Seismic behaviour of Cross-Laminated Timber structures: a state-of-the-art review

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ABSTRACT

Cross-Laminated Timber (CLT) structures exhibit satisfactory performance under seismic conditions. This is possible because of the high strength-to-weight ratio and in-plane stiffness of the CLT panels, and the connections capacity to resist the loads with ductile deformations and limited impairment of strength. This study summarises a part of the activities conducted by the Working Group 2 of COST Action FP1402, by presenting an in-depth review of the research works that have analysed the seismic behaviour of CLT structural systems. The first part of the paper discusses the outcomes of the testing programmes carried out in the last fifteen years and describes the modelling strategies recommended in the literature. The second part of the paper introduces the *q*-behaviour factor of CLT structures and provides capacity-based principles for their seismic design.

KEYWORDS: Cross-Laminated Timber, seismic performance, experimental testing, Finite-Element numerical modelling, *q*-behaviour factor, capacity-based design.

1. INTRODUCTION

Timber constructions have undergone a revival of popularity over the last years; this positive trend is associated to a combination of several factors. Firstly, wood-based structural products generate fewer pollutants compared to the mineral-based building materials (steel and concrete) because are obtained from sustainable and renewable resources. Secondly, timber structural elements are prefabricated off-site and transported to the building location, where they are quickly assembled. Finally, the strength-

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to-weight ratio of wood is a great advantage for structures erected in seismic-prone areas, because it limits the total mass of the buildings.

The seismic performance of multi-storey timber structures has been the focus of several research projects. Tests firstly examined the behaviour of light-frame buildings, which were the most common timber structural systems all over the world. Results of full-scale shaking table tests showed a highly dissipative behaviour, with most of the plastic deformations concentrated in the sheathing-to-framing joints and the anchoring devices (hold-downs and angle brackets) still in the elastic phase [1-5]. More recently, the increasing interest in high-rise structures (the so-called 'tall buildings') required a higher level of seismic performance. Therefore, the focus has shifted to massive and more effective systems, such as Cross-Laminated Timber (CLT) [6]. Compared to light-frame buildings, CLT structures have a higher in-plane stiffness and a greater load-carrying capacity; differences are attributed to both the physical parameters of the timber panels and the mechanical properties of the connections used (hold-downs and angle brackets, stronger and stiffer than the connectors used in lightweight structures). In particular, full-scale tests of CLT structures highlighted that the CLT panels act almost as rigid bodies, while the connections provide all the ductility and the energy dissipation [7, 8].

CLT structures are generally divided into two groups, depending on their dissipative capacity. The first group refers to buildings assembled using large monolithic walls, i.e. panels with high length-to-height ratios. The second group refers to buildings assembled using segmented walls, i.e. systems of narrow panels fastened together with vertical step joints. In the first case, the energy dissipation takes place only into the anchoring connections used to prevent the rocking (hold-downs) and sliding (angle brackets) of the CLT walls. Therefore, such structures have a low to medium capacity to dissipate the seismic energy. In the second case, if properly designed, the vertical step joints enhance the ductility of the buildings, thus resulting in a high capacity to dissipate the seismic energy.

Nowadays, the use of CLT structural systems in Europe is codified only into the ETAs (European Technical Assessments) issued for the specific building products, whereas design principles have not yet been included either in Eurocode 5 [9] or in Eurocode 8 [10]. General design principles for CLT structures have been included in the Austrian National Annex to Eurocode 5 [11], while similar pieces of information are not yet available for any other European country. Based on the current situation in Europe, the COST Action FP1402 'Basis of Structural Timber Design - from research to standards' was established in 2014, as part of the initiatives dedicated to the development of the new Eurocodes.

This paper summarises a part of the activities conducted by the Working Group 2 of COST Action FP1402; it presents a state-of-the-art review of the studies that focused on the seismic performance of CLT structures and recommends principles for the design practice. The first section discusses the outcomes of the experimental testing programmes that have examined the seismic behaviour of CLT structural systems. The second section shifts the focus to the modelling approaches recommended in the literature to predict the seismic performance of CLT structures. The third section introduces the *q*-behaviour factor (denoted as the 'seismic reduction factor' in some structural design codes) of CLT buildings, necessary in the seismic design to scale down the elastic response spectrum to the design spectrum. Finally, the fourth section proposes capacity-based design principles for CLT structures. Results of past testing programmes are used as a basis to develop provisions capable of ensuring that all plastic deformations occur in selected ductile components and no other part (less ductile or brittle) exhibits any anticipated failure.

