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SEISMIC ANALYSIS AND DESIGN OPTIMIZATION OF LIGHT TIMBER FRAME VERTICAL ADDITIONS ON RC LOW-CODE BUILDINGS

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Abstract

Vertical additions to existing buildings offer economic and environmental advantages but pose concerns regarding seismic safety, particularly in aging structures lacking adequate seismic design. This study proposes a strategy to minimize the seismic impact of light timber frame (LTF) vertical additions (VA) while ensuring benefits such as modularity, prefabrication, and sustainability. A reinforced concrete (RC) archetype building, representative of 1970s Italian residential structures, serves as the case study. The design of a two-storey LTF VA is performed using a parametric optimization approach with a simplified equivalent 3-degree-of-freedom (3-DOF) model. Key design parameters include the number of timber walls and sheathing-to-framing nail spacing. The optimization process was conducted ensuring that the second story of the addition remained elastic, while the first story was allowed to yield and dissipate energy through hysteretic behaviour. Seismic performance was assessed using seven spectrum-compatible ground motions, comparing the as-built configuration of the case study building with its performance after the addition. The assessment was conducted via nonlinear time history analysis on both refined finite element (FE) models and the simplified 3-DOF equivalent model to evaluate the accuracy of the simplified approach. Findings reveal that optimized LTF VA can enhance seismic performance and reduce structural vulnerability compared to non-optimized configurations. While the simplified modelling approach has inevitable limitations, the study underscores the importance of well-designed timber vertical additions in improving seismic resilience of existing buildings.

Keywords: Vertical Extension, Vertical Additions, Light-frame Timber Structures, Equivalent SDOF Model, Seismic Resilience of Existing Buildings, Design Optimization.

1 INTRODUCTION

As urban populations continue to grow, the demand for additional housing and infrastructure is rising. In Europe alone, cities are expected to accommodate 30 million more residents by 2050, significantly increasing pressure on available space [1]. A conventional response to this demand has been urban expansion, but this approach often leads to land consumption and urban sprawl, which in turn contributes to environmental degradation, including loss of natural habitats, increased air and water pollution [2], and a heightened risk of flooding [3].

One promising strategy is vertical expansion, which involves constructing additional stories on existing buildings rather than extending urban boundaries [4]. This method reduces the need for new infrastructure such as roads and utility networks, while also enabling simultaneous building renovations and energy retrofits [5]. As a result, vertical additions can enhance urban efficiency, reduce environmental impact, and revitalize aging building stocks. However, despite these benefits, the structural implications of adding new stories, particularly in seismically active regions, pose a significant challenge. The increased mass can amplify seismic forces and alter the dynamic response of the existing structure, often necessitating extensive and costly retrofitting to meet seismic regulations [6].

To mitigate these challenges, the use of lightweight construction materials is a key consideration. Timber, with its high strength-to-weight ratio, offers a particularly viable solution. For instance, LTF structures, compared to heavier materials such as steel and concrete, significantly reduce the additional gravitational load imposed on the original building. Moreover, timber construction aligns with sustainability goals, as it is a renewable material with a lower carbon footprint [7]. Additionally, its modularity and prefabrication allow for faster installation, minimizing both costs and disruptions to occupants.

Several studies have explored how lightweight rooftop structures influence seismic vulnerability, with findings suggesting that certain configurations can even improve overall seismic performance. In particular, previous research on light and limber moment frames [8] and cross-laminated timber (CLT) vertical additions [6] has indicated that these systems can potentially reduce seismic demand. However, the specific seismic effects of light-frame timber vertical additions remain less explored.

This study aims to address this gap by not only investigating the impact of LTF vertical additions on the seismic response of existing reinforced concrete (RC) buildings but also minimizing the impact through an optimized design. A simplified 3-degree-of-freedom (3-DOF) model is developed to predict how key design parameters of the LTF addition (i.e. the spacing of the sheathing-to-framing connections, and the number of LTF walls arranged in plan) influence the seismic behaviour of the existing building. As part of the optimization strategy, the two-story vertical addition is designed such that the second story remains in the elastic range, while the first story is allowed to yield and dissipate energy through hysteretic behaviour. To assess the accuracy and limitations of this simplified approach, nonlinear time history analyses are performed on both the 3-DOF model and refined finite element (FE) models of an archetype 1970s Italian RC residential building, before and after a two-story addition. The results provide insights into how optimized LTF additions can be designed to minimize seismic impact, offering a sustainable and structurally sound solution for vertical expansion in earthquake-prone regions.

2 ARCHETYPE CASE STUDY BUILDING

The study focuses on a three-story reinforced concrete (RC) frame building, representative of mid-1970s Italian residential structures. The archetype building was designed following typical construction practices of the period, considering only gravity loads, as many areas were not classified as seismic at the time due to the evolving seismic zoning in Italy [9]. Its characteristics such as structural layout, member dimensions, reinforcement details, and material properties, were derived through simulated design based on construction drawings, design handbooks [10], [11], and technical standards of the period [12]-[14], supplemented by relevant literature references [15]-[17].

