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ALTERNATIVE SEISMIC DESIGN OF MULTI-STOREY TIMBER STRUCTURES

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ABSTRACT: Research on the structural response of large, multi-story timber buildings under horizontal loading conditions, such as wind and earthquakes, remains limited in the current literature. In particular, there is a need for improved understanding of the nonlinear behavior of these structures and how energy dissipation mechanisms influence their seismic performance. This study focuses on nonlinear time history analysis of multi-storey timber buildings, emphasizing the role re-centering mechanism in the seismic response. A set of 3D timber building configurations was designed according to Eurocode force-based design principles and subsequently evaluated using nonlinear dynamic simulations. The results show that the early stage design choices, such as how the vertical load will be distributed, whether the CLT walls will contribute to that or not, and whether a self-centering mechanism will be used, can fundamentally change the overall seismic behavior of the structure in the non-linear phase. Key insights are summarized regarding the weight acting on the CLT walls in relation to two common hold-down response mechanisms, elastic-perfectly-plastic self-centering.

KEYWORDS: timber floor, vibration control, human induced vibrations, experiments

1 – INTRODUCTION

The growing adoption of engineered timber systems, such as cross-laminated timber (CLT) and glued-laminated timber (glulam), in multi-storey buildings has introduced new opportunities for sustainable and seismic-resilient construction. However, despite their increasing use, the understanding of the seismic behavior of large-scale timber buildings remains limited, particularly concerning energy dissipation mechanisms and system-level response under horizontal loading [1, 2] as well as the validity of the code-based design, which is mainly controlled by a behaviour factor (q, in the case of Eurocodes). In timber structures, energy dissipation occurs primarily at connections, such as panel-to-panel joints, hold-downs, and foundation interfaces, rather than through material yielding at the load-bearing elements, as is common in steel or reinforced concrete systems [3]. This leads to a complex hysteretic response that is highly sensitive to connection detailing, construction tolerances, and interaction effects. Additionally, the role of soil–structure interaction (SSI) in the seismic performance of timber buildings has been largely overlooked in experimental studies, despite having the potential to be a key factor in the overall structural response [4]. While there have been efforts to integrate damping systems, such as buckling-restrained braces and friction-based connectors, into timber structures [5, 6], these innovations have not yet been widely validated through large-scale experimental testing. Furthermore, due to limitations in laboratory capacities and the logistical challenges of testing entire buildings, experimental investigations are often constrained to single components or planar frames, making it difficult to understand system-level performance and, most importantly, fundemental issues such as self-centering capacity and the actual hierarchy among the energy-dissipating parts of the building. This paper presents the structural design of a typical five-storey mixed-use (i.e. residential and office) timber building, carried out according to Eurocode provisions [7]. The design adopts a behaviour factor (q) to represent the global ductility and energy dissipation capacity of the structure, as is common in code-based seismic design practice. A key aim of this study is to investigate differences between two cases; first, CLT walls are contributing to carrying the vertical loads and using them as re-centering purpose, and second, the vertical loads in the tributary area of the CLT walls are taken by the columns placed right next to the walls, and the re-centering is provided by a self-centering hold-down system. In terms of design, both options are treated similarly since the design is essentially elastic and relies on a simple q factor (i.e. behavior factor).

2 – METHODOLOGY

2.1 STRUCTURAL DESIGN APPROACH

The structural design in this study represents a five-storey mass timber building intended for mixed-use occupancy. The ground floor is assumed to serve office function, while the remaining upper stories are designated as residential. The building is located in a moderate seismic

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hazard region (Athens, Greece), with design actions derived from Eurocode 8 [7]. The design philosophy follows a capacity-based approach in which energy dissipation is concentrated at connections and base restraints, while vertical load-bearing members are detailed to remain elastic.

The selected system is 23mx23m in plan dimensions, with 3.2m height in the ground floor and 3.0m height in the upper floors. The section of a load bearing frame is shown in Figure 1.

