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On the Role of Seismic Damage Tolerance on Costs and Life Cycle of CLT Buildings

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Abstract: This paper presents a contribution to reviewing the most common seismic design procedures of CLT buildings and their implications on structural features and technological solutions. Attention is particularly focused on the overall seismic performance, damage tolerance, construction costs and environmental impact. It is intended as a baseline for a more comprehensive study, thus the assessment is made with reference to a real building, representative of a class of common buildings recently designed and erected in many Italian regions exposed to low and moderate seismic hazards. As usual, the analysis was 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 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 whenever damage tolerance is accounted for in design procedures.

Keywords: CLT buildings; damage tolerance; structural timber; connections; construction costs; life cycle assessment

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1. Introduction

The introduction of high-performing engineered wood products (i.e., Glue-Lam, Cross-Laminated Timber, Laminated Veneer Lumber, etc.) encouraged the construction of timber buildings worldwide [1]. Today, 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 uses [2]. The beneficial properties of the base material related to reduced weight, thermal insulation and its natural origin place CLT buildings at very high levels of structural, seismic, energetic and sustainability performance [3–5].

Traditional CLT buildings are erected with platform construction technology: they are made of horizontal CLT floors installed between two vertical CLT walls (as a "sandwich"), interrupting the vertical continuity of the walls [6]. Wall-to-wall and wall-to-foundation contact consist of mechanical connections made with (a) hold-downs (HDs) bearing tensile forces due to moment demand, and (b) angle brackets (ABs) aimed at covering the shear demand. These connections play a crucial role in the seismic response of CLT buildings [7,8]. They are designed according to the strength criteria provided by Eurocode 5 [9], i.e., fail according to Johansen's plastic failure mechanisms under seismic-like actions characterized by localized timber embedding combined with the yielding of metal fasteners [10–16]. Basically, such mechanisms are ductile, but sometimes they are anticipated by brittle failure modes if end edge distances or spacing between the metal fasteners are not respected, nullifying the dissipative behavior of the CLT building. As a main drawback, traditional HDs and ABs point out the activation of permanent damage mechanisms allocated in the vertical timber panels due to the embedding of wood, as documented in Figure 1, where typical

failure mechanisms of HDs and ABs observed during experimental testing are shown. Due to timber damage, vertical timber panels result in being heavily damaged, making their reparation impractical after an earthquake, so the demolition of the building (or some of its portions) results in being the most rational option, entailing additional costs due to both demolition and reconstruction, recycling of materials and environmental impact.

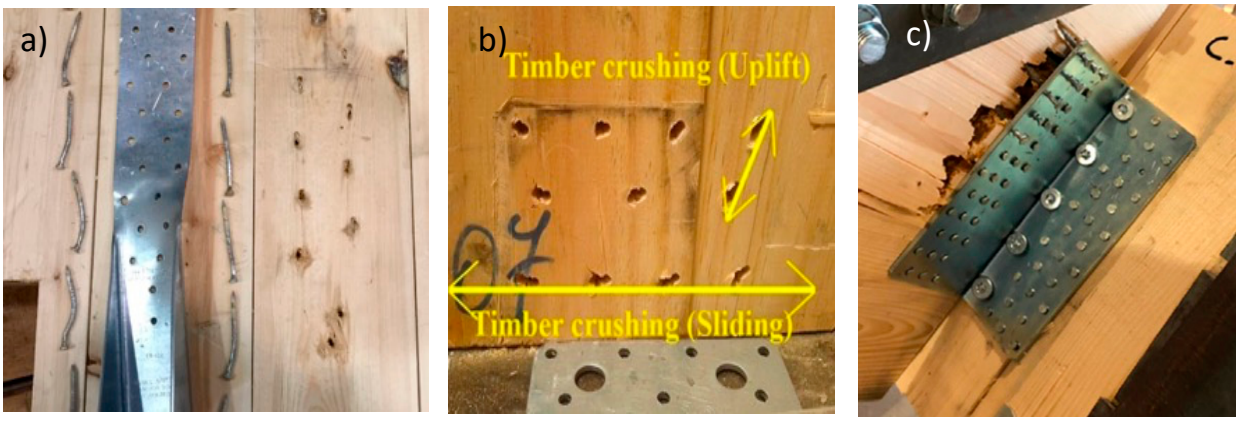


Figure 1. Typical failure modes and damage in connections made with hold-downs and angle brackets: (**a**) Liu et al. 2029 [12], (**b**) Benedetti et al. 2019 [13], (**c**) D'Arenzo et al. 2021 [15].

The issue of damage prevention in CLT timber panels represents a (relatively) recent research topic. To overcome this problem, alternative construction technologies, connection systems and their design criteria are available in the literature. Connecting systems, in substitution of HDs and ABs have been proposed with the aim of concentrating the damage in the metallic parts only (steel bars or plates with particular shapes), as documented in [17–20]. As a shortcoming, these connections, which yield tension and/or shear after a seismic event, must be removed from the vertical timber panels, substituted or changed in position along the base of the wall to repair the connections: these operations produce damage of the CLT panel, additional repairing or demolition costs and environmental impact related to the phases of 'use' and 'end of life' within the Life Cycle Assessment (LCA) framework.

Indeed, low-damage timber systems were first claimed in New Zealand in 2005 [21]. They are made with post-tensioned rocking timber walls connected by means of 'plug and play' connecting systems (placed at the base of the vertical walls only) to the foundation [22]. These connections, which consist of fuse-type metal bars in which the plastic damage is concentrated, can protect the vertical timber panels against damage due to the embedding of wood. The metal bars can be easily substituted after a seismic event, restoring the original structural behavior of the seismic-resistant walls. Taking inspiration from this idea, studies on damage-tolerant and replaceable connecting systems in substitution of traditional HS and ABs have been introduced in the literature. For instance, Loo et al. 2014 [23], proposed a damage-tolerant slip-friction device that works as a tensile-resistant element when placed at the base of CLT walls, while Wrzesmiak et al. 2016 [24], introduced damper-based HDs using high force-to-volume damping devices made of a steel shaft sliding within a tube. Hashemi et al. 2021 [25], proposed a novel low-damage and replaceable connection made with a friction-based device for CLT rocking structures integrated with springs to allow the recentering of the shear wall.

The conceptual design of the connections plays a key role in the combination of the seismic damage limitation and tolerance, and overall costs, including those associated with the structural maintenance, reduction and optimization of the Life Cycle process for CLT buildings (Figure 2). In this framework, structural solutions oriented towards damage control in structural and non-structural components, cost reduction related to demolition of structural and non-structural components and improvement of the aspects influencing

the LCA should be selected in engineering practice. Moreover, both in the case of new or existing buildings designed with rules that are obsolete or not codified, BIM tools can also be useful to include in the overall design process, allowing management of the damage control, repairing, costs and Life Cycle of buildings [26].