2.1 Description of the archetype building

The building has a rectangular footprint of 23.0 m × 15.4 m, with an inter-story height of 3.0 m. The longitudinal frames (x-axis) feature beams with 30 cm × 50 cm cross-sections, while in the transverse direction (y-axis), wide-shallow beams measuring 20 cm × 80 cm are present. However, 30 cm × 50 cm beams are used in the staircase area to support the cantilevered steps. The column layout consists of 30 cm × 30 cm and 30 cm × 40 cm perimeter columns, the latter positioned along the shorter sides of the building, and 30 cm × 40 cm internal columns. The external infill walls are cavity walls composed of two 10 cm thick leaves of lightweight hollow clay bricks.

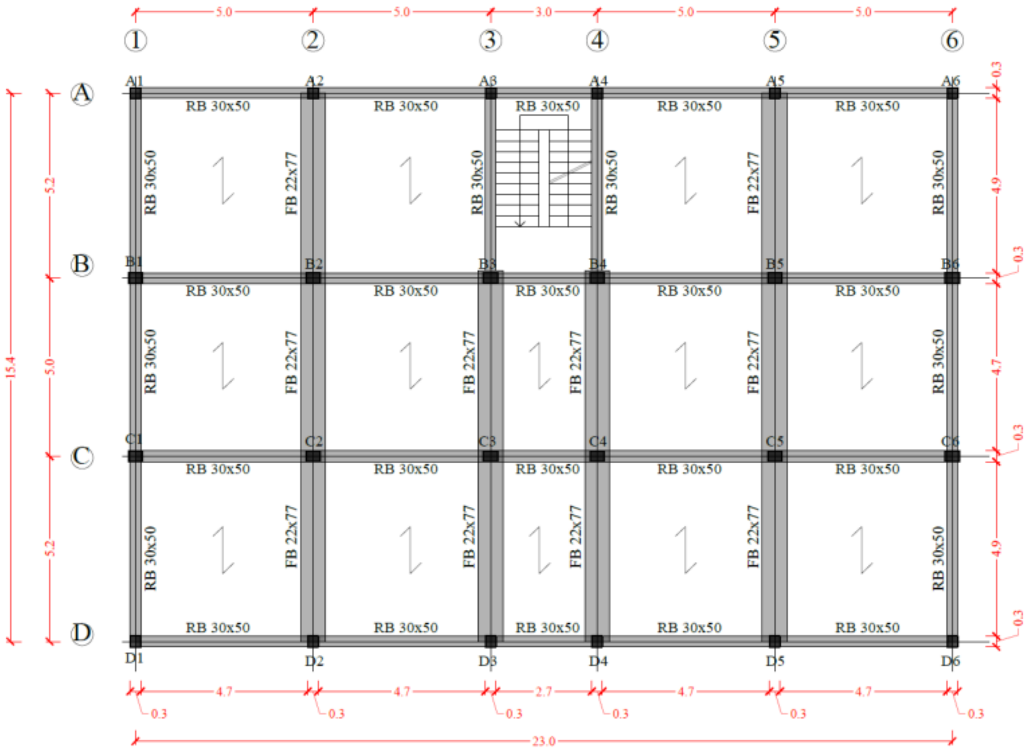


Figure 1: Structural floor plan of the archetype building.

The floor system is a *one-way beam*-and-*clay block* system, a common solution in Mediterranean construction [18]. It comprises 8 cm wide RC joists, spaced at 40 cm, with 16 cm hollow clay blocks, topped by a 4 cm thick RC slab, resulting in a total floor thickness of 20 cm. The floor contributes to a structural dead load of 2.30 kN/m² and a non-structural load of 3.20 kN/m², which accounts for flooring finishes and partition walls. The live load is assumed as 2.0 kN/m², in line with standard residential occupancy requirements. The seismic mass distribution, obtained from the combination of structural, non-structural and variable loads as per Italian seismic code prescriptions [19], is 408 tons for the ground story, 394 tons for the intermediate story and 288 tons for the roof.

The material properties assigned to the building are based on typical values from the literature [20], [21], ensuring they accurately reflect mid-20th-century Italian RC structures.

2.2 Finite element modelling

The finite element (FE) modelling was conducted using SAP2000 to simulate the nonlinear and cyclic behaviour of the case study building. The RC members were represented as frame elements, with fibre hinges at their ends to model concentrated plasticity and capture nonlinear behaviour. To enhance computational efficiency, the floor slabs were assumed to act as rigid diaphragms.