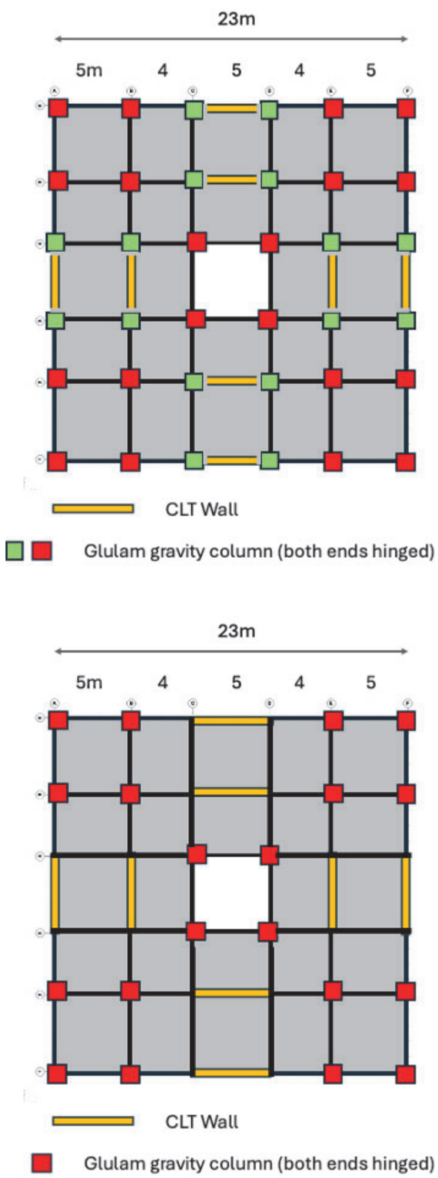

Figure 1: Plan view of the designed structure

The lateral load-resisting system consists of cross-laminated timber (CLT) wall panels, connected to the foundation using either EPP (elastic-perfectly-plastic) or SC (self-centering) hold-downs and connected to each other vertically through energy-dissipating steel elements as shown in Figure 2. Vertical loads are carried by glulam columns that are assumed to be pinned at both ends. The structural configuration was selected to represent modern timber construction practices that emphasize prefabrication, modularity, and dry assembly techniques [3].

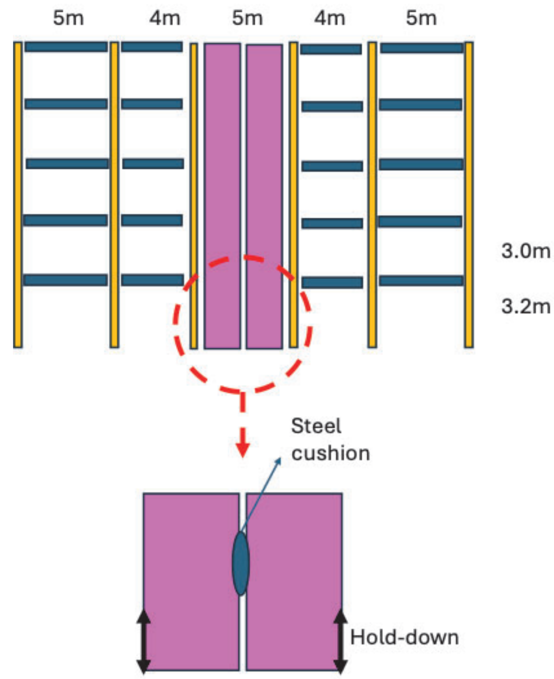

Figure 2: Section view of one of the lateral load bearing frames of the designed structure

Key design parameters include:

- Seismic design peak ground acceleration (PGA): 0.16g (475-year return period)
- Soil type: Type C (medium stiff soil)
- Behavior factor (q): 4.0
- Fundamental periods (Eurocode spectrum): Tb = 0.2s, Tc = 0.6s, Td = 2.0s, see Figure 3
- Wind speed: 33 m/s (Eurocode 1 basic wind velocity)

The design process ensures that the building remains within acceptable force and deformation limits of the energy dissipating elements, while the timber elements remain elastic.

2.2 MATERIAL PROPERTIES

The structural design of the building utilized material properties representative of commonly used engineered timber products in Europe. The glulam columns were specified as GL30c class, with a characteristic compressive strength parallel to the grain of $f_{c,0,k} = 24.5$ MPa and a 5th percentile modulus of elasticity $E_{0.05} = 10.8$ GPa. The unit weight of glulam was taken as $\gamma_{\text{glulam}} = 4.22$ kN/m^3. For the wall panels, cross-laminated timber (CLT) of strength class C30/T21 was assumed, with characteristic strengths in in-plane bending, tension, and compression of $f_{m,k} = 30$ MPa, $f_{t,k} = 19$ MPa, and $f_{c,k} = 24$ MPa, respectively. The modulus of elasticity for the CLT panels in the major bending direction was taken as $E = 12$ GPa.