2. TESTING OF CLT STRUCTURES

The seismic performance of CLT buildings has been the central topic of several testing programmes. The experiments have been carried out on single connections, monolithic and segmented wall systems (i.e. CLT walls and connections), and full-scale buildings featuring different numbers of storeys and layups. In this section, the outcomes of the testing programmes are discussed; further information is also available in Pei *et al.* [12].

2.1 Testing of connections

Mechanical connections used in CLT buildings are typically divided into two groups. The first group refers to the connections used to prevent the rocking and sliding of the walls, i.e. the hold-downs and the angle brackets. Such metal connectors are fastened to the CLT walls using threaded nails or screws with a small diameter, and have been developed based on the connection systems used in light-frame structures [13]. The second group refers to the step joints used to prevent the relative sliding between contiguous walls or between a floor panel and the underlying wall. Those joints are usually assembled using self-tapping screws made of carbon steel, with partially or fully threaded shank [14].

The hysteretic behaviour of the connections with hold-downs and angle brackets has been the focus of several research projects. Gavric *et al.* [15] and Flatscher *et al.* [16] carried out the most complete testing programmes as part of the SOFIE and SERIES Projects, respectively. Results highlighted a dissipative behaviour and ductile failure mechanisms, with the only exception of the situations where the angle brackets, designed to resist primarily in shear, were loaded in tension. In such situations, they exhibited some inappropriate failures caused by either withdrawal of the nails from the floor panels or pull-through of the anchoring bolts (Figures 1a-1b). However, those connectors proved to have good mechanical properties under lateral and axial loads. Conversely, the hold-downs showed high strength capacities when loaded in tension and a weak mechanical behaviour if subjected to lateral loads, due to the buckling of the metal flanges. Furthermore, tests of connections with hold-downs conducted by Tomasi and Sartori [17] pointed out two additional failure mechanisms that may occur if high tension loads are transferred along the connections, i.e. tensile failure in the net cross-section of the metal flange and buckling of the anchoring to the foundations (Figures 1c-1d).

Shear tests of panel-to-panel joints, performed by Gavric *et al.* [18] on half-lapped and spline joints with partially threaded screws, led to a good hysteretic behaviour. However, some brittle mechanisms occurred in cases where the requirements for end and edge distances were not satisfied. More recently, Hossain *et al.* [19] conducted similar tests on panel-to-panel joints with double-angled fully threaded screws. Results showed significantly higher strength and stiffness capacities than those obtained with partially threaded screws, although the loads transferred along the joints caused some brittle failures with splitting of the timber members.

The experimental activities finalised at the cyclic characterisation of the connections used in CLT structures are still ongoing, with special attention to the applications in mid- and high-rise buildings. Tests have been carried out on a large number of connections, by varying the thickness and geometry of the connectors [20-23]. Furthermore, several hold-downs [24], angle brackets [25] and screws [26] have been examined by considering the simultaneous presence of lateral and axial loads. This proved that the coupled shear-tension action influences their mechanical properties and dissipative capacity. Lastly, great effort has been devoted to investigating the performance of some innovative connection systems. Polastri *et al.* [27] analysed the hysteretic behaviour of X-RAD connectors, which showed great potentials to resist the coupled shear and tension loads with large ductility ratios. Loo *et al.* [28] developed a slip-friction connector, composed by a plate of abrasion resistant steel and two plates of mild steel between which it slides. Kramer *et al.* [29] proposed an energy dissipation system for self-centring wall systems, based on the concept of the steel buckling-restrained braces and composed of a milled element designed to yield and a steel pipe in which it is enclosed. Similarly, Sarti *et al.* [30] investigated the performance of a replaceable dissipater, composed of a mild steel bar confined by a steel tube filled with grout or epoxy. Finally, Hashemi *et al.* [31] introduced a slip-friction connector that allows for the self-centring of the wall without requiring the adoption of post-tensioned tendons. Compared to traditional connections with hold-downs and angle brackets, these systems attain large ductility ratios while limiting the residual drift and peak accelerations.

