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AN INSIGHT ON THE ROLE OF THE DESIGN PROCEDURES ON DAMAGE TOLERANCE AND COST OF CLT BUILDINGS IN SEISMIC AREAS

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Abstract

Physical characteristics of timber as a construction material offer a high-performance option for the design and erection of buildings in seismic-prone area. Massive Cross-Laminated Timber (CLT) buildings represent a vehicle to increase the building sustainability and a robust workaround to heavyweight constructions.

This paper is intended as a contribution in the reviewing the most common design procedures for the seismic design of CLT buildings and their implications on structural features and technological solutions, focusing the attention particularly on overall seismic performance, damage tolerance, construction costs and environmental impact. As a starting point of a more comprehensive study, the assessment is made with reference to a real building, which is representative of a wide class of buildings recently designed and erected in many Italian regions exposed to low and moderate seismic hazard. As a common reference, the analysis has been carried out according to a two-dimensional model of the panels, assumed to be elastic, varying the type of connections at the base, the presence of pre-stressing steel bars for rocking control and of dissipative devices. The main outcomes of the study can be summarized as follows: (i) the structural seismic behavior of CLT buildings is significantly influenced by the structural schemes adopted for walls and connections; (ii) construction costs and environmental impact decrease when enhanced design procedures are used.

Keywords: CLT, Damage tolerance, Timber Buildings, Connections, Construction Costs, Sustainability in construction.

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1 INTRODUCTION

Low and mid-rise Cross-Laminated Timber (CLT) buildings represent a valid alternative to traditional masonry or reinforced concrete buildings, both for residential and non-residential use [1, 2, 3]. In principle, the beneficial properties of the base material related to reduced weight, thermal insulation and its natural origin place timber buildings at very high levels of structural and seismic, energetic, and sustainability performances [4].

The rapid progression of these constructions on the market has been mainly boosted by the technology and production advances of the timber components and subassemblies; nevertheless, at International (European) level this process has not been complemented by a rational development and updating of the design, mainly seismic, codes. This circumstance is confirmed by the current form of the Eurocode 5 [5] and Eurocode 8 [6], which currently partially cover seismic design and are currently in a revision and enhancement phase [7, 8]. At a National (Italy), the Technical Document CNR DT-206/R1 2018 [9] can be addressed as one of the early technical documents defining a more comprehensive framework of seismic design rules for timber buildings coherent with the capacity design method [10]. Experimental and numerical studies proved that, for *traditional* CLT buildings, a rational design of the connections at panel-to-panel and panel-to-foundation interface is crucial to achieve satisfactory seismic response [11]. Herein, ‘traditional’ CLT buildings are those having connections made with hold-downs (HDs), bearing tensile forces due to moment demand, and angle-brackets (ABs) aimed at covering the shear demand. Those connections are designed according to Johansen’s failure mechanisms under seismic-like actions, i.e. localized timber embedding combined with yielding of metal fasteners [12]. This circumstance points out the activation of permanent damage mechanisms allocated in the timber panels due to embedding of wood, as documented in Fig. 1, where typical failure mechanisms of HDs and ABs observed during experimental testing are shown. Due to timber damage, vertical timber panels result heavily damaged making their reparation impractical after an earthquake, so that the demolition of the building (or some its portions) results the most rational option entailing additional costs due to both demolition and reconstruction, recycle of materials and an environmental impact.

To overcome the problem of damaging of CLT walls, alternative connection systems have been proposed in literature for CLT buildings, the so-called *low damage* CLT buildings. A novel class of mechanical connections recalled the attention of the technical community [11, 13, 14]. They are conceived to concentrate the seismic damage in steel plates (or bars) only, protecting the vertical timber panel against permanent damage. Indeed, the concept of ‘low-damaging’ or better ‘damage tolerance’ for timber systems was firstly claimed in New Zealand referring to post-tensioned rocking timber walls [15]; they consist of self-centering rocking walls provided or not with additional plug and play dissipating systems at the base of the wall made with mild-steel buckling-restrained bars [16].

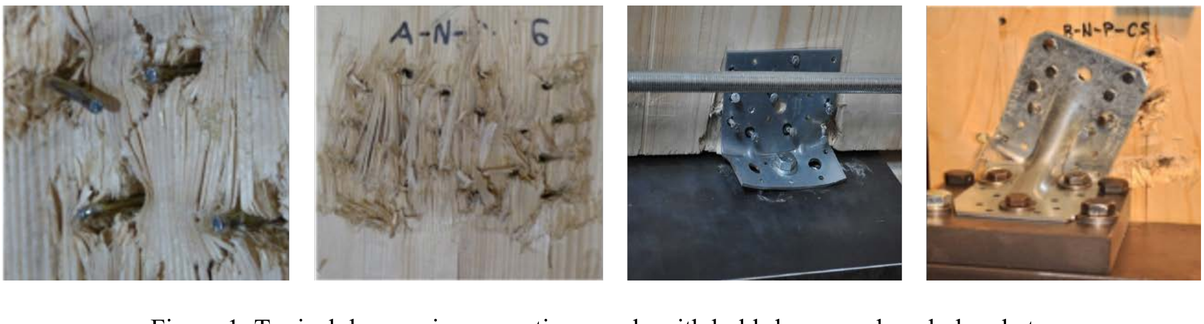


Figure 1: Typical damage in connections made with hold-downs and angle-brackets.

To obtain high-performant CLT buildings, a rational design approach devoted to optimizing and integrating the global performance of CLT buildings - in terms of seismic, economic and Life Cycle Assessment (LCA) - is needed (Fig 2). In this framework, structural solutions oriented towards damage control in structural components, and then costs reduction related to demolition of structural and non-structural components and improvement of the aspects influencing the LCA should be selected in engineering practice [1].