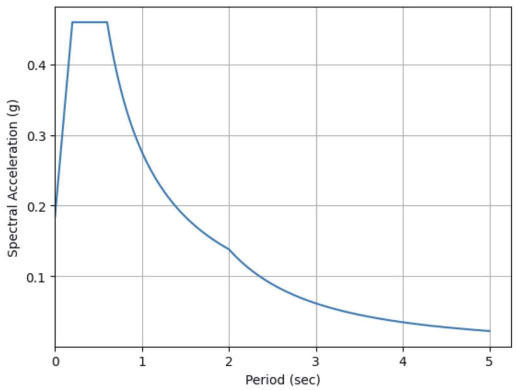

Figure 3: Design acceleration response spectrum

For floor build-up layers, material densities were selected as follows: 6.02kN/m^3 for OSB boards, 16.7kN/m^3 for anhydrite-gypsum topping, and 9.81kN/m^3 for final finishes. A nominal unit weight of 0.26kN/m^2 was assumed for timber joists, and partition walls were modeled as uniformly distributed loads of 0.5 kN/m^2. The modulus of elasticity for rigid links and gap elements used in modeling hold-downs and braces was assumed to be $E = 300$ GPa, representing steel components. These values were used consistently in both the seismic mass estimation and the nonlinear response simulation.

2.3 LOAD AND MASS CALCULATIONS

The seismic weight and vertical load components of the five-storey mass timber building were calculated based on material properties and assumed architectural finishes. The floor self-weight per unit area was computed by summing contributions from 20mm OSB boards, a 40mm anhydrite-gypsum topping, timber joists (estimated as 0.26kN/m^2), partition walls (0.5kN/m^2), and final finishes (20kg/m^2), resulting in a total floor weight of approximately 1.66kN/m^2. Vertical load-bearing glulam columns were sized as 360 ×280mm for the ground and first storeys, and 270 ×280mm for the upper levels, based on typical GL30 glulam configurations. Column self-weights were increased by 5

The axial load on the ground floor columns, incorporating $1.35G + 1.5Q$ and accounting for increased live loads on the first floor (due to assumed office and storage use), was estimated to be approximately 531.66kN. For the second-storey columns, the total axial load was approximately 356.86kN. Masses for the nonlinear dynamic analysis were defined per storey, corresponding to seismic masses of $M_1 = 28.0$ t, M_2–$M_4 = 16.3$ t, and $M_5 = 11.7$ t, and were distributed across wall panels accordingly.

Lateral seismic forces were derived using the Eurocode8 design spectrum for soil type C, with a peak ground acceleration (PGA) of 0.16g, a behavior factor $q = 4$, and a fundamental period of vibration $T_1 = 0.50$s, yielding a spectral acceleration $S_a(T_1) \approx 0.46g$. The total base shear demand was computed from the effective seismic weight and spectral acceleration and distributed among the storeys using an inverted triangular distribution based on mass and height. The design base shear is 99.95kN, and the designoverturning moment was calculated as approximately 1124.89 kNm, providing the basis for evaluating lateral resistance contributions from CLT panels, energy dissipators, and hold-down devices.

2.4 DESIGN CHECKS FOR THE CLT PANELS

The in-plane bending capacity of the CLT wall panels was verified under combined axial and flexural actions. The maximum axial force in the CLT panels was computed from nonlinear static analysis, reaching up to 82.5kN, while the maximum bending moment was approximately 73.6kNm. The cross-section properties were calculated using an effective inertia approach, assuming that four-sevenths of the panel thickness contributes to bending stiffness, in line with simplified composite behavior assumptions. The resulting axial and bending stresses were superimposed to evaluate tension and compression demands on the critical faces. A slenderness check was also carried out, yielding a relative slenderness ratio of 0.287 for the CLT panel. Since this value is less than 0.3, no reduction was applied to the design strength. The design compressive and tensile strengths were calculated using a material factor of 1.25 and assuming instantaneous loading (i.e. $k_{\text{mod}} = 1.0$). The panel was found to be adequate in both compression and tension, with demand-to-capacity ratios of 0.76 and 0.59, respectively. This is inline with the design assumption that the CLT panles will remain elastic during the seismic excitation.