The masonry infill walls were modelled using an equivalent diagonal strut approach adopted in previous studies [22], with the influence of openings accounted for based on established methodologies from the literature [23]. To simulate the nonlinear and cyclic response of the infill walls, the diagonal strut elements were assigned a *multilinear plastic link* with a *pivot* hysteresis model, following recommendations from the literature [23].

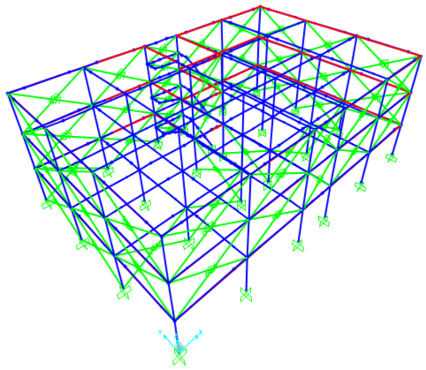


Figure 2: Finite element model of the archetype building.

3 DESIGN OPTIMIZATION OF THE TIMBER VERTICAL ADDITION

The design optimization of the LTF VA was conducted using an equivalent 3-DOF simplified model. This model was obtained by coupling three single-degree-of-freedom (SDOF) nonlinear models (Figure 3): one representing the existing RC building and two representing the two additional stories. In line with the optimization strategy, the second story of the VA was designed to remain elastic, while the first story was allowed to yield and dissipate energy through hysteresis.

The optimization process aimed at minimizing the seismic demand on the existing structure, quantified as the peak mean displacement (PMD) of the building's SDOF model, obtained from nonlinear time-history analyses. The displacement of the SDOF model of the building is defined as the representative displacement (RD) of the whole structure (see Section 3.1). The seismic demand was evaluated under seven spectrum-compatible ground motion records, with the peak mean displacement of the SDOF model computed as the average of the absolute peak values across the seven simulations. To ensure a comprehensive optimization, two separate 3-DOF models were analysed: one for the transversal direction (y-direction) and one for the longitudinal direction (x-direction), allowing for direction-specific tuning of the VA design. The equivalent modelling and analyses were performed using the software SAP2000 [24].

To calibrate the properties of the equivalent SDOF model representing the existing building, monotonic and cyclic pushover analyses were conducted on the refined FE model of the entire structure. Similarly, the mechanical properties (linear stiffness, nonlinear response, and cyclic behaviour) of the equivalent SDOF model representing each storey of the LTF VA were derived based on the refined FE model of a reference LTF wall with fixed geometry.

The refined FE model of the reference LTF wall was developed and validated against experimental results [25]. The wall connection system was designed so that shear brackets and hold-downs were over-resistant compared to the sheathing-to-framing nailing connections, ensuring that nonlinear deformations occurred exclusively in the nailing connections.

Using the validated FE model, the results were extended to different configurations of the sheathing-to-framing connection, allowing for the characterization of various LTF wall alternatives LTF with different nail spacings (*s*). This parameter was used in the optimization for the first story, while the minimum nail spacing was fixed for the second storey to ensure elastic behaviour.

The second design parameter is the number (*n*) of LTF walls arranged in the plan of the two-storeys vertical addition, which remains the same for both storeys.

Multiple structural layouts were generated for each configuration of the reference wall by varying the number of walls in each direction. Finally, the mechanical properties of the equivalent SDOF model for the vertical addition were determined by aggregating the contributions of the n timber walls in the considered direction.

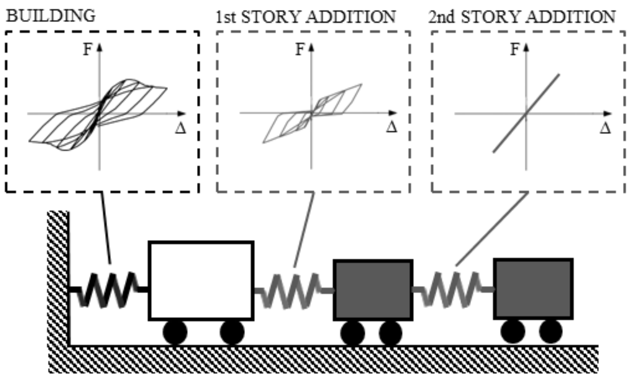


Figure 3: Schematization of the equivalent 3-DOF model.

3.1 Equivalent SDOF Modelling of the Archetype Building

The equivalent SDOF model of the archetype building was developed following the approach proposed by Oviedo et al. [26]. This methodology ensures that the SDOF system accurately represents the seismic response of the MDOF structure by deriving its properties from the first-mode response of the building.

As part of this approach, the representative displacement (RD) of the building was identified as the displacement at which the participation function $\beta_1\phi_1$ is equal to 1. Where:

- ϕ_1 represents the first mode shape-vector,
- β_1 is the modal participation factor of the first mode.