This paper presents an insight on the relationship between design criteria of the connecting systems and global performances of CLT buildings in seismic area by means of a comparative analysis of traditional and damage tolerant CLT buildings. This task is accomplished by means of the analysis of a real two-storey CLT school building, located in Northern of Italy, and its seismic design made varying the geometry of the walls and typology of connections. A comparative analysis of the different structural layouts has been then performed depending on the results of nonlinear static analyses (e.g., pushover).

So doing, some reflections on the relevance of optimized and integrated design procedures for CLT buildings are proposed, highlighting the major role of the connecting system design on the overall structural response of the buildings in terms of seismic capacity, ductility, damage control and tolerance, interaction with architectural and non-structural features, overall construction costs, and impact on the Lifecycle performance of the buildings.

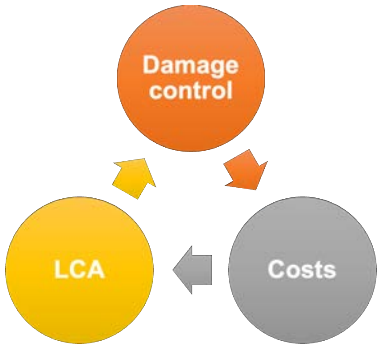


Figure 2: Optimized design approach for timber buildings.

2 ON THE DESIGN THE CONNECTIONS

In the context of the capacity design methodology, the design of the connections is severely ruled by the Strength Hierarchy (SH) criterion established among the components present in panel-to-panel and panel-to-foundation contact zone [8].

To date, National and International codes [5, 6] and guidelines [9] refer to a single type of dissipative mechanisms – those required for the traditional CLT buildings - based on the Johansen’s theory. For such an approach (identified as Approach A1 in the following) the non-dissipative elements are the HDs and ABs, while the dissipative ones are the timber-to-steel interactions characterized by a combination of timber embedding and cyclic yielding of steel metal fasteners (Table 1). Due to the cyclic nature and intensity of earthquakes, a high and permanent damage of timber panels in the connecting zones is accepted in the absence of detailed evaluations of the damage level of the timber panels. This may be the case of buildings located in high seismicity regions, where high levels of ground shaking may appear even at the Damage Limit States and high demand at the connection level may occur to activate the overall ductility required to survive at the Life Safety and Collapse limit states [17]. As a consequence, a relevant impact on the costs related to demolition and/or reconstruction of structural and non-structural component, recycle of materials, and sustainability of the construction seems to be intrinsically accepted.

As above discussed, alternative damage tolerant connections can represent an effective solution to reduce costs related to damage of structural components after an earthquake, although HS rules are not still included in the current standards. The seismic dissipation is concentrated into sacrificial 'plug and play' metallic elements which yields in tension or shear (bars, steel plates, etc.), while timber elements are non-dissipative and behaves elastically. As advantage, timber components are preserved to plastic damage after the seismic event and no costs related to demolition and/or reconstruction, recycle arise. This approach has been named as Approach A2 in Table 1.

Furthermore, problems related to damage tolerance of non-structural elements, forced to accommodate the inter-story drift of the seismic resistant-structure, can impact significantly on repair/reconstruction costs and on safety of peoples in the case of low-and high-intensity earthquakes [18]. In the following, some considerations on this issue are addressed.

Type	**Non dissipative element**	**Dissipative element**	**SH formulation**
Approach A1	Steel plates (HD/AB)	Timber-to-steel interaction	$N_{pl} \geq R_d \gamma_{Rd}$
Approach A2	Timber-to-steel interaction	Steel elements (bars, plates, etc.)	$R_d \geq N_{pl} \gamma_{Rd}$

N_{pl} = plastic axial strength of the non-dissipative element - R_d = maximum strength of the dissipative element - γ_{Rd}= over-strength factor of the connection

Table 1: Description of the Strength Hierarchy criteria.

3 METHODOLOGY

3.1 Description of the mockup building

The mockup building is a two-story CLT school building located in Northern of Italy (Fig. 3), whose structural project was available to the authors. The building belongs to the class of use III (as defined by IBC 2018 [19]). The real building is designed to be built in a low-intensity seismic zone, where a peak-ground acceleration on bed-rock is equal to a_g=0.081g at Life Safety (LS) limit state.

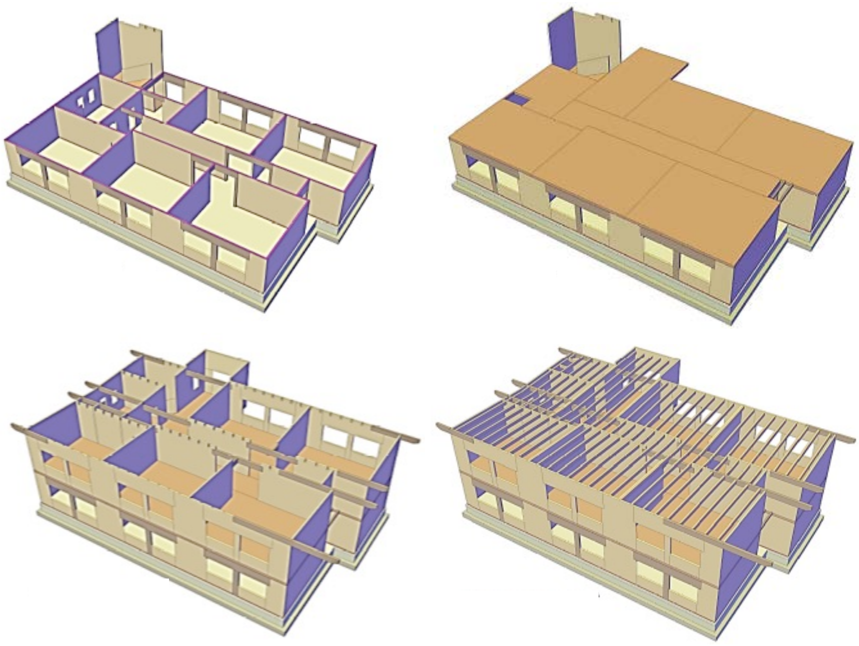


Figure 3: Views of the mockup CLT building.