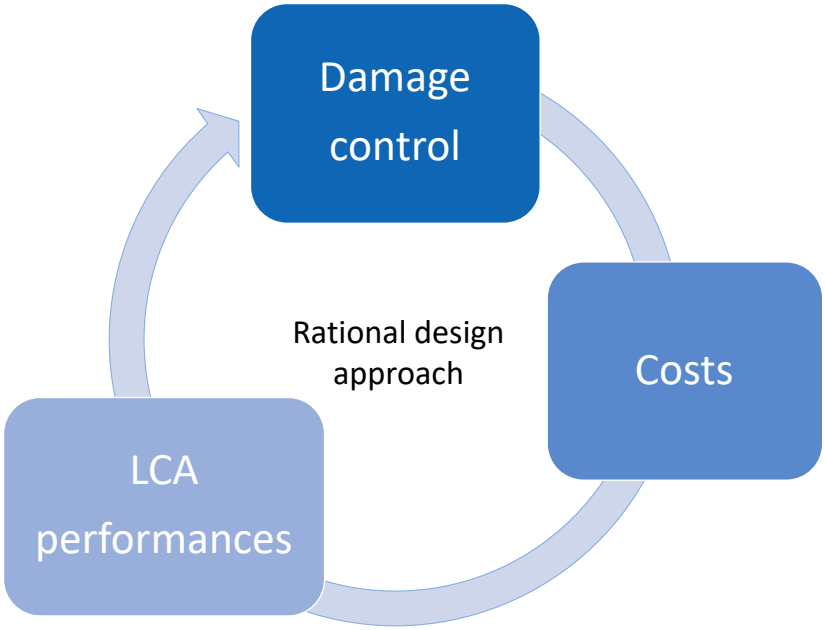


Figure 2. Optimized design approach for timber buildings.

In the field of timber-based structural solutions, studies devoted to investigating the role of the design approaches of the connections and technological construction technologies on structural behavior, expected damage, costs and sustainability of the structural system have not been yet addressed.

This paper presents a first insight into the relationship between these aspects for CLT buildings erected in seismic-prone areas, and its aim is that of promoting a discussion within the scientific community concerning this particular task. The main research objectives can be summarized as follows: (i) to highlight the role of construction technologies and design criteria of the connection between damage tolerance, construction costs and Life Cycle aspects; (ii) to analyze the influence of damage-tolerant connections between the overall seismic behavior of CLT buildings, costs and Life Cycle; (iii) to explore the crucial role played by the non-structural components in terms of costs, damage level, architectural features of buildings and Life Cycle. In this respect, comparative analyses between traditional and damage-tolerant CLT buildings have been conducted. A real two-story CLT school building located in northern Italy was considered a sample building, with its seismic design varying the geometry of the walls and typology of connections. A comparative analysis of the different structural layouts was performed depending on the results of nonlinear static analyses (e.g., pushover).

Research outcomes allowed some reflections on the relevance of optimized and integrated design procedures for CLT buildings, 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, construction costs, impact on the Life Cycle performance of the buildings and interaction with architectural and non-structural features.

2. Role of the Design Criteria

Despite the fact that damage tolerance, costs and LCA together represent three fundamental parameters governing a rational design approach for CLT structures, the relationship between them is not investigated in the scientific literature and each topic is treated sepa-

rately [22,27,28]. In this perspective, this paper investigates the correlation between damage tolerance, costs and LCA of CLT buildings, highlighting that the relationship is mainly governed by the design criteria adopted for the (dissipative) zones in wall-to-wall and wall-to-foundation connecting systems.

In the context of the capacity design methodology, the design of the connections is severely governed by the Strength Hierarchy (SH) criterion established among the components present in the connecting zones [29]. The SH criterion aims to ensure that seismic loads generate an inelastic, ductile response in selected regions of the structure given that the rest of the components behave in the elastic range withstanding the forces associated with the plastic strength of the critical regions. As a result, overall and local protection from brittle phenomena can be achieved.

To date, national and international codes [9,29–31] and guidelines [32] refer to a single type of dissipative mechanism—those required for the traditional CLT buildings—based on Johansen's failure modes. For such an approach, identified as Approach A1 in the following, the non-dissipative elements are the metallic ones (e.g., 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). Thus, in this case, the axial strength of metallic elements HDs and ABs (N_{pl}) must be greater than the Johansen's strength (R_d) of the connection amplified from the overstrength factor (γ_{Rd}). Due to the cyclic nature and intensity of earthquakes, high and permanent damage to 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 for buildings located in high seismicity regions, where high levels of ground shaking may even appear 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 [33]. As a consequence, a relevant impact on the costs related to demolition and/or reconstruction of structural and non-structural components, recycling of materials and sustainability of the construction seems to be intrinsically accepted.

Table 1. Description of the Strength Hierarchy criteria.

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 calculated according to Johansen's theory—γ_{Rd} = overstrength factor of the connection.

As discussed above, alternative damage-tolerant connections can represent an effective solution to reduce costs related to damage to structural components after an earthquake and to improve the sustainability of CLT buildings, although specific HS rules are still not included in the current standards. The seismic dissipation is concentrated into sacrificial (plug and play) metallic elements which work in tension or shear (bars, steel plates, etc.), while vertical timber panels are non-dissipative and behave elastically. As an advantage, timber components are preserved from plastic damage after the seismic event and no costs related to demolition and/or reconstruction or recycling arise. For this approach, named Approach A2 in Table 1, the HS requires that the Johansen's strength (R_d) of the connection must be greater than the axial strength of metallic elements HDs and ABs (N_{pl}) amplified from the overstrength factor (γ_{Rd}). Nevertheless, in Sandoli et al. 2021 [29], different structural behaviors achieved by CLT walls designed according to Approaches A1 and A2 have been compared.

Furthermore, problems related to damage tolerance of non-structural elements, forced to accommodate the inter-story drift of the seismic resistant structure, can significantly impact repair/reconstruction costs, Life Cycle and the safety of people in the case of low-

and high-intensity earthquakes [34]. In the following, some considerations on this issue are addressed.

3. Methodology

3.1. Description of the Sample Building

The two-story CLT building (Figure 3) is representative of a class of common buildings recently erected in many Italian regions; therefore, an extended search of technical data was made resulting in the acquisition of the design reports and drawings. The building accommodates school functions and thus belongs to the Class of Use III, after IBC 2018 [29]. The main consequences of this classification are the level of ground motion design intensity—increased return period of the actions compared with ordinary buildings—and the type of verifications at the Damage Limit State (DS) based on the critical region strength rather than inter-storey drift. The real building was designed to be built in a low-intensity seismic zone, where a peak ground acceleration on bedrock is equal to a_g = 0.081 g at a Life Safety (LS) limit state.

