Collapse Fragility of a 5-story CLT Structure under Chilean Subduction Earthquake Records

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ABSTRACT: Mass timber structures have been used in North America, Europe, and Oceania, and are currently being evaluated in Latin America for midrise buildings in order to reduce the housing deficit and the contribution of the construction industry to greenhouse gas emissions. In highly seismic-prone areas, it is essential to provide resilient timber structures where structural and non-structural components adequately protect life safety and limit earthquake-induced damage and repair costs. In this paper, the Performance-Based Earthquake Engineering (PBEE) framework is employed to assess the probability of collapse of a government-subsidized 5-story platform-type cross-laminated timber (CLT) residential building designed under the draft of the Chilean seismic design code for CLT structures. The building model involved a full 3D representation of the building structure. Wall-to-foundation/floor connections and the overturning restraint system (i.e. hold-downs) were modeled explicitly to represent their hysteretic response. The collapse assessment determined a probability of 8.5% at the spectral acceleration of the predominant period (at 2% in the 50-year ground motion intensity) for a building with the mentioned characteristics, making it suitable for construction in Chile. Further research is needed to achieve loss estimation under the PBEE framework, such as quantifying damage fragility curves of representative engineering details for Chilean construction.

1. INTRODUCTION

Cross-laminated timber (CLT) is a mass timber product increasingly used in the construction of midrise buildings, as a lower carbon alternative to the traditional materials like concrete, steel, and masonry (Breneman, 2009). The usual construction system in CLT buildings is of platform type, consisting of walls and floors/roof composed of solid timber panels with high stiffness and strength in their plane, attached to each other through metal brackets and fasteners (González et al., 2018). In this type of construction, the floor panels are supported by the walls of the lower story, therefore the walls are interrupted at each level (Tamagnone et al., 2018). According to Izzi et al. (2018), the main source of energy dissipation comes from the wall-to-wall parallel connections (see label 3 in Figure 1), wall-to-foundation (see label 2 in Figure 1), wall-to-floor connections (see label 4 in Figure 1), and the overturning restraint system (see label 1 in Figure 1). The wall-to-wall parallel connections are found only in segmented (rather than monolithic) wall systems.

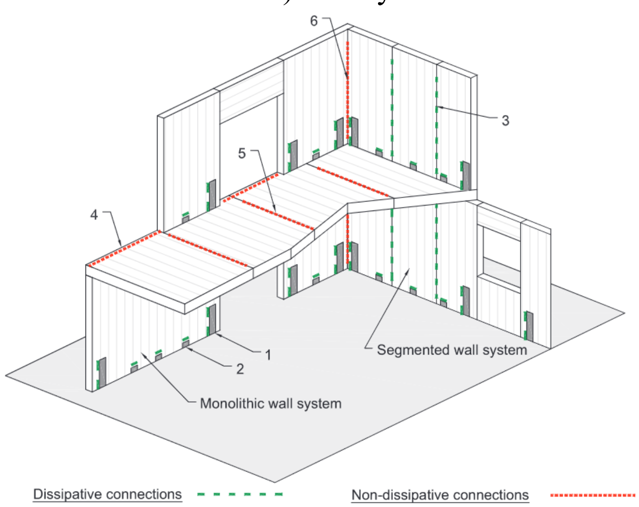

Figure 1: Schematic of a CLT structure (platform system) with an indication of the dissipative and non-dissipative connections (taken from Izzi et al., 2018)