The structure is made of 140 mm thick five-ply seismic-resistant CLT walls, where the thick of the two vertical external layers is equal to 40 mm and those internal (both in vertical and horizontal directions) are 20 mm thick. The strength class of CLT corresponds to solid wood class C24 derived according to ETA-14/0349 [20].

The construction technology follows the platform-type system, where vertical walls are interrupted at inter-story by horizontal CLT floors interposed between two consecutive vertical panels. The floors are composed by seven-layered CLT panels with strength class C24.

It is worth noting that the comparative analysis has no relation with the real design and erection of the building; indeed, available technical material has been used to refer to realistic building geometry, material properties, and live and dead loads. The arrangement of the walls herein addressed is the same provided by the real project and shown in Fig. 3, while different solutions have been adopted for the connections.

3.2 Test cases

To investigate the role of the design procedures on the seismic performances, damage and costs of CLT buildings, different structural schemes have been considered to model the CLT walls. Fig. 4 shows the four structural schemes adopted, divided in two main Groups: *a*) Group 1 (G1) includes Monolithic (M), Coupled (C), and Uncoupled (U) CLT walls, designed considering both Approach A1 and A2 listed in Table 1; *b*) Group 2 (G2) instead includes post-tensioned CLT walls only, characterized by a pure rocking behaviour only (i.e., without additional dissipaters at the base).

Coupling between the walls is assumed to be made of horizontal metal fasteners uniformly distributed along the vertical joints designed to behave elastically. Whereas the geometries of the walls, number of layers of CLT panels and number of the angle-brackets are the same in each scheme. Note that the vertical timber panels are not in direct contact with the reinforced concrete foundation, but it is interposed a horizontal timber beam between the vertical panel and the foundation involving orthogonal-to-grain timber to timber contact at the interface.

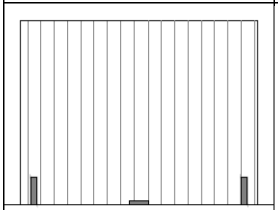
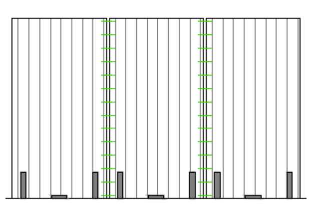
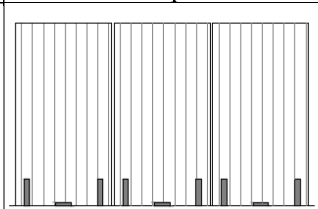
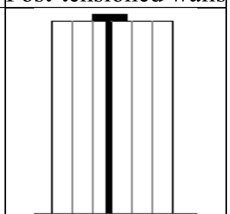
Group 1			**Group 2**
G1-M	G1-C	G1-U	G2
CLT monolithic walls	CLT coupled walls	CLT uncoupled walls	Post-tensioned walls
			
HD at the edges only	HD distributed and vertical joints	HD distributed and no vertical joints	Rocking walls

Figure 4: Schematic representation of geometry and connection arrangements of the investigated cases.

3.3 Design of the connections

For the cases belonging to the Group 1 (Fig. 4), panel-to-panel and panel-to-foundation mechanical connections have been designed considering both the approaches A1 and A2. In the case A2 the connections are supposed made with sacrificial dissipative steel elements, whose cross-section areas have been determined as the minimum area required by the flexural demand and which stratify the inequality $R_d > N_{pl}\,\gamma_{Rd}$.

Commercial HDs have been designed in the case of the approach A1 to bear the tensile forces induced by the moment demand and, consequently, the number of nails is determined to satisfy the inequality $N_{pl} > R_d \gamma_{Rd}$.

The design of the connection for both approaches (A1 and A2) require the calculation of the moment and shear demand on each panel according to an iterative procedure [11]. In a first phase, an attempt value of the first vibration period - belonging to the constant branch of the design response spectrum - has been used to compute the equivalent horizontal seismic actions acting on the building. Those forces have been applied to the numerical model of the building (for the dir. X and Y, separately), which does not include the mechanical connections; this is the way the dimensions of the tensile and shear-resistant elements can be set depending of the moment and shear demand, respectively. In a second step, the same building has been modelled including panel-to-panel and panel-to-foundation connections to re-calculate the first vibration period: if the new period ranges in the constant branch of the response spectrum the design procedure of the first phase is accurate, otherwise a new attempt must be done by varying the period. Note that equivalent seismic actions have been determined by assuming a behavior factor q=2.0 in compliance with relevant design standards [6, 9].

Focusing on post-tensioned CLT walls (Group 2), no additional mild-steel components for energy dissipation are provided at the base of the panel, so that the response of the wall is dominated by the rocking motion. The design of post-tensioned walls (cross-section of the cable and its pre-stressing axial force) has been carried out the displacement-based approach after the New Zealand design guidelines [21].