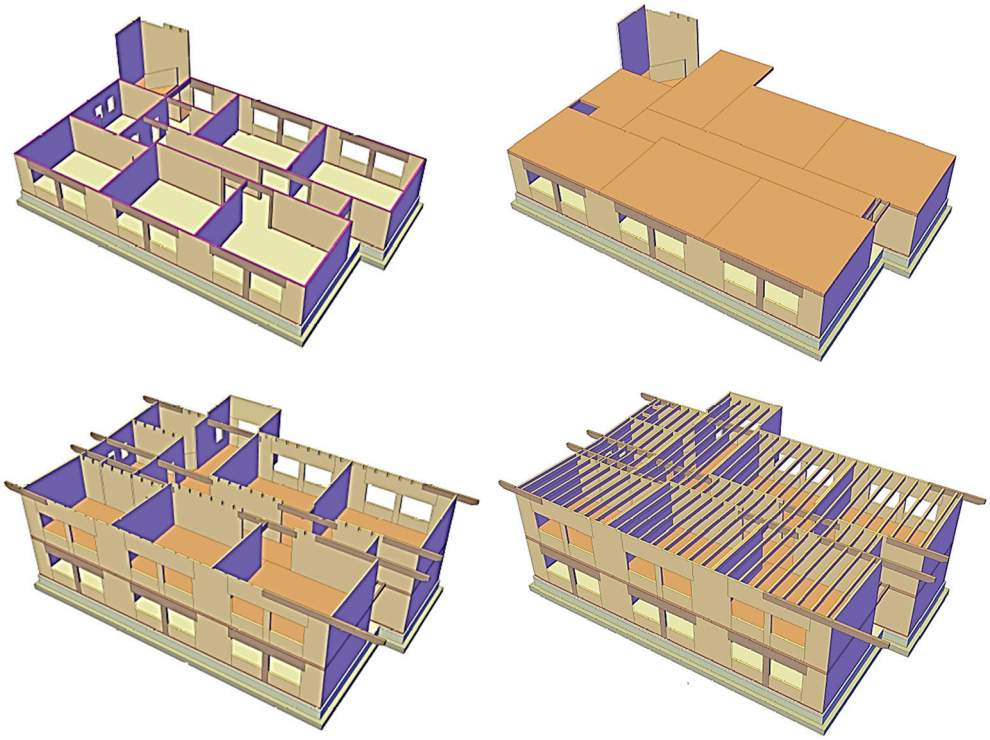


Figure 3. Views of the sample CLT building.

In the real design, the structure was made of 140 mm thick five-ply seismic-resistant CLT walls, where the thickness of the two vertical external layers was equal to 40 mm and those internal (both in vertical and horizontal directions) were 20 mm thick. The strength class of CLT corresponds to solid wood class C24, derived according to ETA-14/0349 [35].

The construction technology followed the platform-type system, where vertical walls are interrupted inter-storey by horizontal CLT floors interposed between two consecutive vertical panels. The floors were composed of 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 was used to refer to realistic building geometry, material properties and live and dead loads. The arrangement of the walls herein addressed is the same as provided by the real project and shown in Figure 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, costs and Life Cycle of CLT buildings, different structural schemes were considered to model the CLT walls of the mock-up building described in the previous section. Figure 4 shows the four structural schemes adopted, divided into two main Groups:

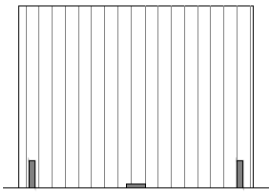
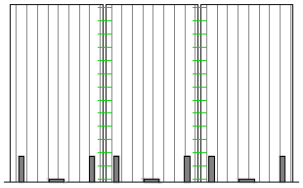
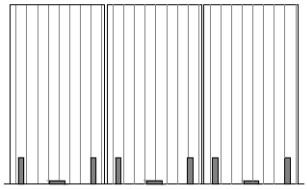
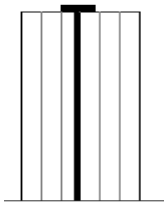
	Group 1		**Group 2**
G1-M	G1-C	G1-U	G2
CLT monolithic walls	CLT coupled walls	CLT uncoupled walls	Post-tensioned walls
			
Tensile-resistant elements at the edges only	Tensile-resistant elements distributed and vertical joints	Tensile-resistant elements distributed and no vertical joints	Rocking wall without additional tensile-resistant elements

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

(a) Group 1 (G1), including Monolithic (M), Coupled (C) and Uncoupled (U) CLT walls, designed considering both Approach A1 and A2, listed in Table 1;

(b) Group 2 (G2), including the post-tensioned CLT system only, characterized by a pure rocking behavior only (i.e., without additional dissipaters at the base).

The coupling between the walls (case G1-C) was assumed to be made of horizontal metal fasteners uniformly distributed along the vertical joints designed to behave elastically and provided with elastic stiffness (k_{ser}), calculated according to Eurocode 5, whereas the geometrical dimensions of the walls and the number of layers of CLT panels were the same in each scheme. Note that the vertical timber panels were not in direct contact with the reinforced concrete foundation, but a horizontal timber beam was interposed between the vertical panel and the foundation involving orthogonal-to-grain timber-to-timber contact at the interface.

3.3. Design of the Connections

For the cases belonging to Group G1 (Figure 4), panel-to-panel and panel-to-foundation mechanical connections were designed considering both A1 and A2 approaches, listed in Table 1. In the case of A2, the connections were supposed to be made with sacrificial dissipative steel elements, whose cross-section areas were determined as the minimum area required by the flexural demand satisfying the inequality $R_d > N_{pl}\ \gamma_{Rd}$.

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

The design of the connection for both approaches (A1 and A2) required the calculation of the moment and shear demand on each panel according to an iterative procedure [29]. In the first phase, an attempt value of the first vibration period—belonging to the constant branch of the design response spectrum—was used to compute the equivalent horizontal seismic actions acting on the building according to the Lateral Force Method. Those forces were 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 on the moment and shear demand, respectively. In the second step, the same building was modeled, including panel-to-panel and panel-to-foundation connections to re-calculate the first vibration period: if the new period ranged in the constant branch of the response spectrum the design procedure of

the first phase was accurate, otherwise a new attempt needed to be performed by varying the period. Note that equivalent seismic actions were determined by assuming a behavior factor q = 2.0 in compliance with relevant design standards [29,32].

Focusing on post-tensioned CLT walls (Group 2), no additional mild-steel components for energy dissipation were provided at the base of the panel, so that the response of the wall was dominated by the rocking motion. The design of post-tensioned walls (cross-section of the cable and its pre-stressing axial force) was carried out according to the Displacement-Based Design approach, as provided within the Australian and New Zealand design guidelines [36]. The background literature confirms that such an approach is typically used for designing post-tensioned rocking structures (made with reinforced concrete or with timber), with the displacement capacity being a key parameter governing the overall behavior of such types of structures [37,38]. Examples of the application of the Displacement-Based Design approach are documented in [39,40].

3.4. Design and Modelling of the Wall System

The cross-section dimensions of the vertical CLT walls (both for those of Group 1 and Group 2) were determined as a function of the vertical loads. The vertical walls were 5-ply panels 140 mm thick, while the horizontal panels (i.e., floors) were 7-ply panels 240 mm thick. The strength class corresponds to solid wood class C24, in compliance with the Standard ETA-14/0349 [35].