2.5 DESIGN CHECK FOR THE GLULAM COLUMNS

The axial design of the glulam columns was carried out considering both the compressive strength and slenderness effects. Two cross-sectional sizes were adopted: 360 ×280mm for the ground and first storeys, and 270 ×280mm for the upper floors. Axial loads were computed for each case using a load combination of $1.35G + 1.5Q$, resulting in design demands of 531.66kN for the ground floor and 356.86kN for the second storey. The columns were checked for slenderness effects using relative slenderness ratio calculations. For the ground storey columns, the slenderness ratio was found to be 0.335, which exceeds the threshold of 0.3. As a result, the design strength was reduced by a buckling factor k_c, derived using Eurocode 5 expressions. The reduced design compressive strength was then used to calculate the axial capacity of each column. The demand-to-capacity ratio for the ground floor columns was 0.86, indicating a safe but relatively efficient design. Similarly, the upper storey columns were found to be adequate with a demand-to-capacity ratio of 0.68.

2.6 DISSIPATOR FORCES

Energy-dissipating elements were assumed to be installed between adjacent CLT wall panels in the form of zero-length links that mimic friction-based or metallic yielding devices. These are designed as steel cushions as shown in Figure 4. Steel cushions have been shown to exhibit stable hysteretic behavior under combined actions as shown in [8].

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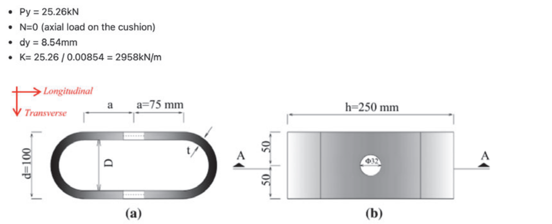

Figure 4: Steel cushions used as energy dissipators between the CLT panels, placed vertically at the panel-to-panel intersection, modified from [8]

The nonlinear static analysis revealed that the maximum force transmitted through the dissipators was 47.7 kN. These elements are critical in the lateral load-resisting system, as they contribute to energy dissipation and help enforce a favorable plastic mechanism in the building. The moment equilibrium analysis indicated that the dissipators carried approximately 56% of the total overturning moment, while the CLT panels themselves resisted the remaining 44%. This distribution highlights the effectiveness of the connection design in promoting controlled energy dissipation and minimizing stress concentrations in the timber panels.

2.7 TESTED DESIGN VARIATIONS

Two design variations have been tested. It is important to note that, during the elastic design phase, the period of vibration of the two design cases were similar (0.50sec), while the initial stifnesses of the hold-downs were also similar, leading to seemingly identical design for both cases. The nonlinear behaviour of the hold-downs, however, are fundamentally different since one is an EPP hold-down while the other one is a SC.

In order to achieve a passive self-centering mechanism, the vertical load is directed to the CLT, coming from the tributary area around it, as shown in the bottom figure in Figure 1. The self-centering is taken care of by the SC hold-downs. In the top figure of Figure 1, the vertical load coming from the tributary area of the CLT walls is directed to the columns that are placed right next to the edges of the CLT walls (green columns). Although this configuration allows the CLT to carry its own weight only, removing several design issues for the design of the CLT itself, it comes with another problem, that is the compatibility of the CLT acting as cantilever and the frame, especially in the upper floors. This will be addressed further in the paper.

3 – NONLINEAR TIME HISTORY ANALYSES

Although the seismic design of the structure follows a force-based approach using elastic properties and a global behaviour factor (q) as prescribed by Eurocode 8, the actual structural response under strong ground motion is inherently nonlinear. In the case of timber buildings, this nonlinearity is not distributed throughout the structural elements but is instead concentrated in discrete components such as panel-to-panel connections, dissipative cushions, and hold-down devices. These elements govern the ductility and energy dissipation capacity of the system, which the q-factor is intended to represent in simplified design procedures. While nonlinear analysis is not explicitly required in the standard design workflow, it is employed here to validate the assumptions of the elastic design and to assess whether the intended plastic mechanisms and deformation capacities are realistically achieved under dynamic loading. Nonlinear time history analyses thus provide insight into the consistency between the code-based design assumptions and the actual inelastic performance of the structure.

3.1 MODELING ASSUMPTIONS AND LIMITATIONS

The structural model was developed in OpenSees [9] using a two-dimensional representation of the wall system in the primary direction of seismic loading. The model focuses on a single frame consisting of two adjacent CLT wall panels per side, capturing the critical vertical and lateral load paths. Floor diaphragms were not explicitly modeled; instead, it was assumed that the floor system provides sufficient in-plane stiffness to distribute seismic forces evenly among the wall lines. The analysis considers mass lumped at each storey level, concentrated at nodes representing the center of mass of the wall panels. Linear-elastic behavior was assumed for the CLT panels and glulam columns, while nonlinearities were concentrated in discrete connection elements, such as hold-downs and energy dissipators, modeled using zero-length link elements with elastic-plastic or flag-shaped behavior.