In Chile, use of CLT is very limited, unlike Europe and North America, where midrise and highrise buildings have been built using CLT panels as the main material for walls and floor diaphragms (e.g., Wells 2011; Abrahamsen 2016; Poirier et al. 2017). However, recent studies (e.g. Benedetti et al. 2017; Perez et al. 2017; Valdivieso et al. 2019; Villegas and Valdivieso 2021; Benedetti et al. 2021) seek to promote the use of the CLT platform system using radiata pine grown in Chile as the raw material. These studies have proposed design recommendations such as seismic performance factors (i.e., seismic response modification factor R = 2.0 and story drift limit of 0.2%). The seismic performance factors were developed based on the comments of the Chilean building seismic design standard (NCh433, 2009). A story drift of 0.2% aims to minimize the non-structural damage after an earthquake and the R coefficient is computed as the ratio between the base shear in the structure close to the collapse divided by the elastic base shear according to NCh433. These design procedures are awaiting incorporation in national design codes. Considering the high seismicity of the country and the lack of formal code-based recommendations, it is essential to provide resilient timber structures where structural and non-structural components adequately protect life safety and reduce earthquake-induced damage and repair costs.

Performance-Based Earthquake Engineering (PBEE) allows for assessing the risk associated with seismic hazards, potential damage, and economical losses according to the location where they are built. In this paper, the PBEE framework is employed to assess the probability of collapse of a government-subsidized 5-story cross-laminated timber building for housing designed under the Chilean seismic design code considering the proposed seismic performance factors developed in Valdivieso et al. (2019).

2. BUILDING DESCRIPTION

The building, which is in the design stage, was defined for government-subsidized housing in Chile and illustrated in Figure 2. The building has five floors with a height of 2.6m, with two apartments on each level (55 m^2 per apartment). The staircase that provides access to the apartments is located in the middle. If constructed, the building will be built on a typical soil type in the city of Santiago (i.e., V_{s30} > 350 m/sec).

The gravity design of the building was based on the recommendation by Gonzalez et al.

(2014) and Guindos (2019). The lateral design was based on the reduced spectrum design (i.e., lateral force is reduced R times) of the NCh433 (2009). The design considers a seismic response modification factor R = 2.0 and story drift limit of 0.2% (i.e., at the reduced force design level), as has been proposed for the Chilean standard for this kind of building. The lateral system considers a monolithic wall system with hold-downs and shear keys as overturning and sliding restraint systems, respectively. The design leads to floors and walls made of 3-ply CLT (i.e., 120mm of thickness), using Chilean radiata pine as raw material. Each wall considered two hold-downs (e.g., Simpson Strong-Tie HHDQ14 model at the ground level) and two shear keys (e.g., Simpson Strong-Tie HL55G at the ground level) as illustrated in Figure 3.

3. PERFORMANCE CRITERIA

For the evaluation of the building performance under the PBEE framework, the guidelines reported by the Chilean Association on Seismology and Earthquake Engineering (ACHISINA, 2017) were used, even though it was not developed considering timber structures. At the design level earthquake (DLE), i.e., ground motion intensity with return period of 475 years, the mean (i.e., across all ground motion records) peak (i.e. across all stories in the two directions) story drift ($mIDR_{max}$) should be smaller than 0.5% (i.e., for immediate occupancy). However, this limit is based on concrete-based buildings. For the maximum considered event (MCE), i.e., ground motion intensity with return period of 2475 years, collapse risk is used as a proxy for life safety, and is measured in terms of the collapse margin ratio (CMR), following an approach similar to FEMA P-695. Furthermore, for achieving NCh433 goals at the MCE level (i.e., even structures show damage, avoid collapsing during an earthquake of exceptionally serious intensity), the probability of collapse at the MCE level should be smaller than 10% (Medalla et al., 2020). For this study, the MCE ground motion intensity is defined at a return period of 2475 years (In Chile, the MCE level varies depending on the code).

