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# FIRST INSIGTHS ON THE SEISMIC RESPONSE OF EXISTING BUILDINGS WITH TIMBER VERTICAL ADDITIONS

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**Abstract**

*Timber-based structural systems used for vertical additions on existing buildings represent a strategic lightweight and low-carbon solution into facing the issues related to urban densification, sustainability and reuse of buildings in the cities. Nowadays, examples of multi-storey timber vertical additions are available worldwide, but no practice-oriented design rules and structural models are well established in the technical community.*
*This paper presents a first contribution aimed at quantifying the main parameters affecting the structural response of existing buildings with Cross-Laminated Timber (CLT) vertical additions and guiding their seismic design (modal parameters, internal force distribution due to seismic loads, etc.). With reference to an explanatory case, represented by a two-story masonry building with in-plane deformable floors, the dynamics of the system depending on the number of vertical additions - from one to three - and the presence/absence of the mechanical connection systems between upper and lower structure has been explored.*
*The analysis delivered an estimation of the vibration periods, participating masses and modal shapes, so that a baseline for a comprehensive analysis of the structural response can be drawn. Comparisons among the results coming from the modal analysis allowed to make first considerations on the effect of CLT vertical additions on the overall behavior of the system aimed at defining seismic design rules for practice design-oriented applications, not so common in the national and internation context.*

**Keywords:** Vertical Additions, Cross-Laminated Timber, Masonry, Structural Modelling, Modal analysis, Seismic Design Rules.

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## 1 INTRODUCTION

Disasters triggered by natural events occurred in recent years, such as those related to climate change and earthquakes, highlighted the need of structural solutions compliant with the paradigms of sustainability and safety of buildings simultaneously. In other terms, there is a growing interest for structural solutions capable of mitigating the impact of the carbon dioxide introduction into the atmosphere over the entire life cycle of the building and of providing optimal structural solutions from structural performance standpoint.

In such a context, the potentialities associated with timber-based structural solutions in combining sustainability and safety of buildings is now recognized by academics, professionals and industrials [1-3]. Timber-based engineered products - such as laminated wood elements, solid panels in Cross-Laminated Timber, LVL, etc. - have had a huge growth in the recent years for practical applications, making the wood construction sector in strong progress [4]. Among the various applications, timber-based structural systems are having wide use as lightweight and low-carbon vertical additions (or extensions) on masonry, steel or reinforced concrete existing buildings (Figure 1). Vertical extensions to existing buildings allow the expansion of the volumes of existing constructions, to meet the raising demand for urban housing and space office and the reuse of buildings in urban areas without expanding onto rural lands resulting from population growth in highly urbanized areas; moreover, the adoption of prefabricated (timber) elements facilitates the construction phases [5-7].

The task of designing the vertical additions - not necessarily made with timber - is frequent in the Italian building stock. They have been (and are still today) designed without the support of seismic design-oriented rules, also due to the absence of national and international standards [8, 9]. In fact, the current Italian Building Code (IBC 2018) [8] (as well as the standard ASCE 7 [10]) do not provide design rules regarding vertical expansions, except for the sentence reported in Chapter 8 sect. 8.4.3 where it clarifies that in case of vertical extensions the building must be verified with respect to seismic actions calculated according to the current design earthquake provided by the code. This assertion obligates the designers to achieve seismic safety levels of the entire structure equal to those required for new buildings without any discount into minimizing the structural interventions of the substructure, although low additional inertial forces come from the vertical expansions.

On the other hand, no specific research dealing with the structural behavior of buildings with timber vertical additions can be found in the technical literature, except for a study regarding vertical expansions on r.c. buildings [11]. Some studies, instead, have been recently conducted on steel vertical extension on masonry buildings with intermediate isolation systems, where the buildings are modelled as non-conventional Tuned Mass Dampers (TMDs) playing the role of passive vibration control systems [12]. Such an approach is effective and can be adopted also to analyze the dynamic behavior of CLT vertical extensions on masonry buildings, where the effects of mass and stiffness ratios between upper and lower structures can be opportunely modified to control the dynamic behavior of the construction and define/optimize the seismic design approaches [13].

This paper reports first insights on the seismic structural response of existing masonry walls with mass timber vertical additional made with CLT panels. With reference to a sample two-story masonry building with deformable floors, the role of CLT vertical additions varying from one to three additional stories on the dynamics of the raised structure is assessed.

In particular, the outcomes of the resulting parametric modal analysis are illustrated and discussed with the aim of evaluating the overall and local structural response as well as the interaction between the original components and those realizing the vertical extension. Attention is focused on the vibration periods, the participating masses and the modal shapes, whose role in the definition of the seismic response of the structure is figured out in view of the development of specific design rules. More in detail, the parametric analysis involved: a) the mechanical connection systems between the superstructure and the substructure and b) the number of vertical additions on the dynamic response of the walls.