The modeling of CLT structures involved three different levels: (a) CLT 'material', (b) connecting zones and (c) global structural scheme (floors, spandrels, etc.) [29,40–42]. Being out of the scope of this paper, a detailed description of the modeling strategies and their effectiveness and the modeling criteria herein adopted are summarized in the following. The structural models have been developed with SAP 2000 v. 18 software [43].

With regards to CLT 'material', simplified methodologies suitable for macro-scale modeling of the CLT material are available in the literature. Basically, such approaches calculate the elastic moduli of the CLT by reducing the multi-layer material in a single-layer one: for the two in-plane orthogonal directions of the panels, the contribution of the layers having the grain direction parallel with respect to the considered one is accounted for. It has been proved that this approach provides reliable modeling of the CLT material and does not affect the global behavior of the CLT walls. This is because the contribution of the in-plane deformability of the panels, with respect to the global deformability of the multi-story CLT walls, is negligible compared to that provided by the mechanical connections. [2,41]. In this paper, the in-plane behavior of the CLT panel was modeled through homogenized orthotropic 2D shell elements: the panel thickness 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 the vertical and horizontal directions.

More sophisticated is the schematization of wall-to-wall and wall-to-foundation connection zones. Generally, they are modeled on an analogy with the reinforced concrete cross-sections, assuming the tensile-resistant elements are equivalent to steel rebars, the shear-resistant elements are equivalent to stirrups and timber-to-timber and timber-to-concrete contact as the part devoted to resist compression forces [2,44]. In this study, the tensile-resistant elements were schematized as provided with tensile resistance only and unable to resist compression, moment and shear (i.e., pendulums). Instead, the timber-to-timber (or timber-to-foundation) contact was 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 were schematized with an elastic-perfectly plastic (axial) behavior, whose threshold strength is defined according to the selected SH criterion (R_d or N_{pl} in the case of approaches A1 and A2, respectively). Instead, the maximum axial displacement capacity of the tensile-resistant elements was assumed variable and equal to the values $\varepsilon = 1\%$ and $\varepsilon = 2\%$. This strain range was used to account for the uncertainties associated with the calibration of the threshold, especially in the case of HDs nailed to CLT panels where the length on which the axial

strain is evaluated is still difficult to evaluate [29]. The limit value of the axial strain in the orthogonal-to-grain direction of the compressed elements was set equal to 5% [28,42]. Vertical joints fasteners of the coupled walls were modeled with elastic springs, linked to the nodes of the adjacent panels and provided with shearing stiffness (k_{ser}) calculated according to Eurocode 5 [9].

The modeling of the post-tensioned walls is made by means of shell elements 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 system recentering. Timber-to-foundation contact was modeled with zero-length contact elements to simulate the no-tension contact.

To reduce the computational efforts related to the dense mesh adopted to model the CLT panels, two separate models were developed to analyze the structural behavior in the two orthogonal X and Y directions. For seismic analysis purposes, the CLT floors were modeled as in-plane infinitely rigid diaphragms connected to the vertical walls. The flexural and shear contribution of the spandrels was neglected, being simply supported by the adjacent walls [29]. Figures 5 and 6 show the 2D shell models of the buildings relative to Group 1, while Figures 7 and 8 those relative to the Group 2.

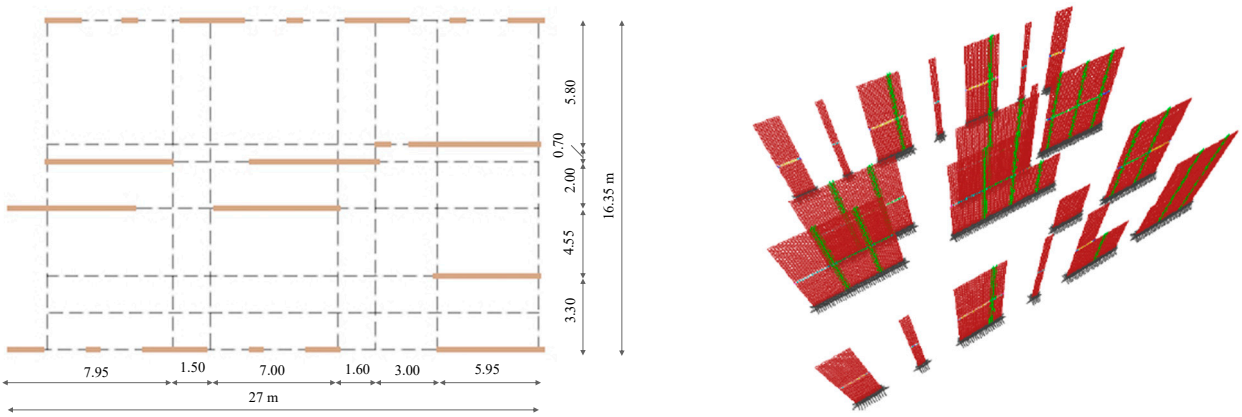


Figure 5. Wall geometry and the structural model of the cases of Group 1 along the X direction.



Figure 6. Wall geometry and the structural model of the cases of Group 1 along the Y direction.

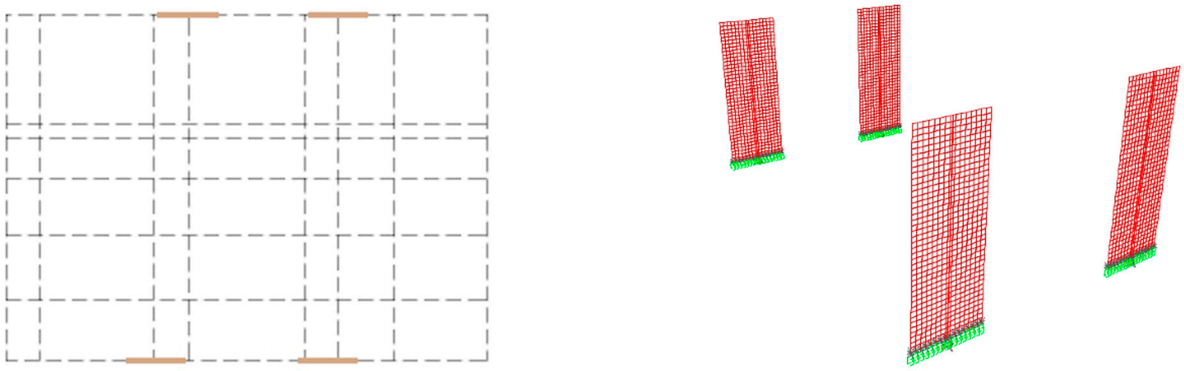


Figure 7. Position of the walls and view of the numerical model—Group 2 along X direction.