2.2 Testing of wall systems

Racking tests of monolithic and segmented wall systems (i.e. CLT walls composed of narrow panels, fastened together with vertical step joints) further explored the hysteretic behaviour of CLT structural systems. For this purpose, several testing programmes have been conducted in Europe [7, 8, 32-34], Canada [20] and Japan [35, 36].

In Europe, Gavric *et al.* [32] carried out cyclic racking tests using monolithic and segmented walls. In the first case, tests considered a square wall and the layout of the anchoring connections was varied;

in the second case, the wall was composed of two narrow panels and tests investigated the influence of the screws in the vertical joint. Hummel *et al.* [33] conducted similar racking tests to those reported above and extended the investigations to walls with an opening. Dujic *et al.* [34] examined the racking behaviour paying particular attention to the effects of the boundary conditions; three situations were investigated: shear cantilever mechanism (rocking response), restricted rocking mechanism (coupled shear-rocking response) and pure shear mechanism. Finally, Hristovski *et al.* [7, 8] performed shaking table tests on monolithic and segmented wall systems; compared to the investigations reported above, which were carried out under quasi-static loading conditions, the shaking table tests were performed under dynamic conditions and provided a detailed insight into the seismic performance of the systems.

In Canada, Popovski *et al.* [20] investigated the racking behaviour of monolithic walls with three aspect ratios, segmented walls with vertical step joints and different layouts of screws, and two-storey assemblies. The experiments used commercially sold and custom-made angle brackets with different fastening systems, a combination of angle brackets and hold-downs, and inclined screws. Conversely, in Japan, Okabe *et al.* [35] and Yasumura [36] examined the racking behaviour of walls made of the local grown Sugi (*Cryptomeria japonica*) rather than the typical European species (e.g. *Picea abies*). Tests adopted monolithic walls with a narrow CLT panel and segmented walls with up to three narrow panels, fastened together with vertical step joints.

Finally, following the increasing use of CLT for the construction of mid- and high-rise structures, several research efforts have been devoted to analysing the racking behaviour of walls equipped with the innovative connections discussed in Section 2.2 at the anchoring to the foundations [37-39] and with energy dissipating U-shaped flexural plates in the vertical joints [40, 41].

All testing programmes confirmed that the layout of the connections (in terms of type, number and position) governs the cyclic behaviour of a CLT wall system. Results highlighted that the CLT panels exhibit minor in-plane deformations and act almost as rigid bodies, while the connections provide all the ductility and the energy dissipation. Furthermore, racking tests of segmented walls demonstrated that the aspect ratio of the panels and the number of screws in the vertical step joints influence the global kinematic behaviour.

Finally, Dujic *et al.* [42] and Yasumura *et al.* [43] investigated the racking performance of walls with an opening. In particular, Dujic *et al.* [42] concluded that openings with up to 30% of the surface do not affect the maximum load-carrying capacity, although they reduce the stiffness up to 50% of the value of a wall panel without apertures. More recently, Shahnewaz *et al.* [44] further explored the behaviour of wall systems with openings and proposed some equations capable of predicting their stiffness when the size and aspect ratio of the openings, as well as aspect ratio of the wall are varied.

2.3 Testing of full-scale structures

Following the extensive research efforts on connections and wall systems, full-scale tests were also performed on single and multi-storey CLT structures. The investigations have firstly been carried out on a one-, three- and seven-storey buildings as part of the SOFIE Project [45-47]; all structures were erected using narrow CLT panels, anchoring connections with hold-downs and angle brackets, and vertical step joints with partially threaded screws.