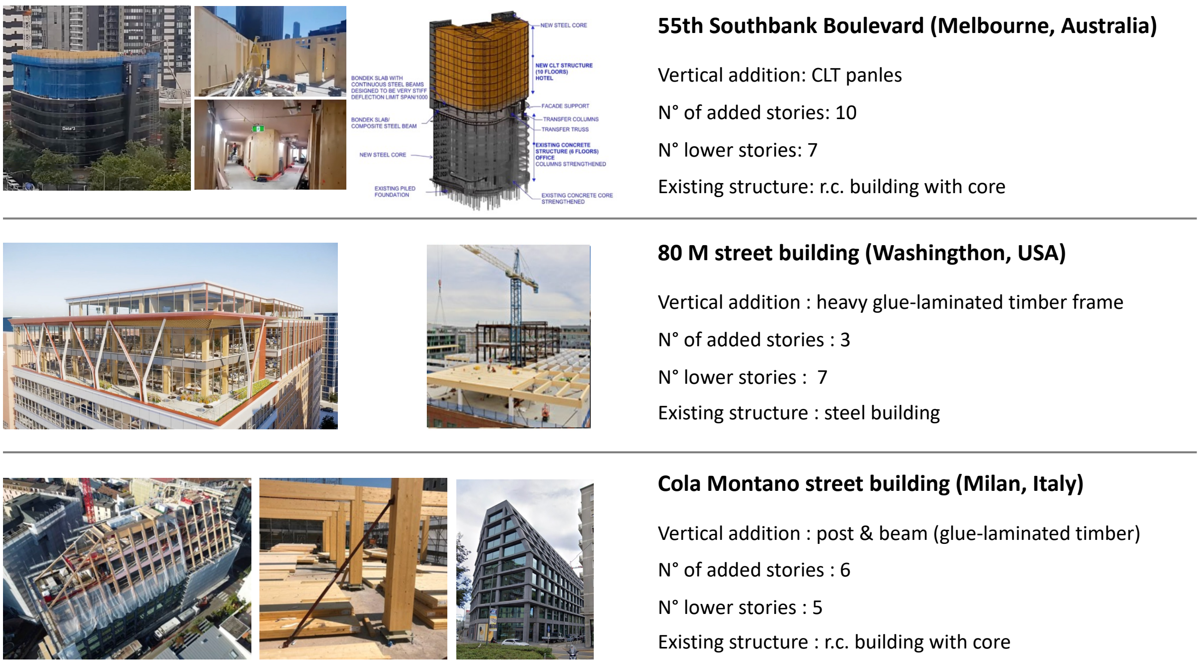


Figure 1: Real cases of timber vertical additions in urbanized areas

## 2 BASELINE OF THE STRUCTURAL PROBLEM

In the line of the objectives of this contribution, a quantitative assessment of the structural problem made of the design of the vertical additions on masonry buildings, a sample two-storey wall with regular openings has been considered. The wall, sketched in Figure 2, has been extracted from an existing three-dimensional masonry building having in-plane deformable floors. Thus, the single wall was analyzed separately from the others, loaded by masses evaluated by tributary area.

The structural elements (spandrels and piers) are made with regular masonry having constant thickness equal to 25 cm, whose main elastic properties are summarized in Figure 2. The selected masonry wall was assumed to be loaded by the gravity loads transferred by parallel beams wooden floors. The unit vertical loads considered for the wooden floors are: structural dead loads $G_{1k}$= 3.0 kN/m$^2$, non-structural dead loads $G_{2k}$= 1.5 kN/m$^2$, and live loads $Q_k$= 2.0 kN/m$^2$. The floor masses have been computed considering the corresponding tributary area (over a length of 1.95 m).

The vertical addition is made with load-bearing solid CLT panels, used for both the vertical walls and floors (Figure 2). The panels consist of 5-layer panels 4 cm thick (total thickness of the panel equal to 20 cm). The unit loads of the CLT floors are equal to: $G_{1k}$= 0.89 kN/m$^2$, $G_{2k}$= 1.5 kN/m$^2$ and $Q_k$= 2.0 kN/m$^2$. The elastic moduli of the CLT reported in Figure 2 were derived according to the European Technical Assessment [14].

The mechanical connections between the vertical wooden upper structure and the masonry substructure are made with hold-down (HD) and angle brackets (SA), typically used in platform-type construction technology. The HDs are devoted to withstanding the tensile forces induced by the overturning moment associated with earthquake and/or wind actions, while the SAs resists the shear force at the base of the timber panels (sliding) [15].