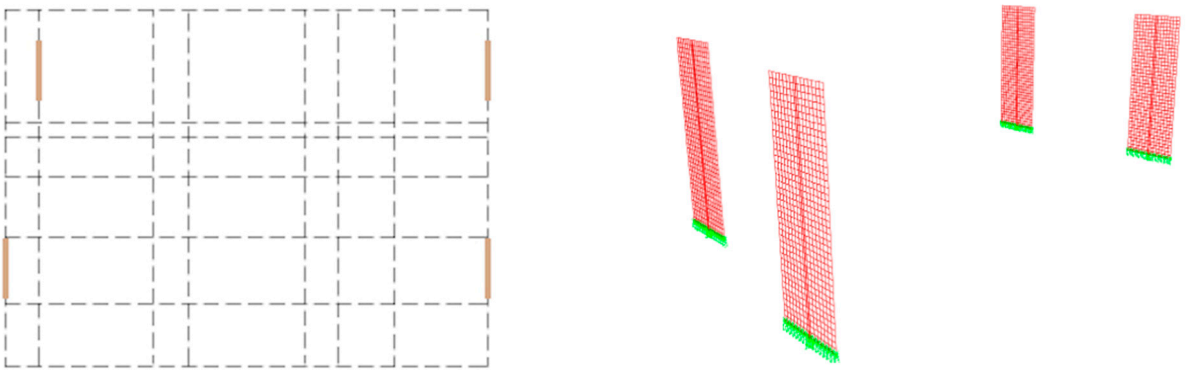


Figure 8. Position of the walls and view of the numerical model—Group 2 along Y direction.

3.5. Structural Analyses

The structural analysis of the models provided in Figure 4 is based on non-linear static analyses (e.g., pushover). The in-plane behavior of the CLT walls was modeled by adopting a lumped plasticity model, where the non-linear behavior is concentrated in the connecting systems only (see previous section).

The pushover analyses were conducted by adopting an increasing lateral force distribution, applied to the center of masses of the models, for the X and Y directions separately. Two different force distributions were considered: one proportional to the seismic masses at each story and one proportional to the first modal shape obtained through modal analysis. In the following, the results relative to the second force distribution are discussed because the results were less conservative. The lateral forces were increased up to the failure mechanisms of the walls, represented by the achievement of the ultimate displacement capacity within the connecting zones.

4. Discussion of Results

4.1. Structural Behavior

Figure 9a,b reports the pushover curves of the four analyzed numerical models described in Table 1 for the X and Y directions separately. Moreover, the curves obtained with the two different values of the axial strain of the tensile-resistant elements (ε = 1% and ε = 2%) are reported for the sake of comparison.

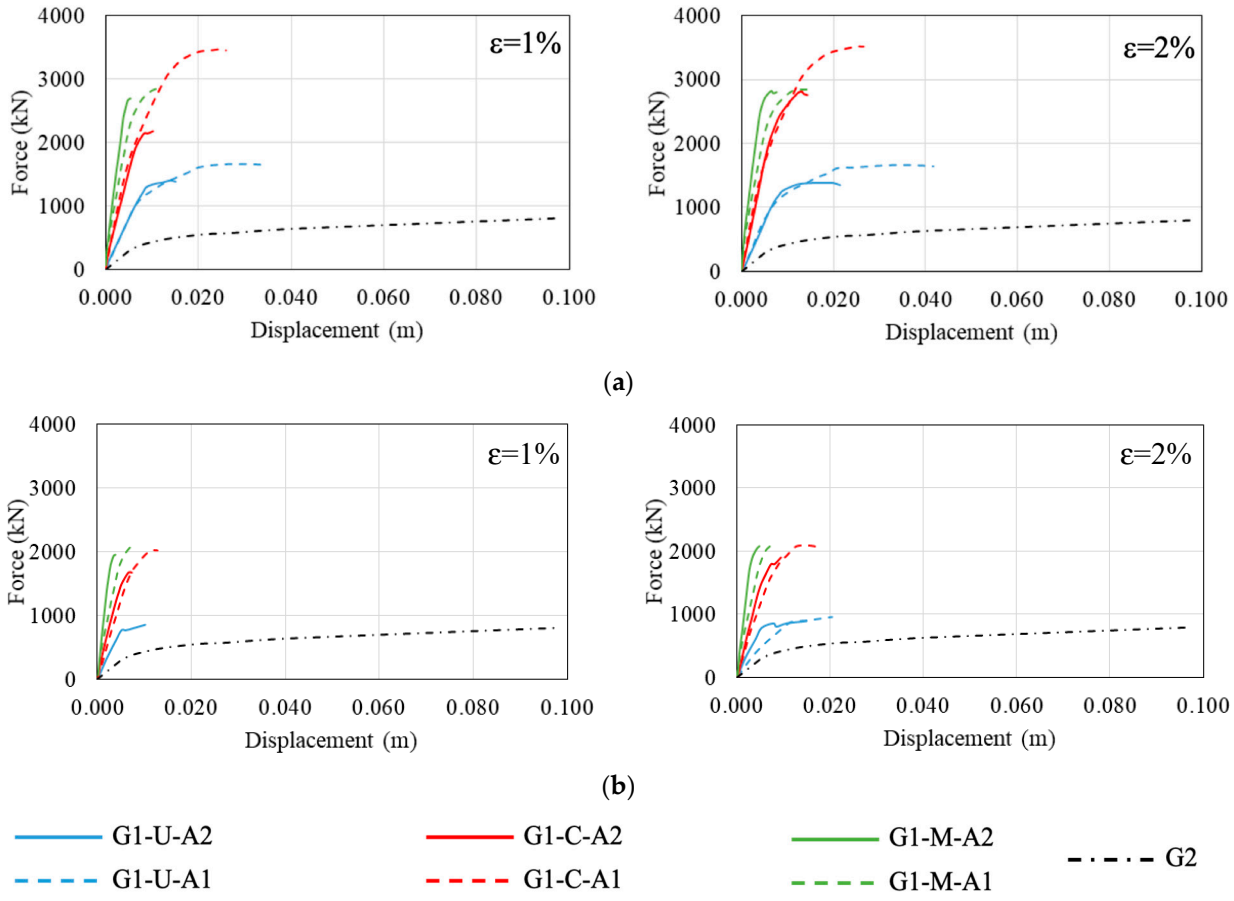


Figure 9. Comparison between the pushover curves: (**a**) walls in direction X, (**b**) walls in direction Y.

The pushover curves point out some differences in terms of the structural response of the systems depending on the layout and performance of the connections. As a comparison between the two SH criteria herein adopted, it is evident that (i) Approach A1 involves higher post-elastic displacements than Approach A2 (either for $\varepsilon = 1\%$ and $\varepsilon = 2\%$ assuming an ultimate axial strain for the tensile-resistant elements) and that (ii) the maximum base shear obtained in the case of Approach A1 is higher than those of Approach A2.

Both statements (i) and (ii) are due to the higher overstrength of the tensile-resistant elements designed with Approach A1 than Approach A2. The higher overstrength of Approach A1 is due to the adoption of the 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 of (i), the overstrength of the tensile-resistant elements designed according to Approach A1 addresses the failure mechanisms towards the crisis of compressed timber before that of the tensile-resistant elements.

The overstrength resulting from 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 the 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.

