the CLT Handbook
- **Seismic Design**
  - Load effects and member resistances are more complex
  - Non-linear behavior of the system needs to be considered
  - Consequently, presentation will focus more on seismic performance of CLT components and buildings

Lateral Load Resisting Elements

- Main lateral load resisting elements
  - CLT shear walls - vertical
  - CLT diaphragms - horizontal
- Typical wood-frame shear walls have unit shear strength provided in SDPWS 2008
- Strength of CLT shear walls to be determined from basic principles of engineering mechanics using NDS for Wood Construction for connection design and procedures given in Chapters 3 and 5 of the CLT Handbook
- The shear strength of CLT elements will vary based on wall aspect ratio, connector locations, fastener size and spacing, and the shear strength of the CLT panel itself

Capacity Based Design Principles

- Recommended for seismic design of CLT structures
- CLT wall panels to be considered as linear elastic
- Ensure predictable yielding in the connections through fastener yielding and wood crushing prior to onset of undesirable brittle wood failure modes
  - Connections with slender dowel-type fasteners and larger spacing/ends distance are preferred
- The Strength Limit States design resistance of wood members at connection locations should be higher than the nominal connection capacity
Capacity Based Design - Connections

- Fasteners loaded in shear to be designed to fail in fastener yielding modes III or IV as defined in the NDS
- Nominal connection capacity in shear for dowel fasteners: \( nZK_F\lambda C_{M1} C_{T1} C_{leq} \)
  
  where:
  - \( n \) = number of fasteners;
  - \( Z \) = reference lateral design value for a single fastener
  - \( K_F, \lambda, C_M, C_t, \) and \( C_{leq} \) are adjustment factors in NDS for format conversion, time effect, wet service, temperature and end grain
- The design demand for any wood member should be higher than the nominal connection capacity

Seismic Analysis and Design Methods

- Equivalent Lateral Force Procedure (ELFP)
- Linear modal response dynamic analysis
- Non-linear time history dynamic analysis
- Performance based seismic design

Equivalent Lateral Force Procedure (ELFP)

- Seismic demand based on Design Response Spectrum
- System specific seismic design coefficients \( R, C_q \) and \( \Omega_0 \) are needed but not currently available in the US codes and standards
- Two methods are currently available FEMA P-695 and FEMA P-795
FEMA P-695 Quantification of Building Seismic Performance Factors

- Primary method for determining seismic design coefficients
- Work has been initiated – lead Dr. John van de Lindt
- Rigorous methodology that requires seismic evaluation of numerous 3-D buildings under a set of 22 bi-axial ground motions
- Seismic design coefficients shall meet the performance objective – less than 10% probability of total or partial collapse during Maximum Considered Earthquake (MCE) motions (2% in 50 years)

ASCE7-10 Performance Based Design

- The reliability of the CLT walls must not be less than that expected for a similar component designed in accordance with the strength procedures of ASCE 7
- Procedure specifies requirements for analysis, testing, documentation, and peer review
- Commentary identifies the minimum performance objective for Risk Category I and II Structures as 10% probability of total or partial collapse at MCE level

FEMA P-795 Component Equivalency Methodology

- Applicable for determination of seismic design coefficients on component level (CLT wall panels)
- Direct comparison of seismic performance of the “proposed” component to the “reference” SFRS component recognized in ASCE7
- Key evaluation parameters: ultimate deformation, strength, initial stiffness, and ductility
- Proposed component (CLT) that has equivalent performance to the components of the reference SFRS (nailed shear walls) can use the same seismic design coefficients as the reference one

Calculation of Deflections

- Deflection calculations should be based on principles of engineering mechanics or derived from testing
- Calculations should account for all sources of deflection including panel bending, panel shear, and fastener shear and uplift deformation
- Deflection estimates may take into account the predominant rocking mode only
Research Projects on Seismic Performance