The equivalent SDOF model was implemented by connecting a joint to a fixed support through a *multilinear plastic link*. The participating mass corresponding to the first mode of the building in the considered direction was lumped to the joint ensuring consistency with the modal properties of the structure.

The nonlinear and cyclic behaviour of the *link* was characterized through a backbone curve and an associated hysteretic model. Thanks to its suitability to calibration the *pivot* hysteresis model was assumed to represent the cyclic response of the building. To define the backbone curve, and calibrate the *pivot* parameters, a monotonic and a cyclic pushover analysis were respectively performed on the refined FE model of the building, applying a first-mode proportional force distribution. The RD of the building was used as the monitored displacement in the pushover analysis, aligning with the considered methodology to establish a direct correlation between the refined model and its equivalent SDOF representation. Both force distribution and participating mass were assumed invariant throughout the damage process of the building, simplifying its dynamic behaviour under inelastic conditions.

The calibration process of the *multilinear plastic link* ensured that the cyclic behaviour and the cumulative energy dissipation of the equivalent SDOF system closely matched that of the refined FE model, allowing the simplified system to reproduce the nonlinear seismic response of the archetype building (see Figure 4).

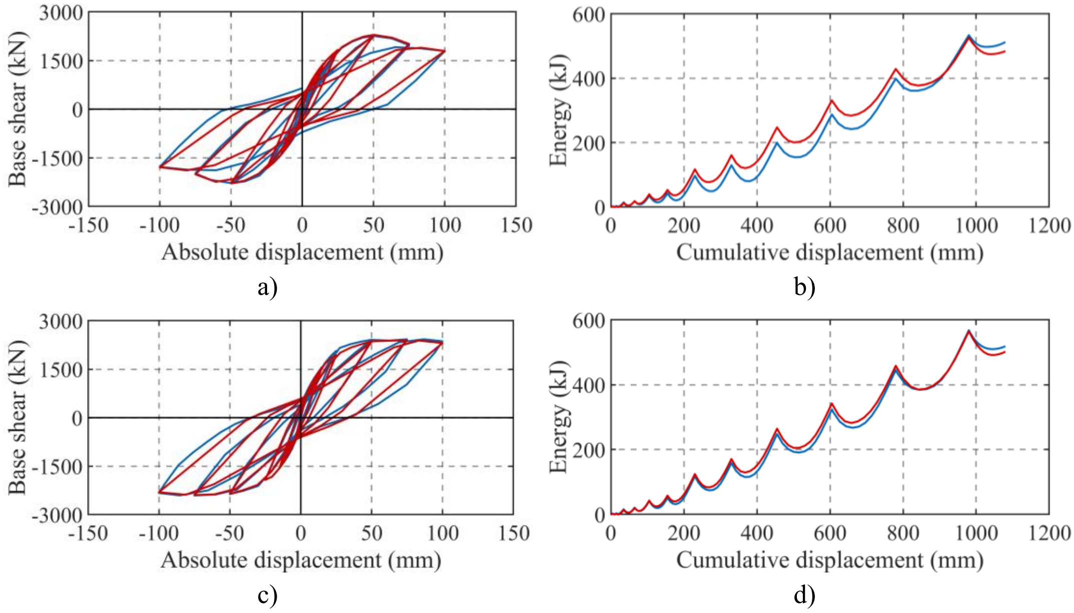


Figure 4: Calibration of the equivalent SDOF model of the building (—) against refined FE model (—) . a) Cyclic pushover and b) cumulative dissipated energy for the x-direction; c) cyclic pushover and d) cumulative dissipated energy for the y-direction.

3.2 Modelling of the Reference LTF wall

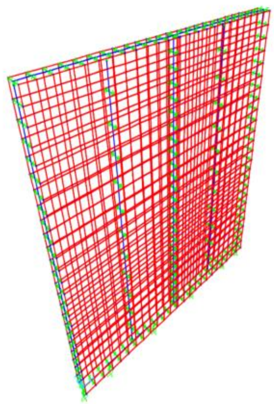


Figure 5: Finite element model of the reference LTF wall.

The reference LTF wall is characterized by the geometry illustrated in Figure 6. To capture its structural behaviour, a refined FE model was developed, incorporating distinct modelling strategies for each component: the timber framing was represented using *frame elements*, while the hold-downs and angle brackets were modelled as *linear elastic links*. *Linear elastic shell elements* were adopted to model the double OSB sheathing.

The sheathing-to-framing connection, which governs the nonlinear response of the wall, was simulated using *multilinear plastic links*. A pivot hysteresis model was assigned to these *links*, with parameters calibrated using data from the referenced experimental test.