Figure 2: Building considered: (a) typical floor plan, (b)exterior longitudinal elevation, (c) inside longitudinal elevation, and, (d) transverse elevation. All dimensions in millimeters

4. NUMERICAL MODELING

In order to evaluate the response of the building and the possible system effect on the response of the structure, a full 3D representation of the building structure was developed. The numerical model was developed in OpenSeesPy (Zhu et al. 2018). The distribution of the CLT shear walls on the numerical model is illustrated in Figure 3 where the typical wall is enclosed in a dashed line. For the typical wall, shell elements (i.e. shellMITC4 element) with an isotropic material were used for the CLT wall panels and nonlinear springs (i.e., zeroLength element) were used for the hold-downs and shear keys in order to capture hysteretic damping (see Figure 4). The distribution of the nonlinear spring elements for representing the story-to-story connections at the level "n" is illustrated in Figure 5. These connections are responsible for the energy dissipation in the monolithic wall system (Izzi et al. 2018). The wall-to-wall perpendicular connection was considered non-dissipative and perfectly rigid. The test data reported by Perez et al. (2017) on full-scale CLT shear walls and by Matus & Salgado (2016) on hold-downs and shear keys were considered for the calibration of the numerical model. The hysteretic response of those connections was modeled explicitly by calibrating the SAWS constitutive from test data through an error minimization procedure. When a hold-down connection is subjected to tensile loads, its behavior can be described by the SAWS model. However, under compression loads, its behavior is characterized by high stiffness and strength due to the contact of the CLT panel with the foundation. To reproduce this effect at the ground level, an Elastic-No Tension material was combined in parallel with the SAWS material, achieving a new material with hysteretic behavior in tension and high strength and stiffness in compression.The model guidelines described in Villegas & Valdivieso (2021) were used as a base for the development of the models.

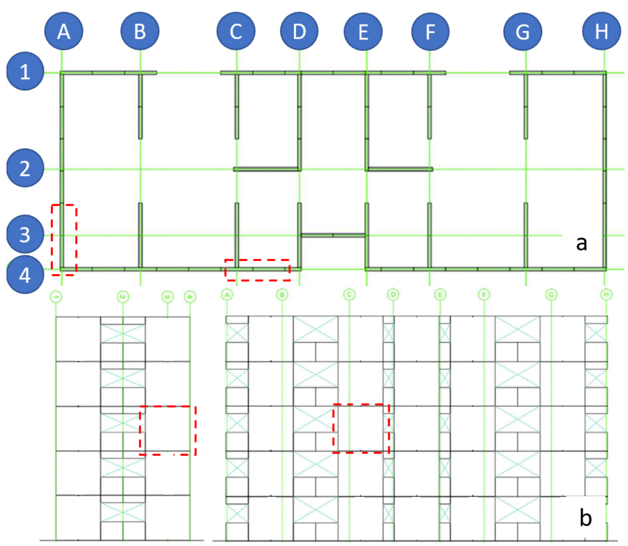

Figure 3: CLT shear wall distribution in (a) plan and (b) elevation (along A-A and 1-1).

Based on Jayamon et al. (2018), 2% of inherent damping was considered and created with Rayleigh damping models. Based on previous numerical studies (Shahnewaz 2020), the floor and the roof were modeled as rigid diaphragms with mass lumped at each story for both models.

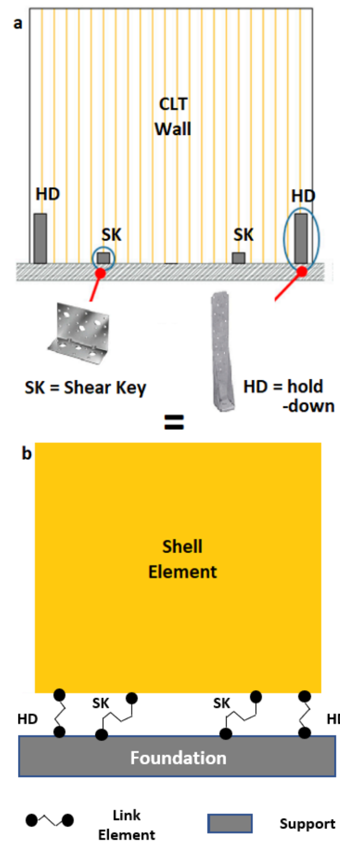

Figure 4: (a) Typical CLT shear wall configuration used in the building, (b) schematic of FE model for CLT shear walls