The model does not include soil–structure interaction (SSI); the foundation was assumed to be perfectly rigid with fixed boundary conditions at the base of the walls. Torsional effects and out-of-plane responses were not considered, due to the symmetry of the plan layout and the assumption of uniform distribution of stiffness and mass. The vertical load from the building weight was included in the model, but live loads were reduced using participation factors per Eurocode recommendations. The structural analysis included equivalent static loading and nonlinear time history analyses; however, the latter were limited to a subset of ground motions and may not fully capture variability in near-fault effects or directionality. Despite these simplifications, the model captures the primary nonlinear response mechanisms and offers useful insight into the force distribution and energy dissipation hierarchy within the system.

3.2 DYNAMIC LOADING AND TIME HISTORY RECORDS

To evaluate the nonlinear seismic performance of the designed structures, a suite of seven recorded ground motions was selected and applied in time history analyses. In accordance with the provisions of the new draft Eurocode8 [7], recorded accelerograms were preferred over artificial signals, with selection criteria focusing on magnitude and source-to-site distance. All records were chosen from established strong-motion databases, with magnitudes (M_w) ranging between 5.5 and 7.0, and rupture distances less than 60km. The records were scaled to match the target elastic response spectrum defined in Eurocode 8 for soil type C and a PGA of 0.16g corresponding to the seismic hazard of Athens.

Following Clause D.3 of the new EN 1998-1, the spectral compatibility of the selected records was evaluated within the period range of $0.2T_1$ to $1.5T_1$, where $T_1 = 0.35$s and $T_1 = 0.50$s are the fundamental periods obtained from eigenvalue analysis of the OpenSees model for two different design options. This corresponds to a target compatibility range of 0.07s to 0.525s and 0.1 to 0.75s, respectively for the first and the second design options. The two sets provide a basis to examine sensitivity to period range selection and ensure compliance with spectrum-matching requirements for nonlinear response-history analysis. The acceleration response spectra of the selected records in respect to the design spectrum can be found in Figure 5.

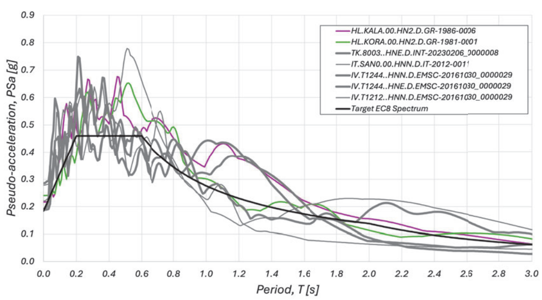

Figure 5: Acceleration design spectrum versus the selected records for Design Option 1, T1=0.35s (above) and Design Option 2, T1=0.50s (below)

3.3 MODELLING NONLINEAR RESPONSE OF CUSIONS AND HOLD-DOWNS

The nonlinear behaviour of the structure is concentrated at the connections, which were explicitly modelled using zero-length link elements in OpenSees to simulate energy dissipation and uplift restraint. Two primary types of connections were considered: hold-downs at the base of the CLT panels, and panel-to-panel dissipative devices located at each floor level. The hold-downs were modelled either using elastic–perfectly plastic materials, or with flag-shape hystsretic backbone with uplift stiffness and yielding capacity representative of proprietary self-centering steel devices, examples of which are shown in Figure 6. Zero-length elements acting in the vertical direction were used to represent these devices at the outermost corners of the wall panels. Additionally, a contact interface with tension-only gap behaviour was implemented to prevent the transmission of compressive forces across the same nodes, thus simulating a realistic separation mechanism under uplift.

Panel-to-panel dissipators were placed at mid-height of each storey between adjacent wall segments and were also modelled as zero-length elements with bilinear, elastic-perfectly plastic response in the horizontal direction. These connections emulate the behaviour of friction-based or yielding metallic dampers and were calibrated to develop significant force under interstorey drift while allowing for energy dissipation. Rigid links were added between the centreline of each CLT panel and the dissipative connection nodes to ensure realistic lateral force transfer and deformation compatibility. This modelling approach enables concentration of nonlinearity at discrete locations, consistent with the expected behaviour.