3.4 Modelling of the wall-system

Modelling of CLT structures involves three different levels: *a*) CLT 'material', *b*) connecting zones, *c*) global structural scheme (floors, spandrels, etc.) [11, 22, 23]. Being out of the scope of this paper a detailed description of the modelling strategies and their effectiveness, the modelling criteria herein adopted in are summarized the following. The structural models have been developed with the software SAP 2000 v.18 [24].

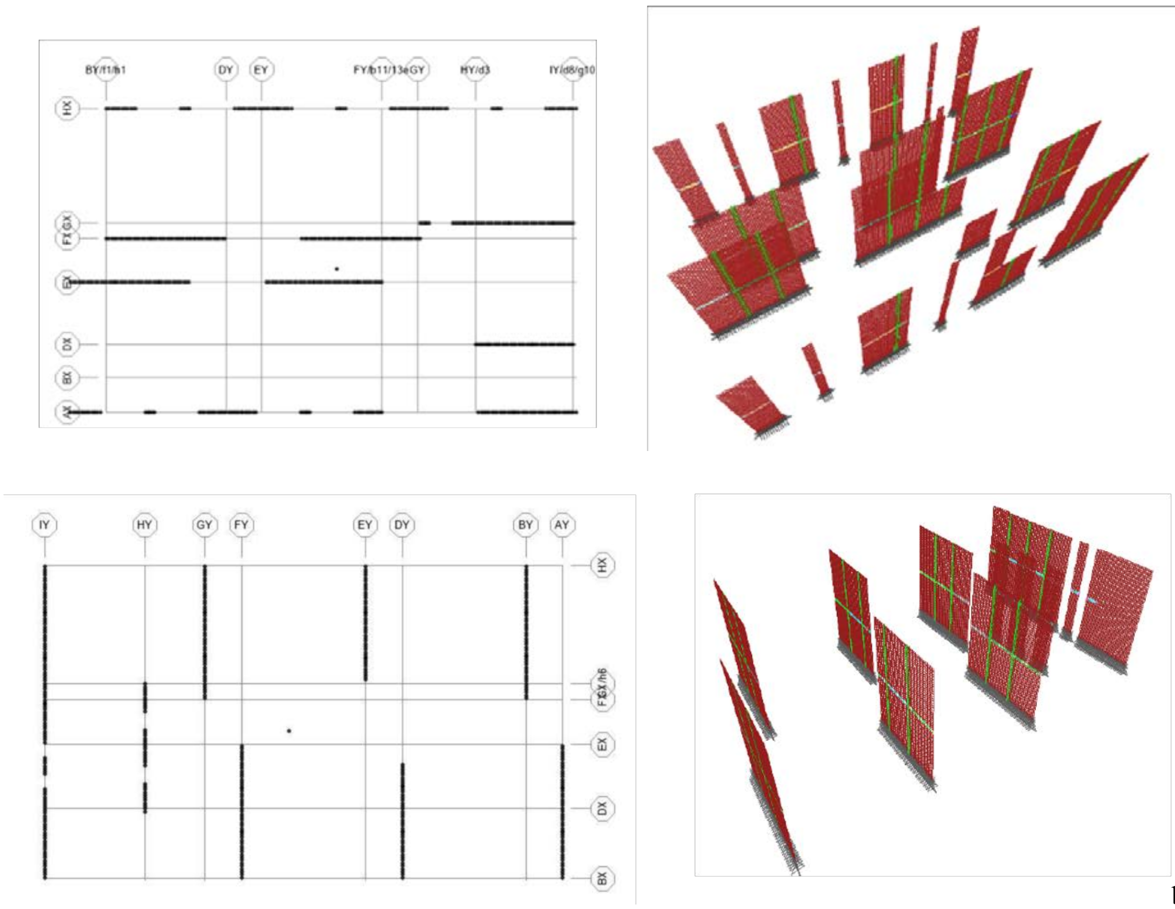
With regards to CLT 'material', simplified methodologies suitable for macro-scale modelling of the CLT material are available in literature proving comparable results [1]. In this paper, the in-plane behaviour of the CLT panel has been modelled through homogenized orthotropic 2D-shell elements: the panel thick is equal to the actual one, while the elastic moduli of the base material are reduced as a function of the effective number of layers present in vertical and horizontal direction.

More sophisticated is the schematization of wall-to-wall and wall-to-foundation connection zones. Generally, they are modelled on analogy with the reinforced concrete cross-sections, assuming the HDs as the tensile-resistant elements, the ABs as shear-resistant elements and timber-to-timber and timber-to-concrete contact as the part devoted to resist to compression forces [1, 25]. In this study, the tensile-resistant elements have been schematized as provided with tensile resistance only and unable to resist to compression, moment and shear (i.e., pendulums); instead, the timber-to-timber (or timber-to-foundation) contact has been schematized with pendulums provided with compression resistance only. Whereas the angle brackets have been schematized with flexural and shear-resistant frame elements. Tensile-resistant and contact elements have been schematized with an elastic-perfectly plastic (axial) behaviour, whose maximum strength is defined according to the selected SH criterion (R_d or N_{pl} in the case of approach A1 and A2, respectively). Instead, the maximum axial displacement capacity of the tensile-resistant elements has been assumed variable and equal to the values ε= 1% and ε= 2%. This strain range has been used to account for the uncertainties associated with the calibration of the threshold; this especially in case of HDs nailed to CLT panels where the length on which evaluate the axial strain is still of difficult evaluation [11]. The limit value of the axial strain in orthogonal-to-grain direction of the compressed elements has been set equal to 5% [11, 22].