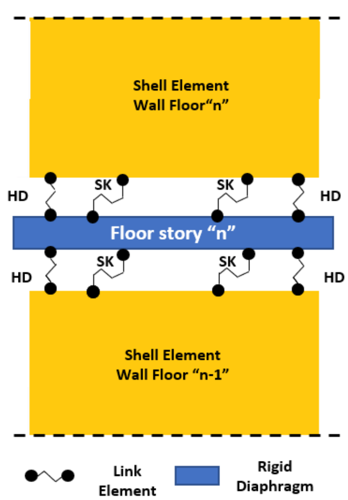

Figure 5: Schematic of FE model for CLT shear walls at the story "n"

The first fundamental periods of the nonlinear building model at both translational directions were determined as 0.29 sec and 0.26 sec. The first mode is predominately in the X direction and the second in the Y direction (see Figure 6).

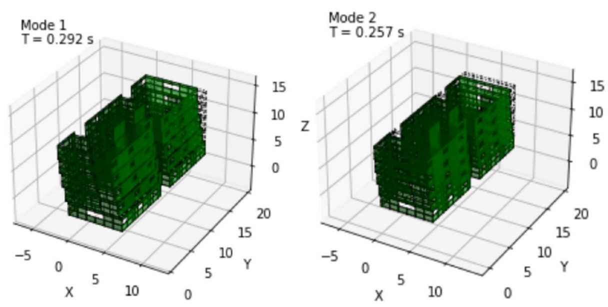

Figure 6: Mode shape of the structure.

5. GROUND MOTION SELECTION

Chile is located in a highly seismic-prone area. Megathrust earthquakes are mainly attributable to the subduction of the Nazca plate beneath the continental South American plate. These subduction sources are considered as the only source of earthquake hazard to perform nonlinear time history Multiple Stripe Analysis (MSA).

A probabilistic seismic hazard analysis (PSHA) was conducted to compute the seismic hazard associated with the site location of the building. For the development of the PSHA, the recurrence model developed by Poulos et al. (2019) and the Ground-motion prediction equation (GMPE) for the Chilean subduction zone developed by Montalva et al. (2017) were used. As Intensity Measure (IM), the spectral acceleration at the predominant mode period was used (i.e., Sa(Tx) and Sa(Ty)). An example of the seismic deaggregation for the site of the building (i.e., located in Santiago with a V_{s30} > 350 m/sec) is presented in Figure 76.

Ground motion records from the SIBER-RISK database were selected to match the target Conditional Mean Spectrum (CMS) for the mean scenario on the predominant modes period of the structure. The procedure developed by Jayaram et al. (2011) and a maximum scale factor of 2.0 were considered. Figure 8 shows the response spectra of all selected motions scaled to the CMS.

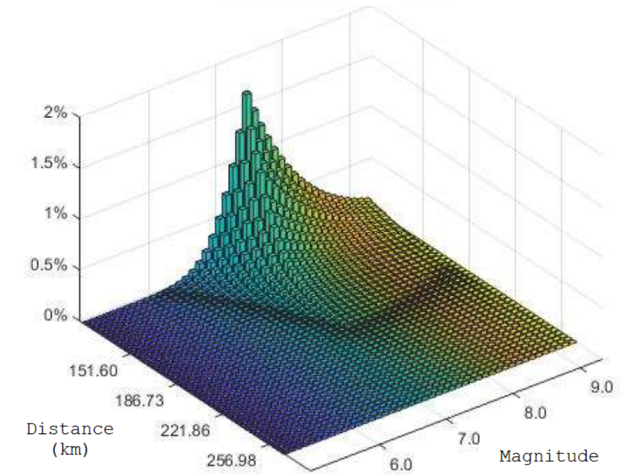

Figure 7: Deaggregation for the considered site associated with the first mode of the 3D structure.

Table 1: Deaggregation results.