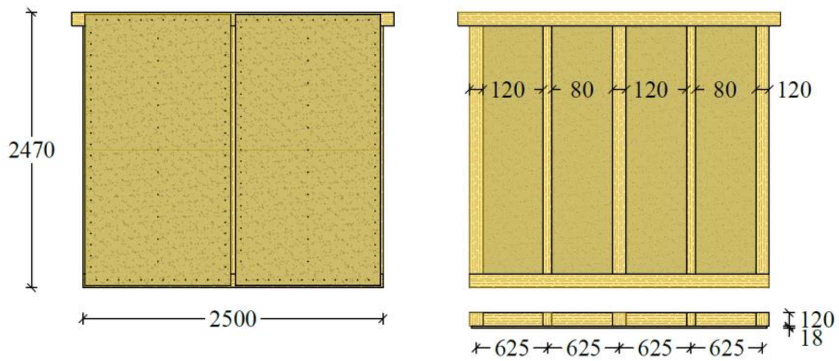


Figure 6: Reference LTF wall.

The accuracy of the developed model was verified through validation against the experimental data, ensuring that the numerical response closely matched the observed behaviour under cyclic lateral loading in terms of force-displacement curves and dissipated energy.

The experimentally validated model of the reference wall was modified by varying the nail spacing of the sheathing-to-framing connection. Seven different configurations were generated, with nail spacing ranging from 50 mm to 200 mm with 25 mm increments.

Monotonic pushover analyses were performed on the FE models of the seven reference LTF wall configurations, obtaining the bilinearised force-displacement curves reported in Figure 7.

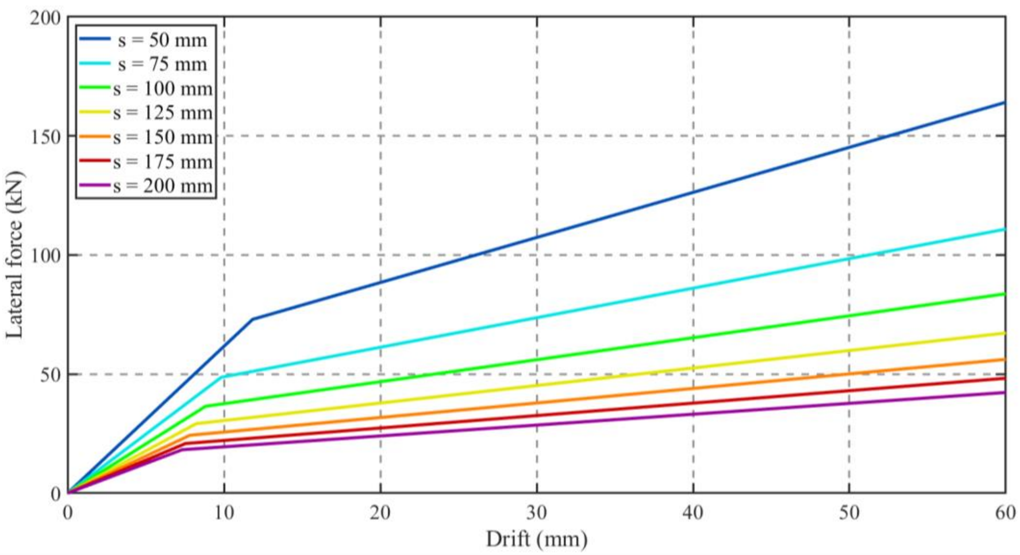


Figure 7: Bilinearised force-displacement curves of the reference LTF wall considering different nail spacings.

3.3 Parametric design and characterization of the LTF VA

The parametric design optimization of the two-storeys LTF VA was carried out independently for each of the two principal directions of the building, neglecting potential coupling effects between them. As mentioned earlier, the nail spacing and the number of walls arranged in each principal direction of the addition were selected as design parameters for the optimization.

Specifically, for the first storey, the seven nail fixing patterns introduced in Section 3.2 were considered, while the number of walls varied from 4 to 24 in the transverse direction of the building and from 4 to 30 in the longitudinal direction, increasing in steps of two walls per configuration. As a result, a total of 77 configurations (7 nail spacings × 11 wall layouts) were analysed in the transverse direction, and 98 configurations (7 nail spacings × 14 wall layouts) were analysed in the longitudinal direction.

For the second story of the vertical addition, the same number of walls as in the first story was maintained in both directions, while the nail spacing was uniformly set to the minimum value. This approach aimed to increase the stiffness of the second story, ensuring elastic behaviour and directing inelastic deformations, and thus hysteretic energy dissipation, exclusively to the first storey. To characterize the SDOF model representing the first story of the LTF VA as the design parameters vary, a rigid floor hypothesis was adopted. Since all walls are identical, they can be considered as springs acting in parallel. Under these assumptions, the backbone curve associated with the multilinear plastic link (describing the nonlinear response of the first storey of the LTF VA along the chosen direction) can be obtained by simply multiplying the ordinates of the displacement-force curves in Figure 7 by the number of walls arranged in the considered direction. Additionally, since the parameters governing the pivot hysteresis model are normalized to the yielding force, the slopes of the loading, unloading, and pinching branches remain consistent through scaling. Consequently, the pivot model parameters for the SDOF system were calibrated based on cyclic pushover analyses performed on the individual reference walls.