The HDs are positioned at the ends of each vertical panel, while the angle brackets in the middle. The resistance to compression forces, instead, is entrusted to the wood-to-reinforced concrete contact, having assumed the presence of reinforced concrete curb between the upper and lower structure. The mechanical characteristics of the hold-downs and the angle brackets were derived from the results of the experimental tests conducted by Fragiacomo et al., 2011 [16]. In particular, the hold-downs are WHT 540-type while the angle brackets BMF 90x116x48x3, both nailed to the wooden panels.

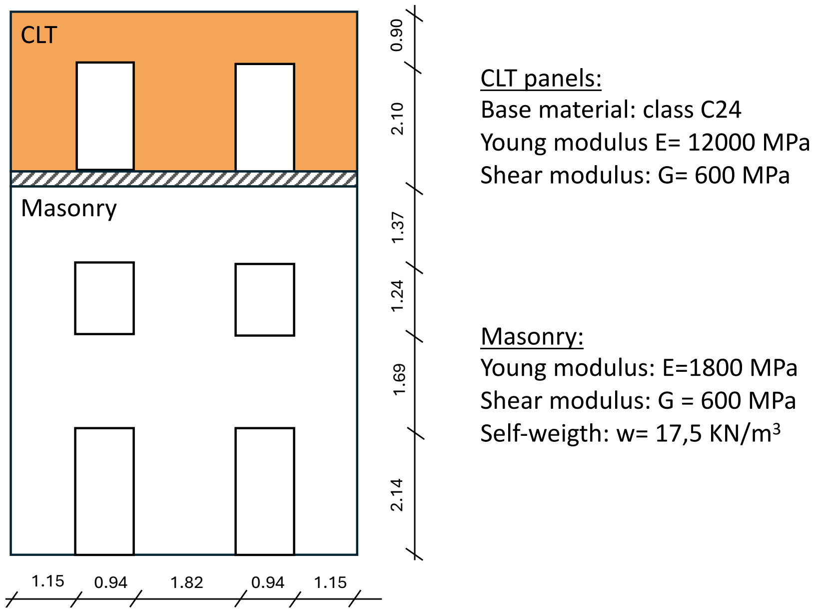


Figure 2: Benchmark case study with one vertical addition

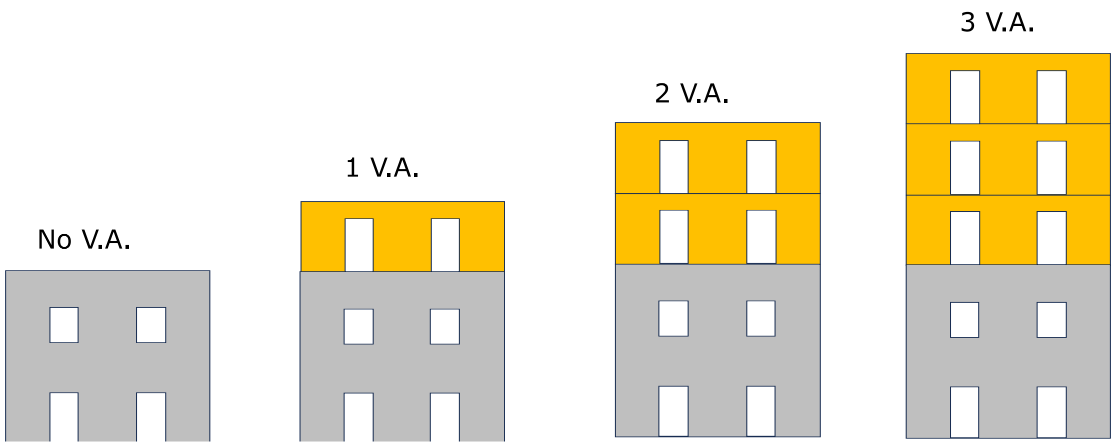


Figure 3: Multi-story vertical additions considered for the analyses

## 2.1 Structural modeling

Figure 3 reports the structural schemes considered for the numerical analyses. They have been carried out *a)* by varying the number of Vertical Additions (VA) from one to three storey and *b)* with and w/o mechanical connections between upper and lower structure (and between two consecutive additional storey); where "w/o connections" means that the vertical continuity of the material is ensured between two consecutive storey and no mechanical connections are included in the numerical model.

Both upper and lower structure was schematized with a two-dimensional elastic finite element model (2D shell) provided with flexural and shear stiffness, while the masses are concentrated at the floor level. The masonry was modeled as an isotropic material, while the CLT panels as an equivalent orthotropic material characterized by reduced normal and shear elastic moduli as a function of the number of effective layers in the two orthogonal directions [15].