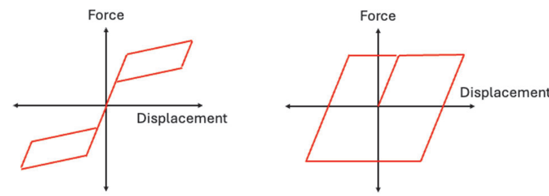

Figure 6: Example elastic-perfectly plastic (left) and flag-shaped (right) nonlinear spring response used for modelling the zero-length elements

4 – RESULTS AND DISCUSSION

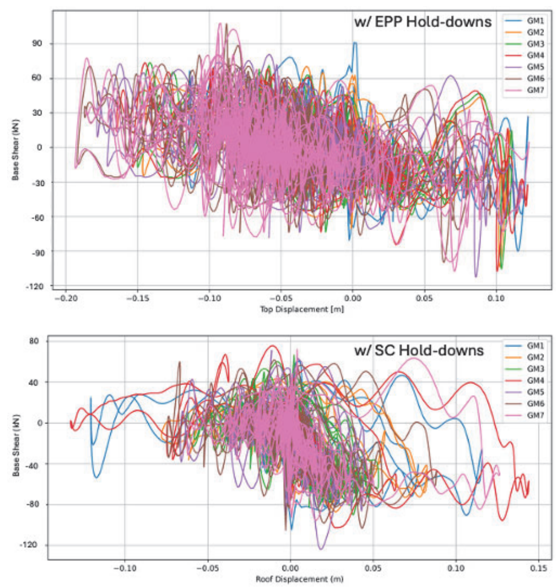

Figure 7: Roof displacement vs base shear plots from 7 records for the self-centering (top) and elastic-perfectly-plastic (bottom) hold-down configurations

The overall roof displacement versus base shear results from the nonlinear time-history analyses are shown in Figure 7. It can be seen that the EPP hold-down results in higher top displacements (up to 20cm) than the case of SC hold-downs (approx. 15cm). This is because, after the hold-down activation force for the SC, the secant stiffness is not zero, still not allowing larger displacements, while in the case of EPP, the stiffness is zero once the hold-downs yield.

The results of the nonlinear time history analyses also highlight the significant impact that early design decisions, particularly regarding vertical load distribution and the choice of hold-down mechanism, have on the seismic behavior of multi-storey timber structures. Despite both configurations exhibiting similar elastic design properties, such as fundamental periods and initial stiffness values, their inelastic responses under seismic excitation diverged notably. In the self-centering (SC) hold-down configuration, vertical loads were intentionally directed onto the CLT

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walls, thereby engaging the panels in a passive restoring mechanism. The nonlinear time history analyses were in agreement with the design base shear, while the total energy dissipating was smaller than its EPP counter-part (i.e. a thinner loop of base shear vs top displacement). This configuration exhibited more stable hysteresis loops and reduced residual deformations, indicating improved re-centering performance. Conversely, in the elastic–perfectly plastic (EPP) configuration, where vertical loads were primarily carried by adjacent glulam columns, the CLT panels acted more as cantilevers. This configuration introduced compatibility issues, particularly in the upper storeys, and resulted in larger residual drifts and a less consistent energy dissipation hierarchy. In contrast, the EPP (elastic–perfectly plastic) design exhibited wider hysteresis loops, indicating greater energy dissipation during seismic excitation. The noticeable difference in energy dissipation capacity between the two configurations suggests that assigning the same behavior factor (q) in the elastic design phase is not justified. This raises concerns about the adequacy of conventional force-based design approaches when applied to systems with fundamentally different nonlinear behaviors.

Moreover, the analyses also show that the distribution of overturning moments between CLT walls and dissipative cushions played a critical role in shaping the global seismic performance. In both configurations, dissipators absorbed a substantial portion of the energy, up to 56 percent of the total overturning moment. However, their effectiveness was strongly influenced by the axial force paths and deformation compatibility provided by the surrounding structural system. The SC configuration promoted a more favorable plastic mechanism by concentrating nonlinearity in the dissipators and minimizing stress demand in the timber elements. These findings emphasize that while force-based code design may treat different configurations as equivalent, their actual seismic response can vary significantly. It is important to note that the nonlinear time history analyses needs to be incorporated in the design of timber structures where ductility, energy dissipation, and post-earthquake serviceability are paramountly important, and cannot be obtained with simplistic elastic design.

5 – ACKNOWLEDGMENTS

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