Lauriola and Sandhaas [45] conducted pseudo-dynamic lateral tests on the one-storey building and Ceccotti and Follesa [46] performed full-scale shaking table tests on the three-storey structure. Tests investigated the lateral deformability capacity caused by the presence of large openings at the ground floor. In particular, the experiments considered two symmetrical configurations parallel to the loading direction (with different layouts of openings) and a third one asymmetrical. Neither of the structures underwent any major damage in the CLT members; furthermore, because the deformability capacity was governed by the rocking of the single walls, tests exhibited lateral deflections proportional to the area of the openings.

Ceccotti *et al.* [47] carried out the shaking table tests on the seven-storey building, assembled using CLT elements and connection systems similar to those used in the one- and three-storey structures. However, because the hold-downs used for the three-storey building were not suitable for high slender structures, special high-strength hold-downs with 10 mm thick plates were placed at the anchoring to the foundations. After several tests, carried out by varying the ground motion record, the building did not collapse and exhibited only local damages close to the connections.

Flatscher and Schickhofer [48] conducted full-scale shaking table tests on a three-storey building, as part of the SERIES Project. Compared to the three-storey structure tested by Ceccotti and Follesa [46], the building was assembled using large monolithic walls rather than narrow panels with vertical step joints. Furthermore, fully threaded screws were primarily used as fasteners, rather than partially threaded screws. Consequently, the lateral deformability of the building was lower than the structures tested within the SOFIE Project, thus resulting in smaller inter-storey drifts.

Popovski and Gavric [49] tested a two-storey structure under quasi-static loading conditions, with specific attention to the lateral strength and deformability capacities. Tests did not exhibit any global instability even after the attainment of the maximum load-carrying capacity; furthermore, only minor torsional effects were observed and the ultimate resistance was identical in both principal directions.

Finally, in Japan, several shaking table tests have been carried out on multi-storey CLT structures. Tsuchimoto *et al.* [50] focused on the static lateral capacity and seismic performance of a three-storey structure assembled with narrow CLT panels. Compared to the structures previously tested in Europe and Canada, the building was assembled using tension bolts and screwed steel-to-timber connections rather than with hold-downs and angle brackets. Kawai *et al.* [51] investigated the dynamic behaviour of a five-storey structure assembled using similar connections to those used by Tsuchimoto *et al.* [50]. Furthermore, Kawai *et al.* [51] extended the analyses to a three-storey structure where the CLT panels were used as outside walls and solid timber frames were used in the inside. Finally, Yasumura *et al.* [52] tested two two-storey structures composed of monolithic and narrow CLT panels, respectively. In the structure with monolithic wall panels, some cracks were observed at the corner of the openings, which propagated both vertically along the grain of the panel surface and diagonally as the horizontal displacement increased. In the structure assembled with narrow wall panels, some gaps were observed between each wall and no cracks were visible at the corner of the openings; additionally, small gaps were observed at the floor joints above the openings and bending failure of the CLT floor panel was detected above the corner of the openings.

3. MODELLING OF CLT STRUCTURES

In recent years, thanks to the technological advancements of Finite-Element (FE) software packages, numerical modelling has become an important tool in both the research field and the design practice. Usually, the planning and execution of full-scale tests is an expensive and time-consuming procedure, requiring consideration of several factors (e.g. thickness and aspect ratio of the timber panels, as well as type and position of the connections) and loading conditions. Furthermore, the simplified formulas used by practitioners are not always capable of predicting the non-linear response of a CLT building, and the mutual interaction between the ground motion and the overlying structure. To overcome those limitations, several numerical models have been proposed; however, most of them are advanced tools for research purposes and only few are used in the design practice, due to the complex implementation procedures and the need of input parameters for which limited evidences exist (e.g. friction).

Numerical models of CLT buildings employ some well-established assumptions: the timber panels are assumed to behave elastically and the non-linear response of the structure is concentrated in the connections. Furthermore, to improve the reliability of the numerical predictions, the panel-to-panel interaction (contact and friction) shall be properly taken into account and modelled in the analyses.