- To help the codification process of CLT as a structural system, research projects on the seismic performance were undertaken or are underway:
  - University of Ljubljana, Slovenia
  - University of Trieste and University of Sassari, Italy
  - Karlsruhe Institute of Technology, Germany
  - The SOFIE Project by IVALSA in Italy
  - Research at FPInnovations in Canada
  - Colorado State and South Dakota State Universities

Research at FPInnovations in Canada

- Cyclic tests on connections with different brackets and fasteners
- Cyclic tests on various configurations of CLT walls to evaluate:
  - Effects of vertical load
  - Influence of different brackets and their position
  - Effect of fasteners in the brackets: annular nails, spiral nails, screws, timber rivets
  - Use of hold-downs
  - CLT walls with half lap joints
  - Two-storey CLT assemblies with floor panels
  - Tall CLT walls (4.9 m, 16’)
  - Influence of foundation (walls on CLT floor panels)
  - Influence of loading protocols

CLT Walls Configurations Tested

The Test Setup for CLT Wall Tests
Fasteners and Brackets Used

CLT Wall Panels Behaved Almost as Rigid Bodies during the Testing

Although slight shear deformations in the panels were measured, most of the panel deflections occurred as a result of the yielding deformation in the joints connecting the walls to the foundation.

In case of multi-panel walls, deformations in the half-lap joints also had significant contribution to the overall wall deflection.

Influence of Vertical Load on the Resistance

- Cyclic behavior of CLT wall panels is not degraded by the presence of axial load.
- Walls with axial load had increased initial stiffness and shear capacity but almost similar ductility.
- Slight change in hysteresis loop shape.

Annular Ring Nails vs. Spiral Nails

- The wall with 12 - 10d annular ring nails (0.134" x 3") per bracket had slightly higher resistance than the wall with 18 - 16d spiral nails (0.153" x 3.5")
- Lower ductility due to a more sudden drop in load.
Failure Modes - Ring Nails vs. Spiral Nails

- Failure modes were slightly different
- Spiral nails exhibiting mostly ductile bearing failure
- Ring nails had a tendency to pull out small chunks of wood

CLT Walls with Screws in the Brackets

- Screws can be also fasteners of choice for CLT walls
- The load carrying capacity drops a bit faster at higher deformation levels than in the case of walls with nails

CLT Walls with Brackets and Hold-downs

- With increased stiffness, strength and ductility values this configuration is one of the best for use in high seismic regions

Walls with Half Lap Joints

- Wall behaviour was influenced by the type of fasteners in the brackets and in the wall lap joint
Walls with Half-lap Joints

- Showed reduced stiffness and slight strength reduction
- Able to have the same or even higher ductility levels
- Shifted the occurrence of yield and ultimate load to higher deflection levels
- Wall 12 (SFS screws) showed increased ultimate deflection

 Longer Walls with Half-lap Joints

- Joints have more significant influence in longer walls
- Half-lap joints enabled CLT walls to carry a significant portion of the maximum load at higher deformation levels

Walls with WT-T Screws at Angle

- Showed lowest resistance from all walls
- Screws at an angle not recommended in seismic regions due to low energy dissipation and sudden pull-out failure mode

Two-storey CLT Wall Assemblies
Connection Deformation

- Concentrated in the bottom connections of the first storey
  - Uplift of 2.36” (60 mm)
  - Top connection uplift only 0.16” (4 mm)

Bottom Detail

Top Detail

Quasi-Static Tests on Two-storey CLT House

- Conducted at NIED Tsukuba
- Building was 7x7 m (23.3’) in plan and 10m (33.3’) high
  - Weight 50t (20t +30t)
  - Walls 85 mm (3.35”)
  - Floors 142 mm (5.7”)
- Designed with q = 1
- 3 Earthquakes (0.15 - 1.2g)
  - El Centro
  - Kobe
  - Nocera Umbra
- Total of 26 tests
  - 14 tests were ≥ 0.5g