Vertical joints fasteners of the coupled walls have been modelled with elastic springs, linked to the nodes of the adjacent panels, and provided with shearing stiffness (k_{ser}) calculated according to Eurocode 5 [5].

The modeling of the post-tensioned walls is made by means shells characterized by the same constitutive relationships used to describe traditional CLT walls complemented with a prestressed element to simulate the vertical cable, whose primary role is to ensure the system recentering. Timber-to-foundation contact has been modelled with zero-length contact elements.

To reduce the computational efforts related to the dense mesh adopted to model the CLT panels, two separate models have been developed to analyze the structural behavior in the two orthogonal directions X and Y. For seismic analysis purposes, the CLT floors have been modelled as in-plane infinitely rigid diaphragms connected to the vertical walls. The flexural and shear contribution of the spandrels has been neglected, being simply supported to the adjacent walls [11]. Figs. 5 and 6 show the 2D-shell models of the buildings relative to the Group 1 and 2, respectively.



a)

b)

Figure 5: Wall geometry and the structural model of the cases of the Group 1: a) direction X, b) direction Y.

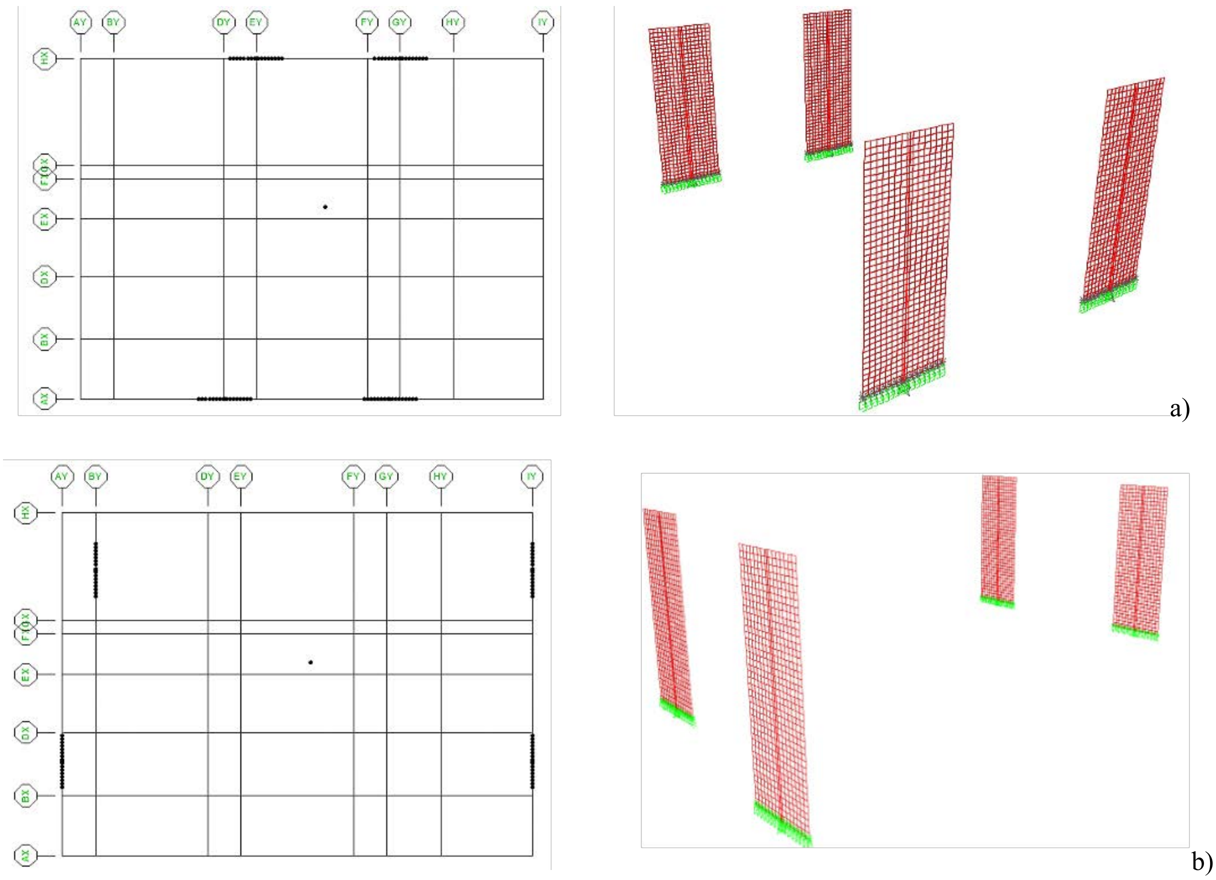


Figure 6: Position of the walls and view of the numerical model - Group 2: a) direction X, b) direction Y.

4 DISCUSSION OF RESULTS

4.1 Structural response

Figs. 7a and 7b report the pushover curves of the four analyzed numerical models described in Table 1 for the directions X and Y, separately. Moreover, the curves obtained with the two different values of axial strain of the tensile-resistant elements (ε= 1% and ε=2%) are reported for the sake of comparison.

The pushover curves point out some differences in terms of structural response of the system depending on the layout and performance of the connections. As a comparison between the two SH criteria, it is resulted that (*i*) the approach A2 involves higher post-elastic displacements than approach A1 (either for ε= 1% and ε=2%) and that (*ii*) the maximum base shear obtained in the case of the approach A1 is higher than those of the approach A2.