To evaluate the influence of the connection systems on the overall behavior of structure with vertical extensions, the various cases represented in Figure 3 were modeled with and w/o mechanical connections between upper and lower structure, as schematically represented in Figure 4 for the case with single VA. As mentioned, the connection area - defined by the contact between the CLT and the r.c. curb (and between two consecutive timber storey) - was modeled considering the following constraints: 1) hold-down (HD), 2) angle brackets (SA) and 3) timber-to-r.c. contact (C).

Each constraint was schematized through *multi-linear elastic link* available in the SAP 2000 software library, which enables to define zero-length elastic elements. The elements of the connecting area (hold-downs, angle-brackets and timber-to-concrete contact) was modelled with linear elastic behavior in tension and in compression. This schematization induces unavoidable (elastic) up-lifting in the panel-to-concrete contact area with the consequent partialization of the contact area between the vertical CLT panel and r.c. curb; this means that the dynamics of the system is oriented to evaluations valid for the Ultimate Limit State evaluations.

From a structural standpoint, such a type of structure is conceived as a non-conventional TMD structure because of the presence of intermediate mechanical connections between the upper and lower structure, and at intermediate story of the CLT additions in case of multi-storey vertical expansions.

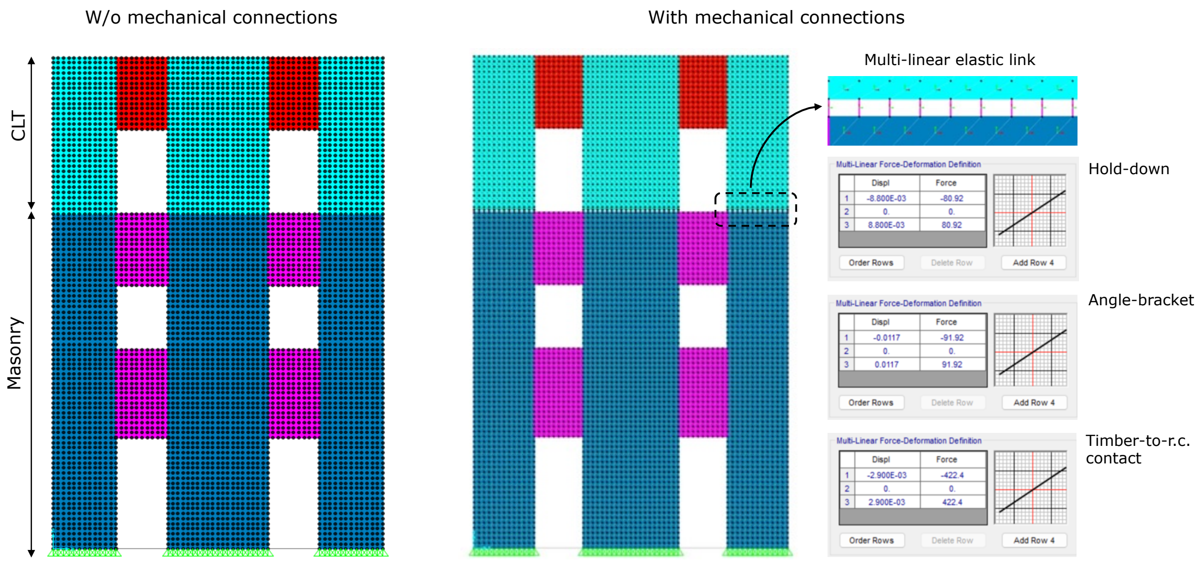


Figure 4: 2-D shell finite element model

## 3 DYNAMICS OF THE RAISED BUILDING SUBSTRUCTURE

The present Section collects some remarks on the dynamics of the system in the original and raised configuration. Attention is paid to the results of the numerical modal analysis aimed at understanding the role of the main parameters influencing the basic dynamic response of masonry substructures with CLT vertical additions and draw early directions for defining optimal design rules.

### 3.1 Vibration periods

Figure 5 shows the comparison between the values of the first vibration periods ($T_1$) obtained through the modal analysis for the cases “2D-shell with connections” and “2D-shell w/o connections”, with one, two and three vertical additions. For comparison purpose, in the same histogram the first period of the structure without extensions is also shown. Instead, Table 1 shows the computation of the percentage increment of period evaluated for the cases: a) period elongation due to the presence of connections ($\Delta T_{1,conn}$), b) period elongation due to the vertical addition alone (with connections) ($\Delta T_{1,VA}$).

The comparison between the first set of vibration periods highlights some interesting findings. Firstly, the mechanical connections play a fundamental role in the seismic response analysis: they cannot be neglected since their deformability induces a significant increment in the fundamental vibration period, especially as the number of additional storey increases (Table 1). On the other hand, the comparison between the vibration periods associated with the model “2D-shell w/o connections” and that of the masonry structure only (No VA) also remarks the effect of the upper structure on the period variation as the number of additional storey increases (Table 1), independently upon the presence of the connections.