The SOFIE Project

- Monotonic and cyclic tests on 3x3m CLT walls at IVALSA, Trento, Italy
- Pseudo-dynamic tests on full-scale single storey box-type specimen at University of Trento
- Full-scale shaking table tests on 3-storey CLT building at NIED, Tsukuba, Japan
- Full-scale shaking table tests on 7-storey CLT building at E-Defense, Miki, Japan

Shaking Table Tests on 3-storey Buildings

- Conducted at NIED Tsukuba
- Building was 7x7 m (23.3’) in plan and 10m (33.3’) high
  - Weight 50t (20t +30t)
  - Walls 85 mm (3.35”)
  - Floors 142 mm (5.7”)
- Designed with q = 1
- 3 Earthquakes (0.15 - 1.2g)
  - El Centro
  - Kobe
  - Nocera Umbra
- Total of 26 tests
  - 14 tests were ≥ 0.5g
Test Video - Configuration C with Kobe 0.8g

Hold-down Failure Modes

Shaking Table Tests on 7-storey Building
- Conducted at E-Defense in Japan
- Building weight 270t (120t + 150t)
- Wall panel thickness
  - 140 mm (5.5”) floors 1 and 2
  - 125 mm (4.9”) floors 3 and 4
  - 85 mm (3.3”) top 3 floors
  - All floor panels 142 mm (5.6”) ”
- Wall panels length 2.3 m (7.5”)
- Two 3-axial records used
  - Kashiwazaki Kariwa Quake
    - X (0.3g) Y (0.68g) Z (0.4g)
  - Kobe JMA Quake
    - X (0.6g) Y (0.82g) Z (0.34g)

Connecting Devices
- Connectors included steel brackets with screws and custom made hold-downs with screws
Building Response - Kobe Earthquake 0.82g

Test Observations

- The building showed satisfactory seismic behaviour under all severe earthquake motions
- Max top displacement of 287 mm (11.3") in X and 175 mm (6.9") in Y direction
  - Max storey drifts 2.4% (X) and 1.6% (Y)
- CLT buildings can satisfy not only life safety criterion in the building codes but also mitigate loss of property, as the damage after the tests was negligible
- Low residual deformations
- CLT buildings may be the first wood buildings to be used in high importance structures (more research needed)

Low Residual Deformations

Modeling - Kinematics Model (USA-Canada)

- Able to produce hysteresis loops for configurations of CLT walls with various input parameters:
  - Wall dimensions
  - Gravity load level and location
  - No. of brackets / H-d and location
  - Number of wall panels (in case of multi-panel walls)
  - No. of fasteners per bracket
  - Connection hysteresis parameters

\[ F(D) = \sum_{i=1}^{n} l_i \frac{f_{ii}}{H} \left( \frac{f_{ii}}{f_D} - 1 \right) \]

\[ f_D = \frac{H_D}{H} \]
Hysteretic Model Used for Modeling of the Connectors

- 10-parameter model developed in the CUREE project

![Graph](image)

Calibrating and Verifying the Model

Database of Calibrated Connector Parameters for the Model

- Brackets with 16d spiral nails (0.153” x 3.5”)
- Brackets with screws 0.16” x 2.75” (SFS1); 0.2” x 3.54” (SFS2)
- HTT 16 Simpson Strong Tie Hold-downs with nails

<table>
<thead>
<tr>
<th>Connector Type</th>
<th>Hysteretic Parameters (kN.m, kN)</th>
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<tr>
<td>SFS1</td>
<td>800</td>
</tr>
<tr>
<td>SFS2</td>
<td>1600</td>
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<tr>
<td>16E-SN with half-lapped</td>
<td>900</td>
</tr>
<tr>
<td>joint</td>
<td>900</td>
</tr>
<tr>
<td>SFS1 with half-lapped</td>
<td>1800</td>
</tr>
<tr>
<td>SFS2 with half-lapped</td>
<td>1800</td>
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</table>