3.4 Seismic input and records selection

To characterize the seismic action on the building, an elastic design spectrum was adopted according to 14[19], representing a medium-high seismic zone of Italy. The spectrum was defined for a return period of 475 years, corresponding to the life-safety limit state. A ground type A was assumed, with a peak ground acceleration of 0.25g and a 5% damping ratio.

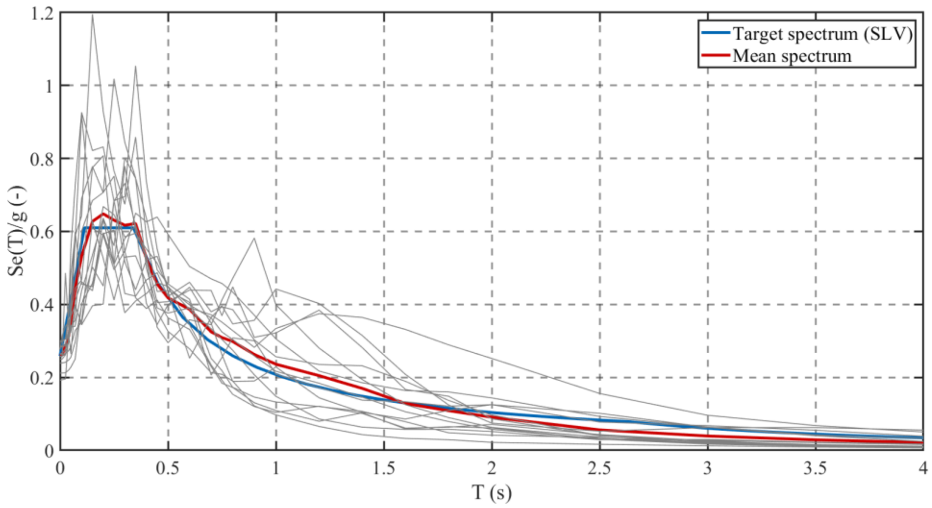


Figure 8: Comparison between the target elastic design spectrum for a return period of 475 years and the mean spectrum of the seven selected ground motion records.

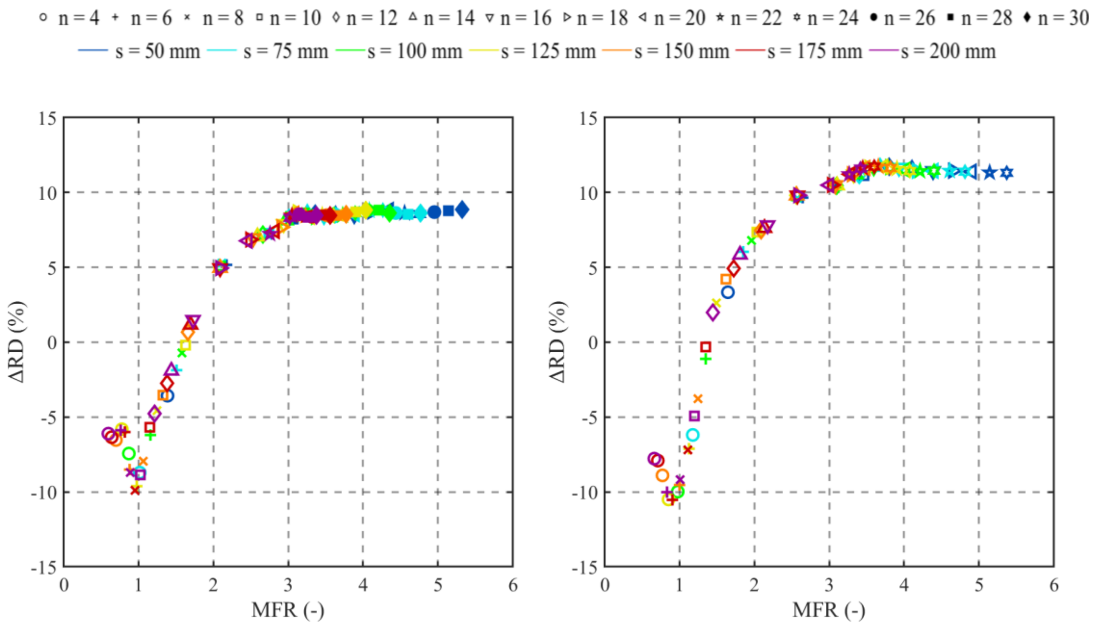
To ensure compliance with seismic code provisions [27], seven natural ground motion records, the minimum required for evaluating the structural response based on the mean values among the analyses, were selected to be spectrum-compatible with the target design spectrum [28]. Specifically, the mean spectral ordinates of the selected records were required to remain within -10% to +30% of the corresponding elastic spectrum values across the relevant range of vibration periods (See Figure 8).