Mean Scenario	*Period (sec)*	
	0.29	*0.26*
Magnitude	*7.83*	*7.83*
Distance (km)	*147.3*	*145.8*

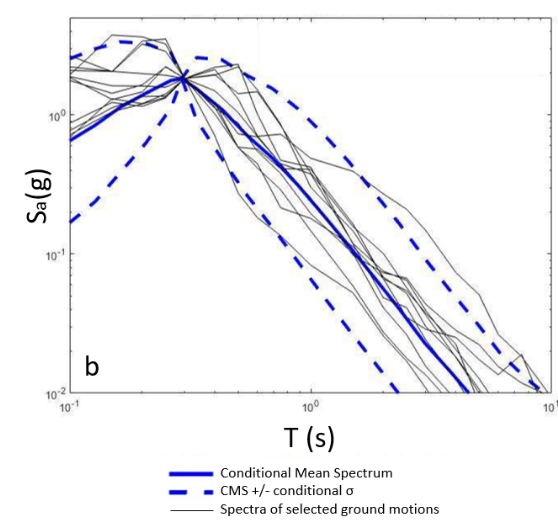

Figure 8: Response spectra of selected motions matched to the target spectrum (X-direction component) for the 3D building model.

6. RESULTS AND DISCUSSION

Multiple Stripe Analysis was performed considering the peak story drift ratio (IDR_{max}) as the engineering demand parameter (EDP). The story drift ratios were considered as a damage measure to assess the structural response under earthquakes. By IDR_{max} we refer to the peak IDR across the stories and directions. Figure 9 shows the MSA curves of all ground motions. For the MSA, 7 stripes were considered (i.e., 0.5g, 1.0g, 1.5g, 2.0g, 3.0g, 4.0g, and, 5.0g). The Sa(T) at the DLE and MCE levels are 0.79g and 1.32g, respectively.

Based on MSA results, the $mIDR_{max}$ at the design level is 0.65%. The result showed that the building could not fulfill the requirements of the ACHISINA guideline as it was developed for concrete buildings. However, at the $mIDR_{max}$, hold-downs and shear keys present almost 4 mm of deformation which is below the yield limit (i.e., 13.7 mm for shear key and 31.9 mm for hold-down) reported by Matus & Salgado (2016) demonstrating that even if the building could not achieve the allowable story drift, the system can achieve the ACHISINA guideline goal at the design level. As was previously reported by Villegas & Valdivieso (2021), the main contributor to the lateral deformation of the wall was racking deformation.

The collapse fragility curve was developed based on the maximum likelihood method by Baker (2015), see Figure 10. As limited test data is available for CLT platform building using radiata pine as raw material, the collapse state is not well defined. Benedetti et al. (2021) reported that for drift higher than 2.5% the stability of CLT platform-type system may be compromised while Shahnewaz et al. (2020) recommended a range between 5% to 10% as the collapse criteria. As a conservative approximation, a 3.5% IDR was considered. The median collapse capacity (50% probability of collapse) was determined. For the first mode of the 3D building model, the $S_{CT(Ty)}$ = 3.31g. Note that at this level, damage was concentrated first at the hold-down followed by damage at the shear keys which is consistent with damage patterns reported by Perez et al. (2017).

For the MCE level, the collapse probability is 0.5% and less than 0.1% for the 2D and 3D numerical models of the structure, respectively. Considering the total system collapse uncertainty ($_{TOT}$ = 0.66) resulted from the uncertainties related to record-to-record (i.e., $_{RTR}$ = 0.4), design requirements (i.e., quality rated as good by FEMA P-695, $_{DR}$ = 0.2), test data (i.e., quality rated as fair by FEMA P-695, $_{TD}$ = 0.35), and modeling (i.e., quality rated as fair by FEMA P-695, $_{MDL}$ = 0.35), the collapse probability is 8.5% for the MCE scenario, which is smaller than the limit defined (i.e., 10%) for achieving NCh433 goal. This is mainly attributable to the strict design requirements of the Chilean seismic design code (i.e., mainly the limit story drift limit of 0.2%).