Nevertheless, the period variations coming from these analyses were expected since the presence of the vertical additions induce irregularity in elevation, both in terms of masses and lateral stiffnesses variations. To this end, Table 1 also reports the values of the overall mass M expressed in tons (including the vertical additions) and the mass ratio $\alpha$ between the upper and lower structure, as the number of additional storey varies. Note that the presence of a single additional story involves approximately a mass ratio $\alpha$ = 23% and this, in the light of the current national and international standards [8, 9], certainly represents a symptom of irregularity in elevation. The Italian standard IBC18 [8] sets at 10% the maximum global mass variation for existing structures, clarifying that in the case of new buildings the variations in mass between two consecutive stories must not exceed the 25%. To this must be added the reduced stiffness of the vertical addition compared to that of the masonry substructure because of the high deformability of the mechanical connections.

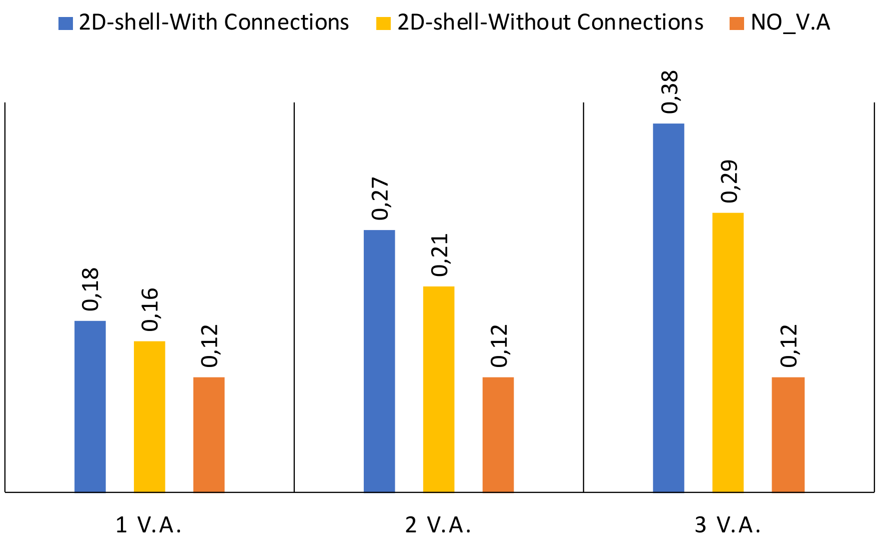


Figure 5: First vibration periods: a) comparisons between the different analyzed cases

| Case | $\Delta T_{1,conn}$ (%) | $\Delta T_{1,VA}$ (%) | M (t) | α (%) |
|---|---|---|---|---|
| No V.A. | - | - | 23,42 | - |
| 1 V.A. | 12,0 | 33,3 | 28,82 | 23 |
| 2 V.A. | 28,5 | 75,0 | 33,82 | 44 |
| 3 V.A. | 31,0 | 141,6 | 38,82 | 66 |
| $\Delta T_{1,coll}$ = increment of period due to mechanical connections. $\Delta T_{1,VA}$ = increment of period due to vertical addition (with connections). | | | | |

Table 1: Periods variations, masses and mass ratios

### 3.2 Participating masses and modal shapes

The histogram of Figure 6 compares the values of the participating masses ($M^*$) associated with the first three vibration modes derived from the modal analysis. The graph is relative to the cases spanning from one to three vertical extensions, modelled in the presence and absence of connection systems.

The histogram shows that either the deformability induced by the mechanical connections and the number of storey affect (significantly) the excited masses associated with the different vibration modes. In fact, the participating mass of the first vibration mode of the walls modelled in the presence of connections is significantly reduced compared to the cases of walls w/o connections. Similarly, the increase of the number of vertical extensions leads to a reduction of the participating mass of the first mode, intensifying the contribution of the participating masses associated with the higher modes. The same graph also plots the value of the participating mass relative to the first vibration mode of the substructure only (i.e., w/o additions), resulted equal to 95% of the total mass. The discrepancy between the participating masses of the substructure compared to the case of the wall vertical additions quantifies the influence of the higher modes on the overall dynamic behavior of the structure.

Figure 7, instead, reports a comparison between the modal shapes of the first vibration mode for the case with one, two and three vertical additions modeled with and w/o the mechanical connections. The mechanical connections influence the modal shapes of the structure: they move away from that close to a pseudo-triangular trend typical of the first vibration mode of regular structures.