3.5 Design optimization results

Figure 9 presents the optimization results, showing on the vertical axis the percentage variation of the mean reference displacement of the building (ΔRD) before and after the addition of the LTF VA. The mean reference displacement is calculated as the average of the peak displacements of the building's SDOF system for each record across the seven nonlinear time-history analyses of the selected ground motion records, serving as an indicator of the building's seismic damage.

On the horizontal axis, the modified frequency ratio (MFR) between the vertical addition and the building is reported. The MFR is calculated as the ratio of the modified fundamental frequency of the LTF VA and that of the building. Unlike classical frequency ratios based on elastic properties, the modified fundamental frequencies are determined using the secant stiffness of each system. The secant stiffnesses are computed based on the average of the maximum displacements experienced by each system during the seven nonlinear simulations (See Figure 10).

This approach represents an adaptation of the classical tuning procedure used for Tuned Mass Dampers (TMDs), extending it to systems exhibiting nonlinear behaviour. Notably, the results show that the optimal configuration (i.e., the one minimizing the mean representative displacement of the building) occurs when the modified frequency ratio approaches unity. This finding aligns with the fundamental principles of TMD theory, suggesting that the benefits of tuned dynamic interaction can still be leveraged even in nonlinear structural systems.



a) b)

Figure 9: Results of the design optimization of the LTF VA in a) x-direction, b) y-direction, expressed in terms of percentage variation in reference displacement (ΔRD).

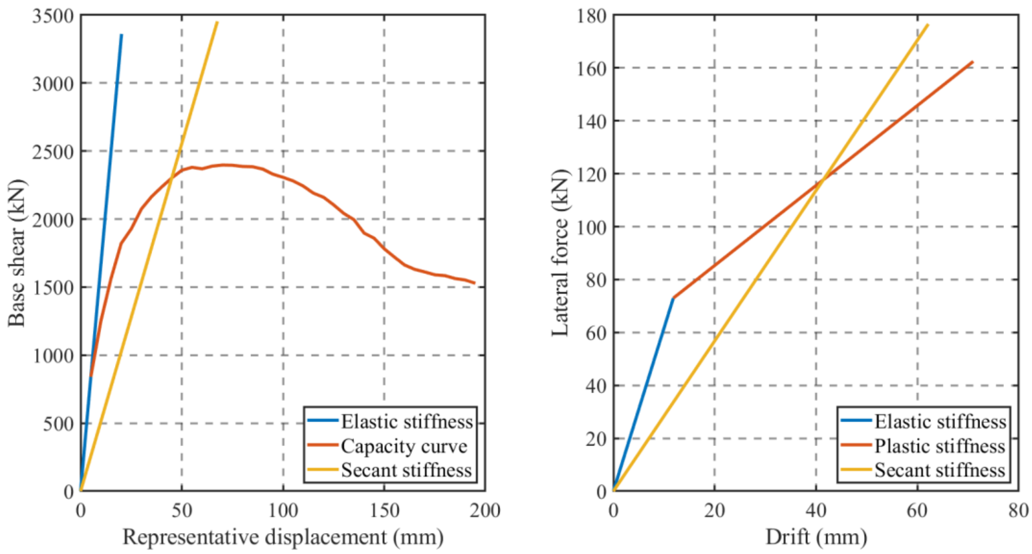


Figure 10: Definition of the secant stiffness used to determine the MFR.

4 COMPARATIVE TIME-TIME HYSTORY ANALYSIS THROUGH REFINED MODELLING

To validate the predictive capability of the simplified optimization approach, nonlinear time-history analyses were performed on the refined finite element (FE) model of the building, both before and after the addition. The analyses focused on the optimized configuration of the LTF VA and a representative non-optimized configuration, with the aim of evaluating how each influences the dynamic response of the structure using a refined modelling approach.

The same set of seven spectrum-compatible ground motion records used in the optimization process was employed. As mentioned, the analyses were performed on three configurations:

1) As built: the existing building without the addition.
2) Optimized Addition: the building with the vertical addition designed according to the optimal parameter combination identified in the previous chapter.
3) Non-Optimized Addition: The building with an addition designed without considering tuning.

For each case, the structural response was assessed using the maximum absolute interstorey drift ratios, calculated as the mean across the seven simulations. By comparing the results, the effectiveness of the optimization strategy in mitigating the seismic demand on the existing structure was assessed.