If the influence of the ultimate axial strain (ε) of the cases belonging to Group G1 is analyzed, it is observed that, despite double values of e, the displacement capacity of the walls does not increase proportionally. This occurs as the failure mechanisms of the walls are ruled by the achievement of the maximum axial strain of timber in compression zones which limits the overall displacement capacity of the walls.

As far as Group G2 is concerned, the pushover curves are the same in comparison with directions X and Y. This is because the same number of walls (four)—and with the same cross-section area and pre-stressed forces—was used in the two orthogonal X and Y directions (Figure 6). Comparisons between the pushover curves of Group G1 and G2 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 the post-tensioned system is significantly higher than that of Group 1 because of dissipating seismic energy through the rooking behavior of the walls, without plastic dissipation concentrated in a few tensile-resistant elements. The overstrength generated from the design approaches A1 and A2 of Group 1 are more evident by looking at Figure 10, where the maximum seismic capacity bearable by the structure at the LS limit state expressed in terms of PGA/g is evaluated according to the N2 method for the cases of Group G1 [36] and according to the Capacity Response Spectrum in the case of G2 [45]. From the structural point of view, post-tensioned systems are optimal because they are designed to satisfy the seismic demand without additional overstrength and have optimal displacement capacity.

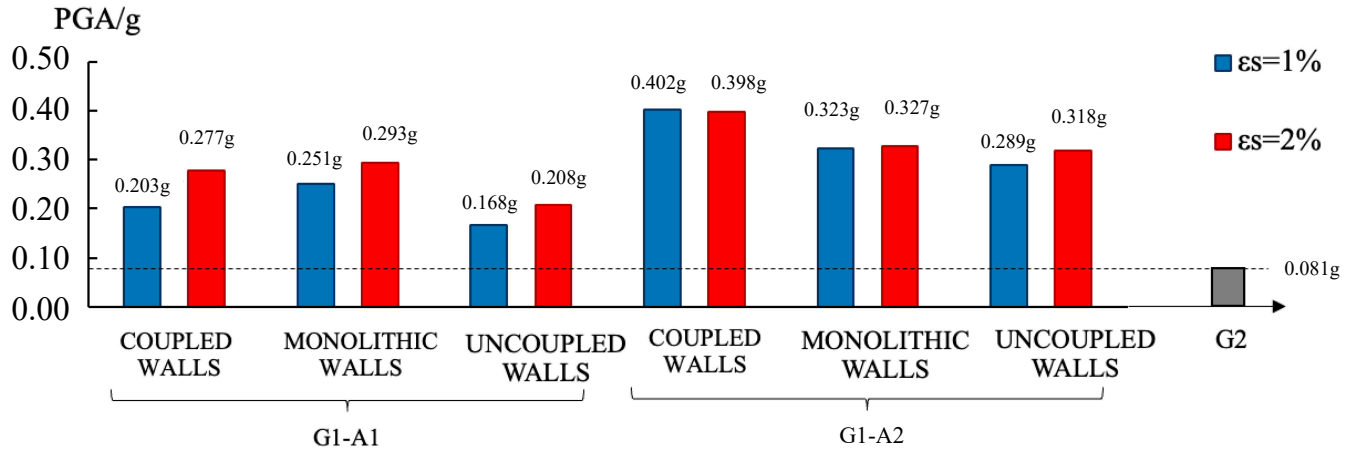


Figure 10. Comparison between the maximum PGAs at ULS bearable by the different structural solutions.

4.2. Remarks on Architectural Layout

In addition to the different structural behavior that emerges in Figure 7, the solutions of Group G1 and G2 give rise to different possibilities in terms of managing the architectural spaces of the building. This latter aspect alone can also be predominant to choose the structural solutions to adopt as a function of the final use destination of the building. For instance, large halls are preferred in the case of non-residential buildings.

By looking at the plans of the building reported in Figures 5 and 6, the reduction in the number of lateral-resistant walls in the case of Group 2 with respect to Group G1 is evident.

Obviously, the lower number of walls implies (a) increased flexibility in the design of the architectural layout; (b) a reduction in the total weight of timber and then of the construction costs and (c) an increment of non-structural components. While statements (a) and (b) are surely positive aspects related to the structural solution G2, point (c) can negatively affect this solution because a higher number of non-structural components (partitions and facade walls) are potentially exposed to damage in case of an earthquake. The correlation between statements (a), (b) and (c) is addressed in the following.

4.3. Remarks on the Damage Tolerance

The obtained results—concerning the structural behavior of the various structural systems listed in Table 1 with respect to the Life Safety Limit State condition—show that damage-tolerant structural solutions (either those of Group 1-A2 and Group 2) are more performant than Group 1-A1 (traditional CLT buildings) in terms of seismic behavior and structural damaging.

In light of this, a Damage Matrix representative of four possible damage levels (from Null to Very High) is represented in a qualitative way in Figure 11. The matrix includes different damage intensities as a function of four Limit States (i.e., Operational, Damage, Life Safety and Collapse) and of the different CLT-based structural solutions analyzed herein, giving an idea of the different damage expected among the various structural solutions. It can be noted that the damage level is gradually decreasing, passing from the solution of Group1-A1 (traditional CLT walls) to Group 1-A2 (damage-tolerant CLT walls) to Group 2 (post-tensioned walls).

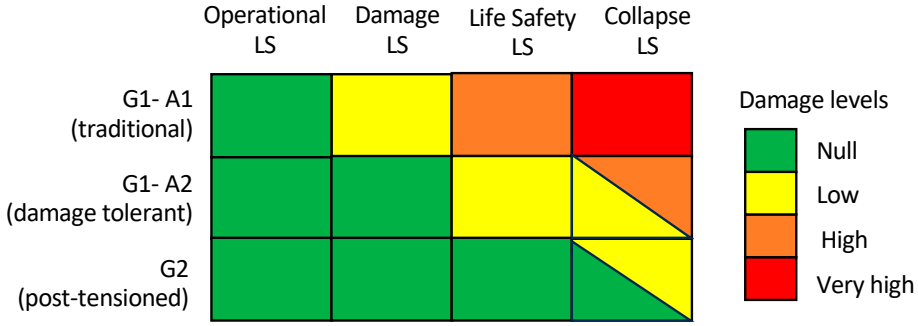


Figure 11. Structural Damage Matrix for different structural solutions of CLT buildings.

Nonetheless, in a general reasoning that involves damaging, LCA and costs analysis, the influence of damage tolerance of the non-structural components on the overall seismic performance of CLT buildings cannot be ignored. In fact, Performance-Based Design (PBD) of buildings requires that the influence of non-structural components—such as infill walls and partitions, ceilings, claddings, contents, etc.—cannot be neglected anymore. Thus, many codes treat the problem of safety related to non-structural components, also including criteria for determining the seismic demand at different Limit States [29,31,46].

Even in the presence of low-damage principal structural systems, damage to non-structural elements has a severe impact on building recovery, increasing the socioeconomic losses even for low-intensity events (i.e., Damage Limit State) [34]. This aspect is not secondary, especially when (i) the displacement capacity and/or the inter-storey drift of the structural system are significant, with the non-structural elements being forced to accommodate such displacements with a high probability of achieving a high level of damage and needing to be removed and replaced with new components and (ii) the number of non-structural elements is elevated.