Considered CLT Wall Configurations

- 4x8’ CLT wall configurations with various number of brackets

![Diagram](image)
Lateral Load Design Values for CLT Walls

- Design values taken as 40% of ultimate load (approx. 0.6% drift) compared to the maximum 3.5 to 4% drift

<table>
<thead>
<tr>
<th>Fastener Type: 16D-8N</th>
<th>Wall Length (R, L)</th>
<th>Single-panel Wall</th>
<th>Multi-panel Wall</th>
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</tr>
<tr>
<td></td>
<td>D</td>
<td>3.9</td>
<td>5.2</td>
</tr>
</tbody>
</table>

Values are ASD
For LFRD multiply by 1.6

Design Example - Wood-Frame Capstone Building

- The six storey building was located in a generic California site
  - Spectral Design Level: $S_{DS} = 1.0g$ and $S_{D1} = 0.6g$
  - Spectral MCE Level: $S_s = 1.5g$ and $S_1 = 0.9g$
- Buildings designed using the results of the numerical model and shear wall test data
- For preliminary evaluation of the R-factor, 4 different R factors were used (R=2, 3, 4, and 6)
- Performance of each building (design) was assessed numerically under a suite earthquake ground motions scaled to Design Base Earthquake (DBE) and Maximum Considered Earthquake (MCE) intensity
Wall Design Configurations

- Wall design configurations and total length selected to satisfy shear demands (in feet)

<table>
<thead>
<tr>
<th>Story</th>
<th>R=2</th>
<th>R=3</th>
<th>R=4</th>
<th>R=6</th>
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<td>Y'</td>
<td>Config.</td>
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<td>2DE</td>
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<tr>
<td>6</td>
<td>25</td>
<td>94</td>
<td>66</td>
<td>25</td>
</tr>
</tbody>
</table>

Model for System Level CLT Simulation

- The model first introduced by Pei and van de Lindt (2009) to simulate response of wood-frame buildings was incorporated in SAPWood v2.0
- CLT diaphragms modeled as rigid plates with 6 DOF
- Resistance of CLT walls modeled with horizontal hysteretic springs
- Overturning restraint modeled with linear elastic vertical springs

CLT Building Modeling with SAPWood

- Example of modeled CLT wall on storey 2

Time History Analyses of the CLT Buildings

- Ground motions: 22 bi-axial far-field records from FEMA P-695 scaled so that average response spectrum matches the design response spectrum at DBE and MCE level
- Ground motions were rotated by 90 deg. so that every CLT building was subjected to 44 biaxial motions for each R-factor design and each hazard level
- For each analysis, the maximum inter-storey drift at any storey and in any direction was recorded
- Fragility or Cumulative Distribution Functions (CDF) curves were developed
Concluding Remarks

- The research shows that CLT construction has adequate seismic performance when slender dowel fasteners are used with the steel brackets and hold-downs
- CLT wall panels behave as rocking rigid bodies with all deformation coming from the connections
- Properly designed CLT structures with symmetrical plans can achieve limited damage under strong quakes
- Use of half-lap joints in longer walls is effective solution to reduce the wall stiffness (seismic input load) and improve wall deformation and ductility properties
- Use of diagonally placed screws to connect walls to the floor is not recommended in seismic zones

Concluding Remarks

- CLT construction is not susceptible to the soft storey mechanism as the wall panels are virtually left intact in place even after a “near collapse” state is reached
- Almost all walls in one storey contribute to the lateral resistance providing additional degree of redundancy
Concluding Remarks

- Capacity based design shall be used for seismic design of CLT structures
- For the building example with slender fasteners, R=2.0 provides less than a 10% probability of exceeding 3.5% drift
- A P695 study is in progress to develop seismic design coefficients for CLT structures in the US
- Because of its strength and robustness, CLT has high potential for use in high wind regions

Thanks for Your Kind Attention