The results provide insight into the influence of the addition on the overall building response, demonstrating how an appropriately tuned design can reduce peak displacements and improve seismic performance.

4.1 Refined modelling of the vertical addition

The refined modelling approach for the vertical addition follows the same methodology used for individual walls, apart from the sheathing and sheathing-to-framing connections. To reduce the computational burden, these components were represented using an equivalent bracing system instead of being explicitly modelled. The equivalent bracing consists of two *multilinear plastic links* acting in tension only, with their backbone curves and *pivot* model parameters calibrated based on the results of the refined analyses of individual walls.

A diaphragm constraint was assigned to all nodes lying on each floor of the addition, ensuring rigid in-plane behaviour. Lateral loads were resisted exclusively by the timber walls, while vertical load transfer was entrusted to a pinned timber frame, which does not contribute to the horizontal stiffness of the structure.

4.2 Results of the comparative time-history analysis

Table 1 presents the results of the nonlinear time-history analyses, reporting the maximum inter-story drift ratio (IDR_{max}) averaged over the seven ground motion records, along with the corresponding variation (ΔIDR_{max}) after the addition with respect to the as-built configuration.

The results highlight that the seismic demand of the building is primarily governed by the first storey, which consistently exhibited the highest inter-story drifts across all simulations. Both the optimized and non-optimized configurations of LTF VA led to a reduction in seismic demand compared to the as-built condition. However, the optimized configuration achieved a more significant reduction in the critical first storey drift, confirming the effectiveness of the proposed optimization strategy in mitigating the building's seismic vulnerability.

Although an increase in the maximum inter-story drift ratio was observed at the third storey in both configurations with the addition, the drift levels remain within the range associated with slight damage, according to thresholds defined in [29]. Notably, the increase in drift at the third storey was nearly twice as large in the non-optimized configuration compared to the optimized one, further underscoring the improved seismic performance achieved through the proposed design strategy.

Table 1: Result of the comparative time-hystory analysis

		As-built	**Optimized LTF VA**		**Non-optimized LTF VA**	
	Storey	IDR_{max} (%)	IDR_{max} (%)	ΔIDR_{max} (%)	IDR_{max} (%)	ΔIDR_{max} (%)
Building	1st	0.69	0.58	-15.47	0.65	-5.27
	2nd	0.52	0.48	-8.53	0.61	+16.42
	3rd	0.23	0.27	+15.34	0.31	+33.43
LTF VA	1st	-	2.48	-	0.38	-
	2nd	-	0.27	-	0.27	-

Additionally, the analysis reveals that the second storey of the LTF addition remains elastic in both configurations, as intended in the design strategy to concentrate inelastic behaviour and energy dissipation in the first storey. While the first storey of the addition yields in the optimized configuration—allowing for hysteretic energy dissipation—it remains entirely elastic in the non-optimized case. Additionally, in the optimized configuration, the peak inter-story drift of the first storey of the addition remains below 2.5%, which is the drift limit for the Life Safety Limit State (LSLS) as prescribed by prominent standards [30],[31], and also below the drift capacity observed in the experimental tests used to calibrate the wall model. This confirms that although yielding occurs, no failure is expected. This behaviour suggests that the seismic demand mitigation observed in the optimized configuration is linked to the intended energy dissipation mechanism in the first storey, validating a key hypothesis of the optimization strategy.

5 CONCLUSIONS

This study proposes a simplified strategy for the seismic design and optimization of light timber frame (LTF) vertical additions (VAs) on existing low-code reinforced concrete (RC) buildings, aiming to minimize seismic demand while preserving the benefits of timber construction—lightweight, modularity, sustainability, and ease of prefabrication.

By using an equivalent 3-degree-of-freedom (3-DOF) model, calibrated through refined finite element (FE) analyses, the study explores how key design parameters, namely the sheathing-to-framing nail spacing and the number of timber walls, can be tuned to reduce the seismic response of the host structure.

The optimization strategy, which concentrates hysteretic energy dissipation in the first storey of the VA while keeping the second storey elastic, proved most effective when the dynamic properties of the addition were tuned to closely match those of the existing structure, resulting in a beneficial interaction similar to that observed in tuned mass damper systems.

Validation through nonlinear time-history analyses of refined models confirms the potential of the optimized configuration to reduce inter-story drift demands in critical storeys of the existing RC structure. A non-optimized configuration of the LFT VA was also analysed, and while it provided some reduction in drift compared to the as-built structure, it was consistently less effective than the optimized solution, particularly in limiting drift demands at the most critical storeys.

These results highlight how well-designed LTF vertical additions can not only avoid increasing seismic vulnerability but actively enhance the seismic response of existing buildings. While the proposed simplified design framework shows promise as a practical tool to support sustainable vertical densification policies in seismically active regions, its applicability should be confirmed through broader studies across diverse building typologies and seismic conditions.

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