These basic aspects emerged from the modal analysis - which deserve further investigations - are fundamental to reflect about the definition of the seismic analysis criteria for structures interested by vertical extensions. In fact, taking into consideration the case of the wall without extensions, the participating mass of the first mode exceeds the threshold value of 75% provided for by the codes for masonry structures [8, 9], and then the equivalent seismic actions can be calculated according to the linear static analysis. On the contrary, the presence of the vertical additions reduces the participating mass associated with the first mode depending on the number of additional storey (i.e., mass ratios) and mechanical connections (i.e., stiffness ratios). Consequently, a complete modal analysis, accounting for the influence of the higher vibration modes, appears more suitable in calculating the seismic action distribution along the wall's height [8].

It is evident that in the case of vertical additions causing irregularities in elevation to the system, the basic principles of the simplified linear static approach, typically adopted for regular structures, should be verified through a modal analysis. Such aspects are better treated in the following Section.

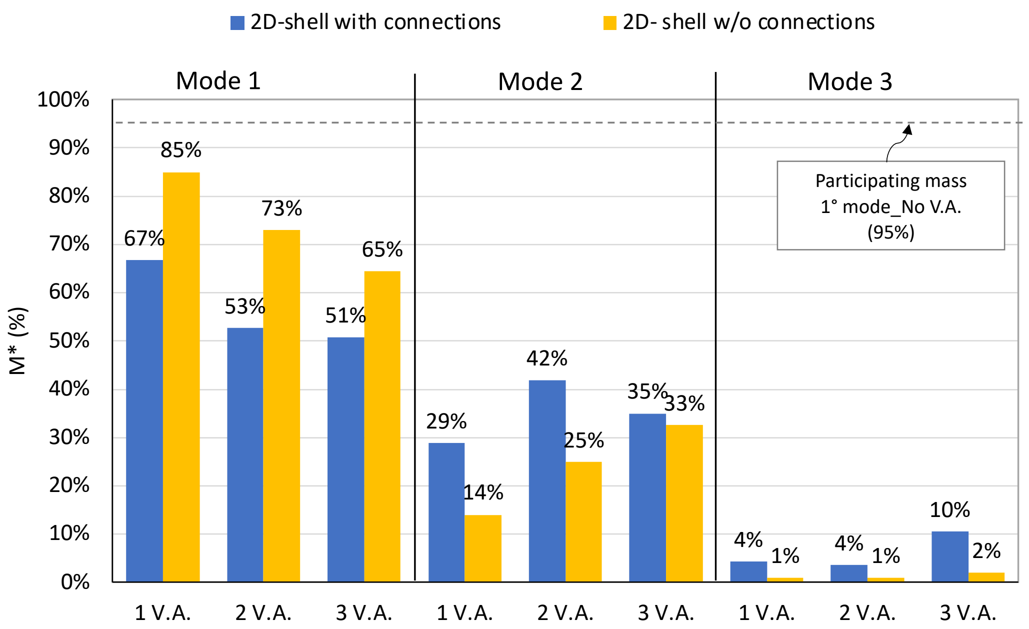


Figure 6: Participating masses

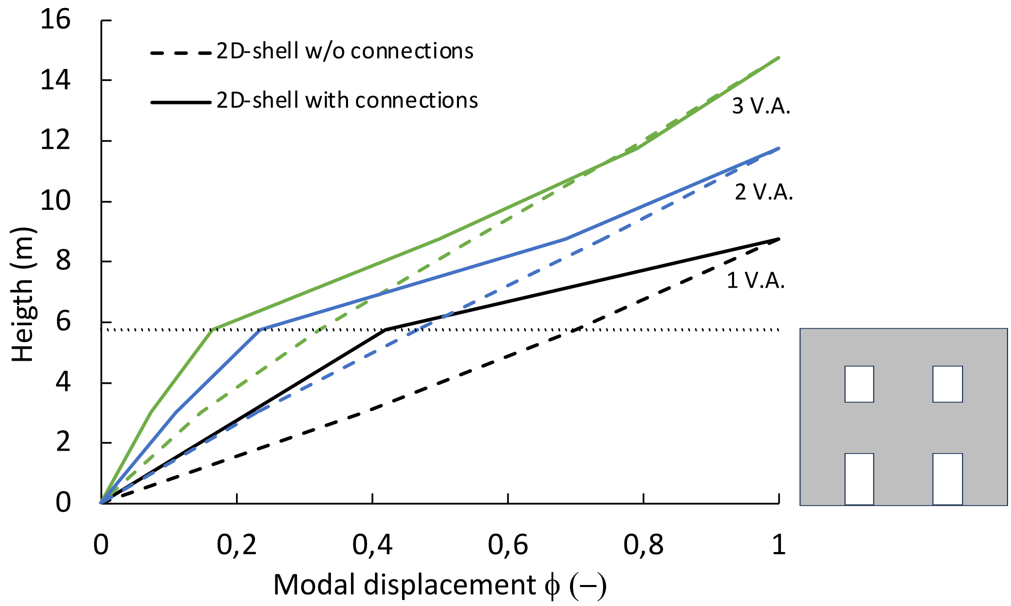


Figure 7: First modal shapes

### 3.3 Distribution of the inertia forces along the wall height

The seismic forces distribution along the wall's heigh depend upon both mass and stiffness ratios between upper and lower structure [12, 17]. Among the other, their quantification is of paramount importance for calculating the seismic demand (i.e., base shear) in seismic verifications or the lateral force profile to adopt in non-linear static analysis [8, 9].