The latter two statements (i) and (ii) 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 (realized with light-weight timber panels or glass façades, among others). From this perspective, the system of Group 1-A2 appears optimal (also with respect to the case of Group 2) because it limits the damage of structural components and contemporarily requires a lower number of non-structural elements to realize the construction. In light of this, the damage levels indicated in the Damage Matrix of Figure 11 should probably be revised if the contribution of the damage to non-structural components wants to be included in the damage analysis. Currently, no data devoted to computing the influence of non-structural components in damage analyses for timber buildings are available in the literature, and surely future studies should be developed.

Studies on the behavior of non-structural timber-based drywall equipped with low-damage connections are available in the literature [47–49]. Obviously, such systems reduce the nonstructural damage, and the serviceability of the construction is preserved, especially for low-intensity earthquakes. Moreover, the economic and environmental costs of a timber wall structure should consider the probability of a reduced or nullified efficiency of non-structural elements, especially in the presence of a severe earthquake.

As a final remark, it is worth noting that another issue affects the damage level of CLT buildings belonging to Group G1 (traditional G1-A1 or equipped with damage-tolerant connections G1-A2), strictly related to the construction technology of these buildings. The issue regards the damage that arises in the horizontal CLT floors due to the rocking motion of the vertical panels under seismic actions: due to the platform-type constructional technology (i.e., floor inserted between two vertical walls), the rocking movements of the vertical panels cause the permanent crushing of the floors in orthogonal-to-grain direction (Figure 12a) because the mechanical properties of timber in such a direction are very low (orthogonal-to-grain compression strength ranges between 2 and 3 MPa, Young modulus about 300–500 MPa). Thus, it is worth revising the idea behind the traditional CLT building and introducing the '2.0 CLT building' composed of full-height CLT walls (i.e., balloon-type technology) equipped with damage-tolerant and replaceable connections at the base of the walls only (Figure 12b).

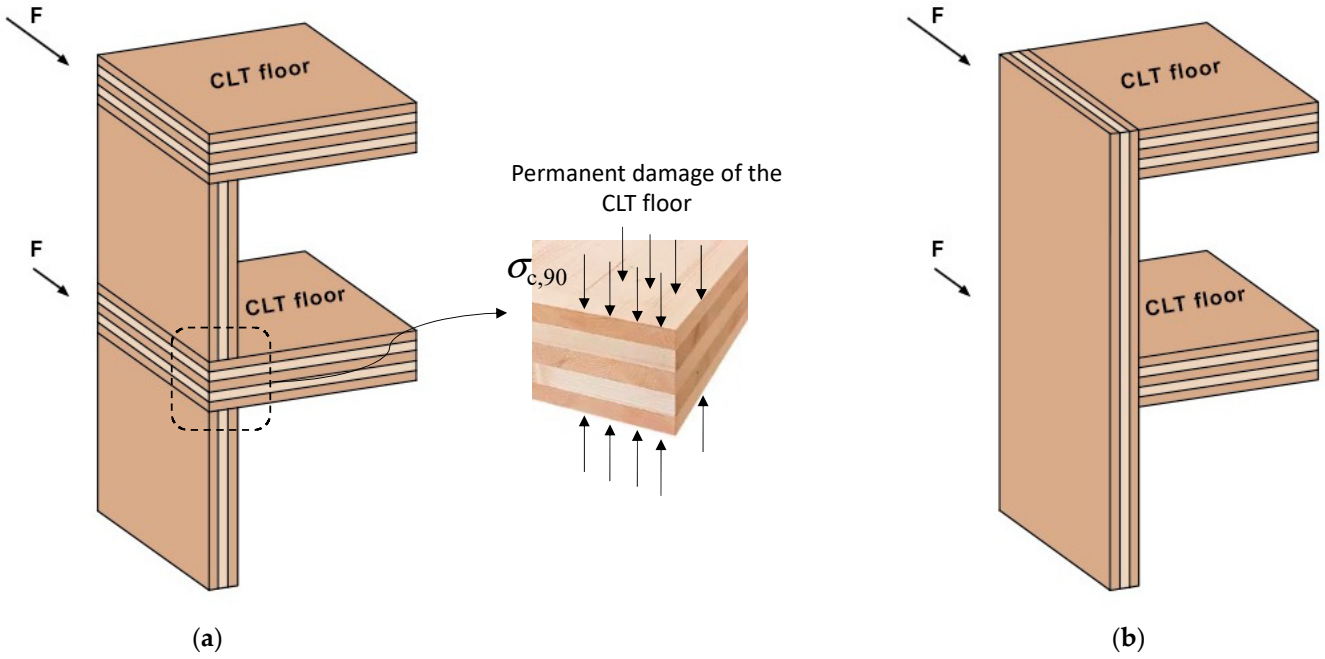


Figure 12. Construction technologies for CLT buildings: (**a**) platform technology, (**b**) balloon technology.

4.4. Construction Costs and Life Cycle Assessment

This section reports some observations on the influence of the costs and LCA aspects related to the different structural schemes and types of mechanical connections investigated in this paper.

A comparison from an economic point of view of the different structural systems of Group G1 and those of Group G2 requires analyzing many factors, concerning design and technological aspects. All the solutions of Group G1 (designed according to Approach A1 or A2) are particularly massive 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. Such solutions require a higher number of walls and larger quantities of material, whereas the technology of Group G2 needs few seismic-resistant walls. 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.

As only the structural components are concerned, the weight of timber in cases of Group G1 resulted in being greater than 30% with respect to Group G2. Obviously, this aspect also has an impact on the construction costs involving transportation and installation costs, material and energy used in the production as well as building phases. For the case study herein analyzed, the impact of the costs of the structural system on the total building costs of Group G1 is about 30%, while in the case of the post-tensioned solution (Group

G2) is about 20%. In Figure 13, the total costs of the solutions relative to Groups G1 and G2, with an indication of the impact of costs of the structural elements, are summarized. Furthermore, despite the fact that costs related to the non-structural component are not directly evaluated in this paper, those attributable to Group G2 are surely greater than those relative to Group G1.