The CMR is an important output from the MSA, defined as the ratio between median collapse capacity and the maximum considered earthquake intensity in the Chilean code (i.e., 2% in 50 years) S_{MT}:

$$CMR = \frac{S_{CT}}{S_{MT}} \tag{1}$$

For the first mode of the 3D building model, $S_{MT(Ty)}$ = 1.32g. Then, the computed CMR is 2.51. which is considered equal to the ACMR. According to Tesfamariam et al. (2021), the ACMR factor does not apply for a site-specific hazard. Based on the results, the computed CMR is higher than the FEMA P695 limit (i.e., ACMR10% = 2.34 for $_{TOT}$ = 0.67), indicating the building satisfied the criterion.

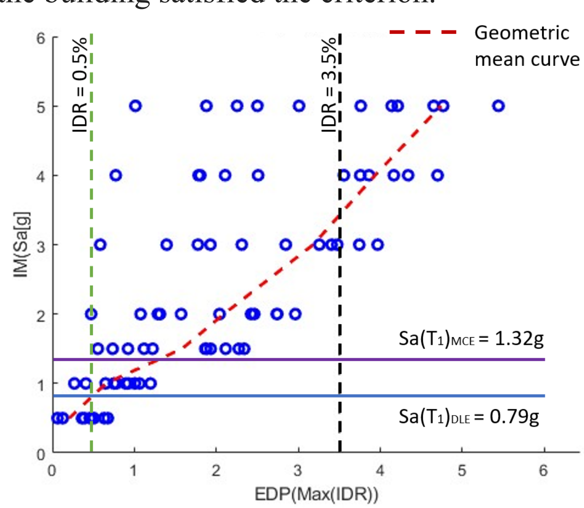

Figure 9: MSA curves of all ground motions for the 3D building model.

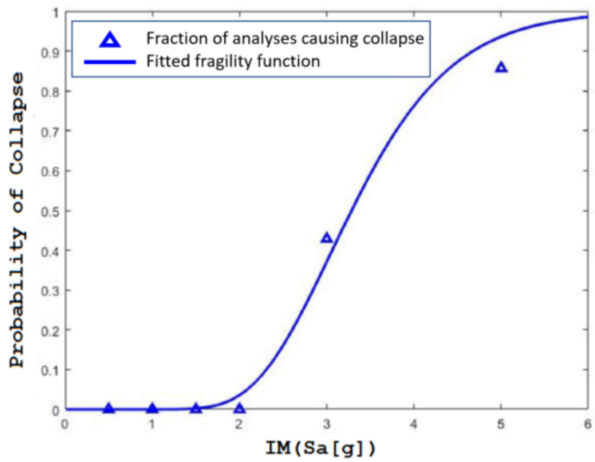

Figure 10: Fragility curve for collapse for the 3D building model.

7. CONCLUSIONS

The study evaluated the lateral seismic behavior and collapse capacity of a government-subsidized 5-story cross-laminated timber building designed under the Chilean seismic design code. The Performance-Based Earthquake Engineering (PBEE) framework was used to assess the performance and collapse fragility curve of the building. Based on the analysis results, the structure can withstand DLE with limited damage to structural components meeting ACHISINA criteria (i.e., IDR almost 0.5%). For the MCE hazard scenario, an acceptable collapse probability (i.e., 8.5%), and CMR factor (i.e., 2.51) were found.

The results support the proposed seismic performance factors (i.e., R = 2.0 and maximum story drift of 0.2%) for the design of platform-type CLT buildings with monolithic shear walls.

Future research is needed to achieve a full PBEE analysis where decision variables, such as estimated economic losses, could be given. The next steps in the Chilean research effort should be focused on defining appropriate limits at the DLE and MCE as well as tracking damage levels in representative structural components considering non-structural finish layers.

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