Both statement (*i*) and (*ii*) are due to the higher overstrength of the tensile-resistant element designed with approach A1 than approach A2. The higher overstrength of the approach A1 is due to the adoption of commercial cross-section for the HDs, while in the case of approach A2 the cross-section area of the tensile-resistant elements is calibrated to be equal to that strictly required by the moment demand. In particular, in the case (*i*) the overstrength of the tensile-resistant elements designed according to approach A1 address the failure mechanisms towards the crisis of compressed timber before that of the tensile-resistant element.

The overstrength resulted from the approach A1 with respect to A2 is accentuated in the cases of coupled walls, probably because of additional strength given by vertical joints in combination with overstrength of the base connections. Whereas this additional strength is not completely exploited in the case of approach A2, being the ultimate displacement of the walls achieved with a plasticization in both tensile and compressed zone. With regards the influence of the ultimate axial strain (ε) of the cases belonging to Group 1 it is evident that, despite double values of ε, the displacement capacity of the walls does not increase proportionally. This is occurs as the failure mechanisms of the walls are ruled by the achievement of maximum axial strain of timber in compression zones which limit the overall displacement capacity of the walls.

As fa as the Group 2 is concerned, the pushover curves are the same in comparison with direction X and Y. This because the same number of walls (four) - and with the same cross-section area and pre-stressed forces - have been used in the two orthogonal directions X and Y (Fig. 6). Comparisons between the pushover curves of the Group 1 and 2 show that in the second case additional overstrength is avoided, because the walls are designed to achieve exactly the base shear demand at ULS. Contrariwise, the displacement capacity of post-tensioned system is significantly higher than that of the Group 1 because dissipating seismic energy through the rooking behavior of the walls, without plastic dissipation concentrated in few tensile-resistant elements. The overstrength generated from the design approaches A1 and A2 of the Group 1 are more evident by looking at Fig. 8, where the maximum seismic capacity bearable by the structure at LS limit state expressed in terms of PGA/g is evaluated according to N2 method [26]. From the structural point of view, post-tensioned systems are optimal because designed to satisfy the seismic demand without additional overstrength and with optimal displacement capacity.

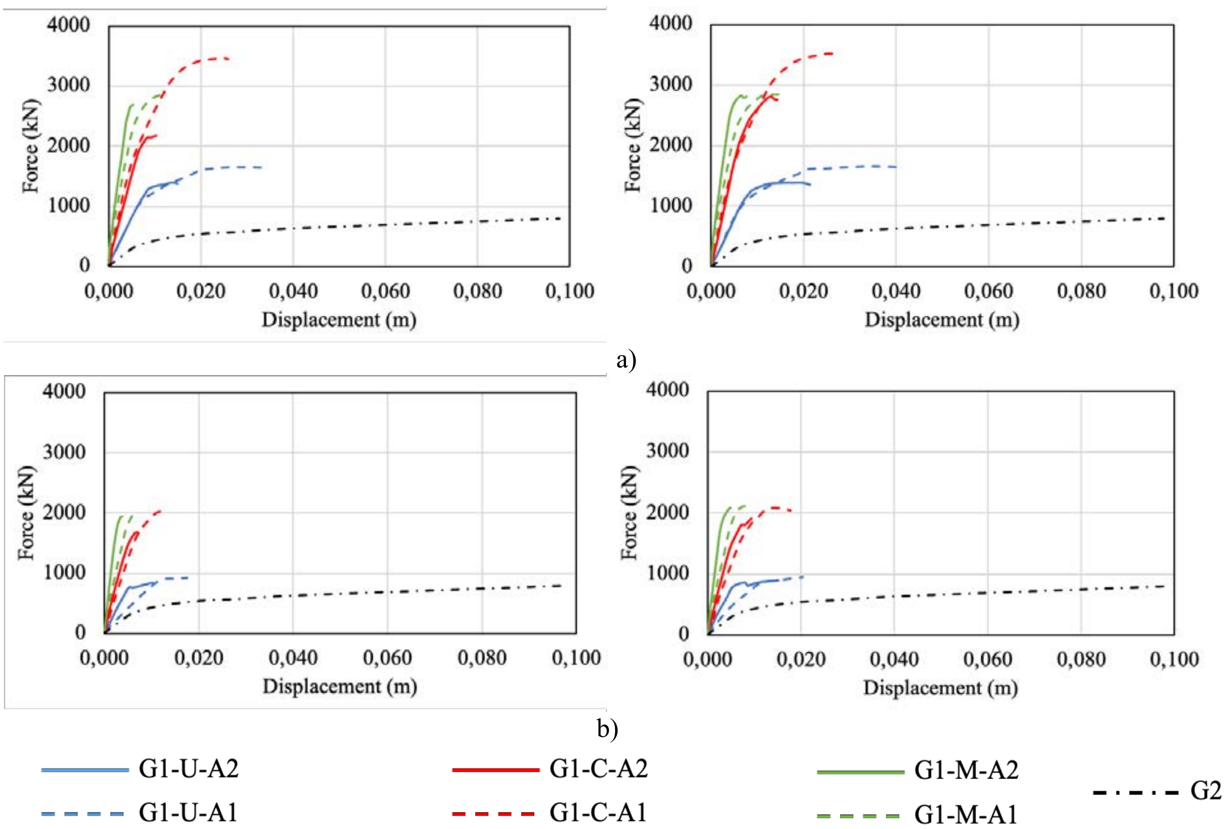


Figure 7: Comparison between the pushover curves: a) walls in direction X, b) walls in direction Y.