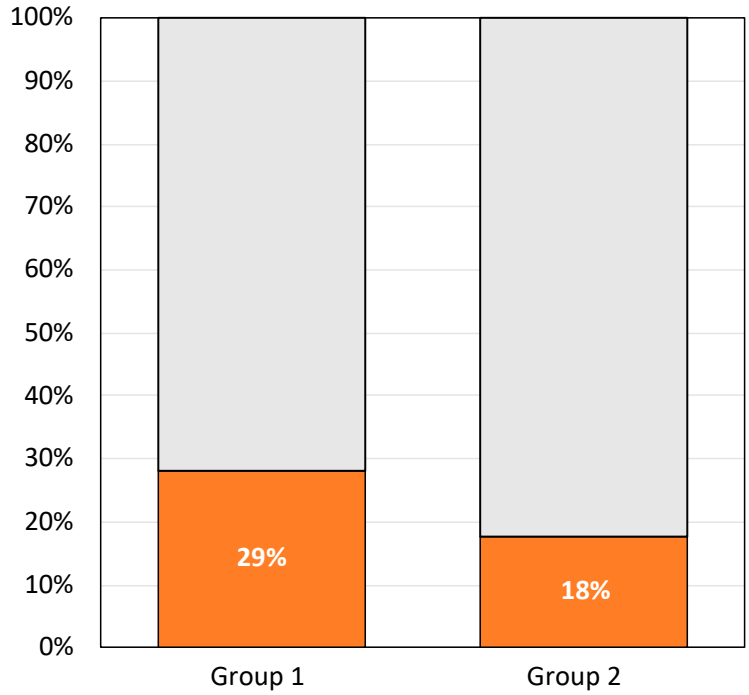


Figure 13. Comparison between construction costs.

As far as the LCA is concerned, it measures the environmental impact associated with all stages of a building's Life Cycle, as provided by the International Organization for Standardization ISO 14040 [50] and the European Standard EN 15978 [51]. LCA consists of five different phases, which should be correctly considered to assess the environmental impact of the construction process: processing and extraction, production, packaging and transports, use and end of life (Figure 12). Depending on the type of material, technological features of the buildings, structural behavior after a seismic event (e.g., damage level) and recyclability of materials, different environmental impacts can be achieved.

In Figure 14, the impact of the different structural solutions on the LCA is represented through red, blue and black circles. The thickness of the circles indicates the relevance of the impact of the structural solutions on each phase of the Life Cycle.

A first consideration concerns the structural solutions of Group G1, where the advantages of Approach A2 with respect to Approach A1 can mainly be referred to as the repairing phase after the seismic event (i.e., use and end of life). In fact, the low damage-based Approach A2, avoiding removal and disposal of damaged CLT walls, reduces demolition and reconstruction costs, material and energy use and pollution due to transport and fabrication. In fact, solutions that provide permanent damage to the connecting zones (e.g., Group 1-A1) require the demolition of the structure involving additional costs. A detailed LCA being out of the scope of this paper, some remarks can be proposed based on the comparative analysis of the design procedures described herein:

- The structural solution of Group G1 involves higher quantities of structural material than that of Group G2, and consequently high environmental impact on the phases of extraction, production, packaging and transport is expected in the first case (red circles).

- 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 (red circles).
- The solutions of Groups 1-A2 and Group G2 have a limited impact (blue circles) on the phases of 'use' and 'end of life' due to limited (or null) damage of the connections, thus limiting the demolition and recycling of material.
- The solution of Group G2 has an impact (which can be also significant) on the phases of 'use' and 'end of life' due to the high amount of non-structural components which can be potentially damaged in the case of earthquakes (black circle).

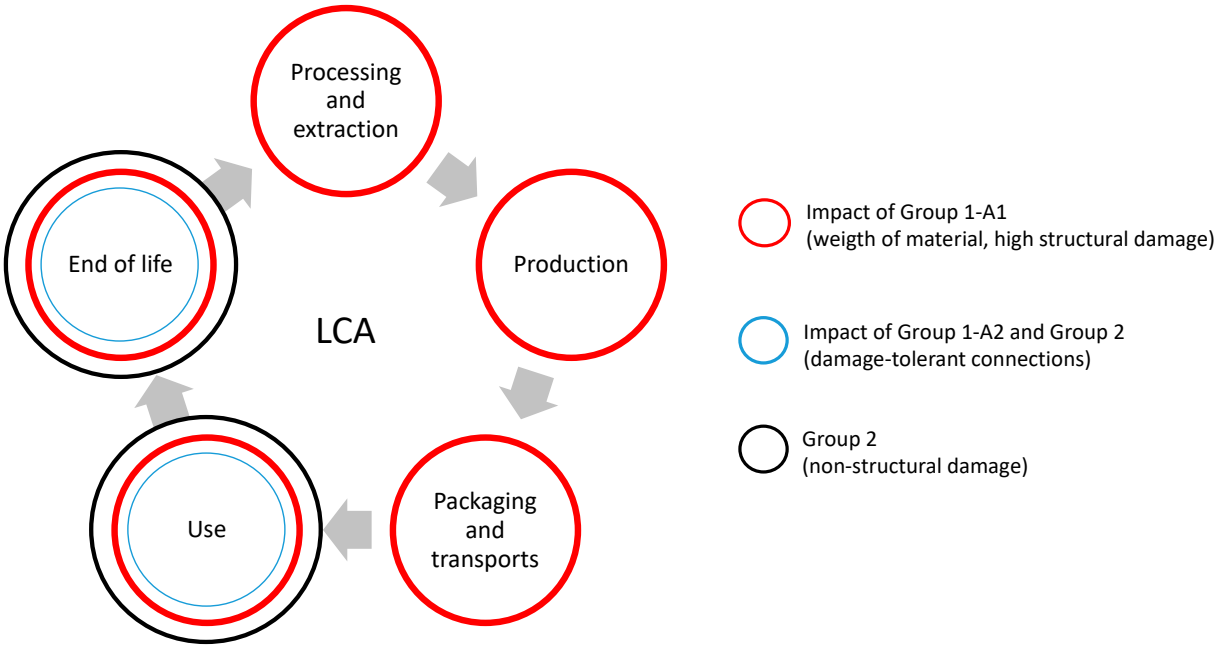


Figure 14. Impact of the different structural solutions on LCA.

As a matter of fact, damage-tolerant solutions represented by Group 1-A2 and Group G2 are both evidently more effective in terms of damage control and sustainability than that of Group 1-A1. Those of Group 1-A2 have the further advantage of reducing the number of non-structural components and then costs related to their eventual damage. Note that both in the cases of Group 1-A2 and Group G2, low-damage non-structural components can also be 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.

5. Concluding Remarks

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 that limit the overall performances of the CLT buildings because they are based on structural considerations only.

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

1. The adoption of damage-tolerant structural systems is a fundamental requisite to achieve high performances of CLT buildings in terms of structural behavior, damage control, costs and environmental impact.
2. Traditional CLT buildings (Group 1-A1) are low-performant because damage is expected in the connecting zones depending on the level of seismic demand. After an earthquake, the structural components must be demolished/repaired and then reconstructed, involving additional costs and environmental impact.
3. Low-damage solutions (Group G2) are efficient from structural, architectural and environmental points 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 expensive for non-structural elements and increase construction costs.
4. 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 the non-structural element is limited.

The comparative analyses described herein are certainly not comprehensive and need further investigations and validations to overcome their limitations. Firstly, 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. Next, studies focusing on the quantification of costs related to the damage to non-structural components and their effect on LCA are lacking a clear identification of the role of the response of components in view of the optimized design approach herein proposed.

Finally, digital BIM-based tools should be developed to implement an automated design procedure including a rational design approach accounting for all the aspects related to damage control, costs and the Life Cycle process.

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