In this Section the seismic force distributions along the wall's height have been calculated by considering the modal superimposition method, considering the higher vibration modes. Such a calculation was performed for the case of the masonry wall with a single and double CLT vertical additions, including the mechanical connections in the 2D finite element numerical model.

Figure 8 reports the force profile along the wall's height, where the force at *i*-th story associated with the different modal shapes is calculated as follows:

$$F_{ij} = m_i \, \Phi_{ij} \, \Gamma_j \, S_a(T_j) \quad (1)$$

where the subscript *i* and *j* represent the *i*-th floor and t *j*-th vibration mode, *m* is the mass at each floor, Φ the modal displacement component, Γ the modal participating factor of the *j*-th mode and $S_a$ the spectral acceleration at period $T_j$. As far as the spectral acceleration is concerned, it has been derived from a response spectrum for the city of Campobasso on soil-type B (Molise Region, Italy), whose peak-ground acceleration is $a_g$= 0,27*g* at Ultimate Limit State.

Figure 8 points out the different contributions provided by the three vibration modes to the computation of the seismic force distribution along the wall's height. The contribution of the second vibration mode resulted not negligible with respect to the first mode, especially at the interconnection level between upper and lower structure (see "VA level" in Figure 8), while the third mode contributes in a reduced part.

In order to quantify the contribution of the different vibration modes, the Figures 9*a* and 9*b* compares the force profiles calculated with a code compliant response spectra analysis with those given by the first vibration mode only, for the cases of single and double CLT extensions respectively. The modal contributions have been combined through the SRSS combination - i.e. only the vibration modes exceeding the threshold of 5% of the participating mass are accounted for - suitable in the current case being the vibration periods distinct among them (i.e., differ more than 10%).

As usual for irregular structures, Figure 9 highlights as the contribution of the vibration modes should be taken in due consideration in calculating the seismic force distribution, especially for increasing number of vertical additions where the first mode is not representative of the seismic force profile, even in the case of significant participating mass, in the order of the one recommended by IBC18 for masonry structures. In these cases, the higher modes play a key role in the distribution of the seismic demand an on the internal forces exchanged between the upper and the lower structure. They have also an effect on the optimal demand-to-capacity ratio deployment in the structures, as well as into selecting representative lateral force profiles for implementing push-over analyses.

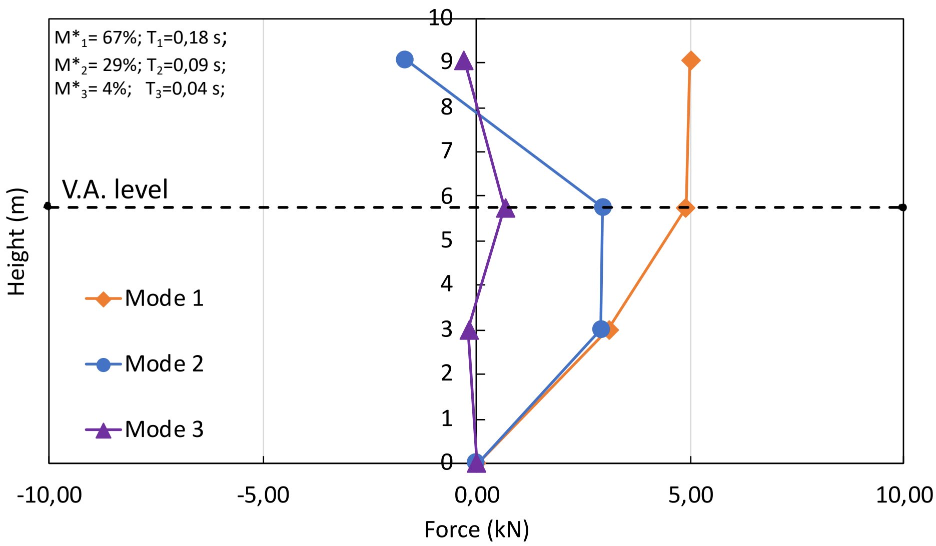


Figure 8: Equivalent lateral force distribution associated with the different vibration modes.