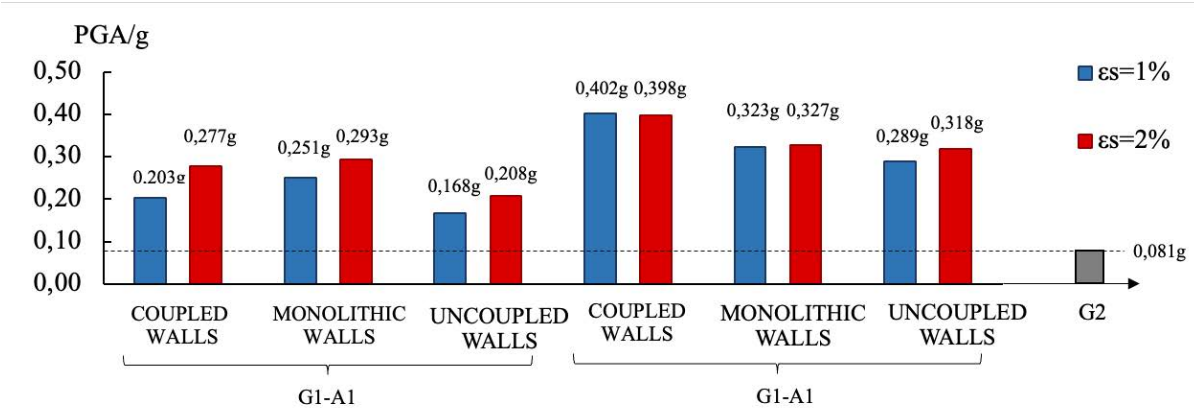


Figure 8: Comparison between the maximum PGAs at ULS bearable by the different structural solutions.

4.2 Some remarks on the damage tolerance

The obtained results - concerning the structural behavior of the various structural systems listed in Table 1 with respect to Life Safety Limit State condition - shown that low damage structural solutions (either those of the Group 1-A2 and Group 2) are more performant than Group 1-A1 (traditional CLT buildings), in terms of seismic behavior and particularly because the damage is gradually decreasing in the connecting zone passing from Group1-A1, to Group 1-A2, to Group 2.

Nonetheless, in these analyses, the influence of damage tolerance of the non-structural components on the overall seismic performance of CLT buildings has not been considered. In the general framework of Performance-Based Design (PBD) of buildings, the influence of non-structural components - such as infill walls and partitions, ceilings, claddings, contents etc. - cannot be neglected any more worldwide. In fact, many codes treat the problem of safety related to non-structural components, also including criteria for determining the seismic demand at different Limit States [6, 19, 27].

Even in presence of low-damage principal structural systems, damage to non-structural elements has severe impact in the building recovery, increasing the socioeconomic losses even for low intensity events (i.e., Damage Limit State) [18]. This aspect is not secondary, especially when (*i*) the displacement capacity and/or the inter-story drift of the structural system are significant, being the non-structural elements forced to accommodate such displacements with high probability to achieve high-level of damaging and needs to be removed and replaced with new components; (*ii*) the number of non-structural elements is elevated.

The latter two statements are typically recognized in post-tensioned systems because the reduced number of seismic-resistant walls due to the high seismic performance of the systems involves (necessary) the introduction of a significant number of non-structural components to realize infill and partitions. From this perspective, the system of the Group 1-A2 appears optimal because limits the damage of structural components and contemporarily requires a lower number of non-structural elements to realize the construction.

On the other hand, studies on the behavior of non-structural timber-based drywall equipped with low damage connections are available in literature [28, 29, 30]. Obviously, such systems reduce the nonstructural damage and serviceability of the construction is preserved especially for low-intensity earthquakes. Moreover, economic and environmental cost of a timber wall structure should take into account the probability of a reduced or nullified efficiency of non-structural elements, especially in presence of severe earthquake.

As final remark, it should be also considered that another issue affects the damage level of CLT buildings belonging to the Group 1. It regards the damage of CLT floors due to rocking motion of the vertical panels: due to the platform-type constructional technology (i.e., floor inserted between two vertical walls) the rocking movements of the vertical panels cause the crushing of the floors in orthogonal-to-grain direction, whose values of strength are very low about 2-3 MPa. Thus, it is worth revising the idea behind the traditional CLT building and introducing the '2.0 CLT building' composed with full-height CLT walls (i.e., balloon-type technology) equipped with damage tolerant connections at the base of the walls.

4.3 Construction Costs and Life Cycle Assessment

This Section reports preliminary observations on the influence on costs related to the different structural schemes investigated in this paper, also including Life Cycle Assessment (LCA) aspects. The latter measures the environmental impact associated with all stages of a building's life cycle as provided by the International Organization for Standardization ISO 14040 [31] and the European Standard EN 15978 [32].

In Group 1, advantages of approach A2 with respect to approach A1 can be mainly referred to the repairing phase after the seismic event. In fact, low damage-based approach A2, avoiding removing and disposal of damaged CLT walls, reduces demolition and reconstruction costs, material and energy use, pollution due to transport and fabrication. In fact, solutions which provide permanent damage of the connecting zones (e.g., Group 1-A1) require the demolition of the structure involving additional costs.

Conversely, a comparison from an economical and environmental point of view of the different structural systems of Group 1 and that of Group 2 requires analyzing many factors, concerning designing, and building aspects. All the solutions of the Group 1 (be it designed according to approach A1 or A2) are particularly massive; they require higher number of walls, and then quantities of material, because the CLT seismic-resistant walls are distributed all over the perimeter and in the inner of the building, while few internal partitions are present. Whereas, in the case of the Group 2, few seismic-resistant walls were required, and in addition non-structural lightweight timber infill panels (or glass façades) are used to close the perimeter of the buildings, as well as internal partitions.