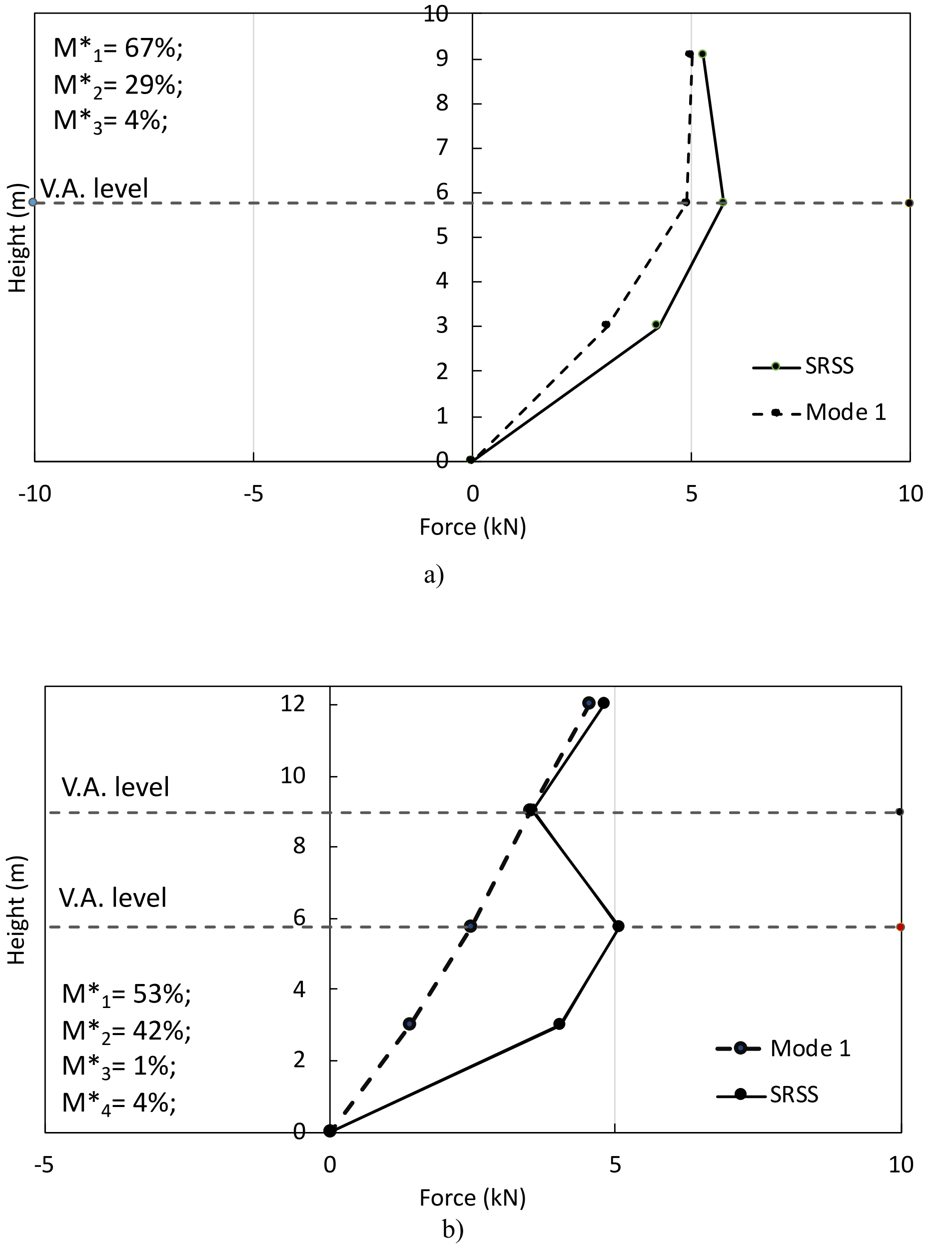


Figure 9: Comparison between the seismic force profiles SRSS *vs* first mode: a) case 1VA, b) case 2VA

## 4 CONCLUDING REMARKS

This paper reports some remarks on the dynamic behavior of masonry buildings with multi-story CLT vertical additions aimed at defining practice-oriented seismic design rules. Although the potentialities of timber vertical additions are clear, specific seismic design rules are lacking and not yet well established.

This contribution aimed at defining an early quantitative baseline of the structural problem, a wall extracted from a simple symmetrical masonry building with deformable floors, by means of an assessment of the dynamics of a representative substructure. A number of variables has been accounted for, namely the stiffness of connection systems and the number of the vertical additions.

Preliminary outcomes of the parametric analyses have pointed out the role of the stiffness of the connections in combination with the ratios between story masses and stiffness of upper and lower structures into determining equivalent seismic actions.
The results, even limited, confirm the need of further analyses for the definition of optimal design recommendations for a really sustainable and valuable structural type, intrinsically irregular in elevation.

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