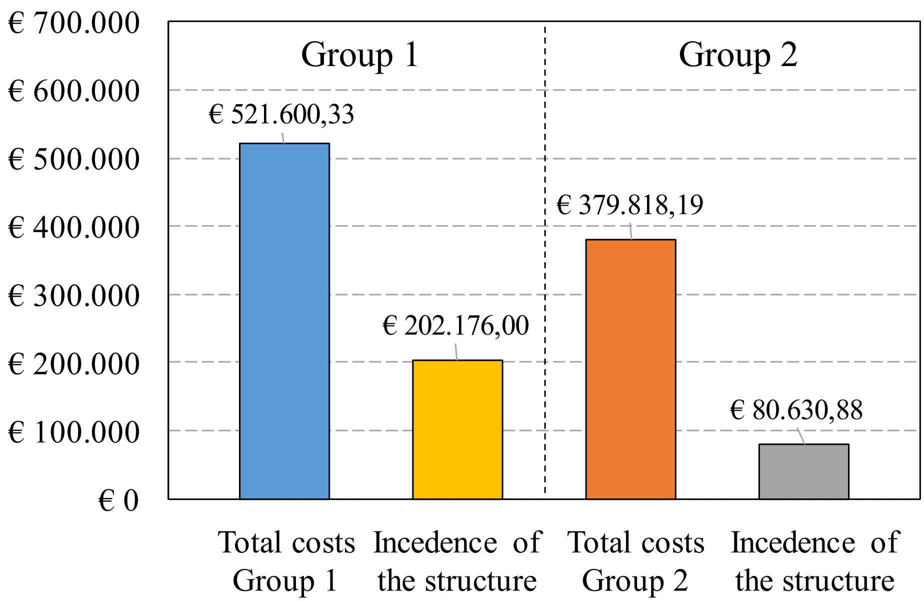


Figure 9: Comparison between construction costs.

As the structural components only are concerned, the weight of timber in cases of the Group 1 is greater than 30% with respect to the Group 2. Obviously, this aspect has an impact also on the construction costs involving transports and installing costs, material and energy use in producing as well as building phases. For the case study herein analyzed, the incidence of the costs of the structural system on total building costs of the Group 1 is about 38%, while in the case of post-tensioned solution (Group 2) is about 22%. In Fig. 9, the total costs of the solutions relative to the Groups 1 and 2, with indication of the incidence of the structural elements, are summarized.

Being out of the scope of the paper a detailed LCA, some remarks can be proposed based on the comparative analysis of the design procedures described herein:

- Structural solution of the Group 1 involved higher quantities of structural material than that of the Group 2, and consequently high environmental impact on the phases of extraction, production, packaging, and transport is expected in the first case.
- Traditional CLT buildings (Group 1-A1) characterized by severe seismic damage in the connecting zones imply higher environmental impact related to the phases of 'use' and 'end of life' due to demolition and disposal: whereas for those based on low damage (Group 1-A2 and Group 2) reduced or null damage is expected, also under high-intensity earthquakes.
- Costs related to non-structural components of the Group 2 are greater than those of the Group 1, because a higher number of nonstructural elements are required for such structural solution.

As matter of the fact, low-damage based solutions represented by the Group 1-A2 and Group 2 are both evidently more effective in terms of damage control and sustainability than that of the Group 1-A1. Those of the Group 1-A2 have the further advantage of reducing the number of non-structural components and then costs related to their eventual damaging. Note that in both cases of the Group 1-A2 and Group 2 low damaged non-structural components can be also enclosed in the structure, but with the disadvantages of additional costs related to the high level of prefabrication and specialized technology required by such systems.

Furthermore, it is worth noting that in the case of Group 2 few seismic-resistant walls are distributed over the perimeter of the building and in the inner. This characteristic is positive because lead to a larger architectural freedom in organizing inner spaces and outer openings, but it is negative because enlarges the need of introducing infill panels and partitions if compared to the systems of the Group 1-A2.

5 CONCLUSIONS

Among the various timber building typologies, massive CLT buildings have grown rapidly in the last decades both for residential and non-residential buildings. This rapid development has not been supported by up-to-date and codified design rules, leading to design approaches which limit the overall performances of the CLT buildings because based on structural considerations only.

In this context, an insight on the influence of design approaches of the connections on the overall performance of CLT buildings has been presented in this paper, highlighting the needs of a comprehensive and integrated design methodologies including the aspects related to damage control, costs and environmental impact also. The results of the conducted research con be summarized as follows:

i. The adoption of low damage structural systems is a fundamental requisite to achieve high performances of CLT buildings in terms of structural behavior, damage control, costs and environmental impact.
ii. Traditional CLT buildings (Group 1-A1) are low performant because damage is

expected in the connecting zones depending on the level of the seismic demand. After an earthquake the structural components must be demolished/repaired and then reconstructed involving additional costs and environmental impact.

iii. Low damage solutions (Group 2) are efficient from structural, architectural and environmental point of view, but the high damage to non-structural components - related to high displacement capacity of the structure - can compromise their serviceability. In fact, if damage control systems for nonstructural components are not installed, costs related to their damage can become very significant. On the other hand, low damage solutions are expansive for non-structural elements and increase the construction costs.

iv. CLT buildings equipped with low damage connecting systems at the base of the panels (Group 1-A2) represent a balance between traditional constructions (Group 1-A1) and those post-tensioned (Group 2), as no damage in structural components is expected and the use of non-structural element is limited.

The comparative analyses described herein are not certainly comprehensive and need further investigations and validations. Attention must be primarily focused on the technology and on the expected dissipative capabilities of the connections, whose optimization requires a rigorous scientific approach and an efficient code implementation.

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