3.1 CLT members

CLT wall panels are generally simulated in FE analyses according to one of the following approaches. The first approach schematises a wall as a set of trusses; the in-plane deformations are either ignored [53, 54] or modelled with diagonal springs [55, 56]. This technique limits the computational effort to the minimum, although it has two major drawbacks: it does not provide a punctual representation of the stress distribution in the panels and the connections are localised on the corners of the wall rather than modelled on their actual position. The second approach schematises a wall panel using 2D shells; the layup of the CLT members is taken into account using either multi-layer shells or according to an equivalent orthotropic approach [57, 58] (typically the Blaß-Fellmoser composite theory [59]). Shell models overcome the limitations of the truss schematisation and are the preferred technique in both the research field and the design practice. Their use is particularly advantageous when the panel has an opening because they allow predicting the stress distribution in the CLT wall, preventing undesired brittle failures. Finally, the third approach schematises a wall panel using 3D solid elements [60]; this technique is a further development of the multi-layer shell method and is generally the most accurate, because it allows accounting for the actual thickness and orientation of each board layer. However, solid models usually require a high computational effort and have had limited applications until now.

As mentioned in the previous paragraph, the modelling of panels with an opening requires specific attentions. Generally, the door and window openings are obtained either by the cutting of the panels or by the assemblage of multiple elements (Figure 2). In the first situation, it is feasible to model the entire system as a unique element and to maintain the same layup of the CLT walls even in the lintels. In the second case, the lintels shall be modelled as independent elements and the fastening to the CLT walls shall be schematised using link elements. Furthermore, in such a situation, the lintels orientation shall be properly schematised to ensure a correct representation of the physical system (depending on the fact that it may be either another CLT member or a solid wood element).

Finally, the floor panels are modelled using similar methods to those discussed above. Typically, truss models with rigid links are used if the floors act as diaphragms, while 2D shell models are used when the out-of-plane behaviour of the panels is taken into account in the analyses. In this context, a recent study conducted by Moroder *et al.* [61] highlighted that the stiffness of the floors may influence the dynamic behaviour of a multi-storey timber structure; therefore, in the authors opinion, it is always recommended to model the CLT floor using shell elements to obtain reliable numerical predictions.

The material parameters of CLT are necessary input parameters in FE analyses. Unlike solid timber and glued-laminated timber, those properties have not yet been harmonised in any European standard. Some reference values for CLT are prescribed in the Canadian CSA O86-14 [62]. However, if specific test data for the layup analysed are not available, it is recommended to determine those parameters in accordance with Brandner *et al.* [6].

3.2 Mechanical connections

Mechanical connections are modelled in FE analyses using link elements or springs. Their behaviour is typically defined according to one of the following methods. The first method considers a uniaxial behaviour where each connection resists only in its primary direction: the hold-downs in tension and the angle brackets in shear [52, 53, 63]. This simplification is commonly adopted by practitioners and leads to conservative results, because it neglects the stabilising contribution of angle brackets in their axial (weakest) direction [60]. The second method assumes a biaxial behaviour where the connections resist both axial and lateral loads simultaneously, and each component is independent [7, 58, 64, 65]. Compared to the first situation, the axial contribution of angle brackets is introduced into the analysis, improving the accuracy of the results. Finally, the third method is adopted in time-history simulations and considers a biaxial behaviour with an interaction domain between axial and lateral loads [66, 67]. In this context, simulations of connections with hold-downs and angle brackets exhibited a quadratic interaction relationship between shear and tension [68].

The constitutive law implemented in the springs depends on the analysis performed. Typically, an elastic response is considered in linear analyses, while multi-linear relationships are used in pushover simulations [7, 52, 64, 69]. In the first case, the dissipative behaviour of the connections is taken into account using the q-behaviour factor. Furthermore, the mechanical properties of the connections are defined as follows: the elastic stiffness is acquired from test data and design values of the maximum load-carrying capacity are determined either in accordance with Eurocode 5 [9] or based on the ETA of the connections. In the second case, the load-displacement laws used as inputs in the analyses are assessed either from the loading curves of monotonic tests or from the envelope curves of cyclic tests. Finally, time-history simulations adopt advanced non-linear relationships, which allow for a detailed schematisation of both the hysteretic behaviour (pinching effect) and the impairment of mechanical properties due to cyclic loading [53, 65-67, 70]. Those relationships require the assessment of many input parameters and are calibrated using experimental data obtained under cyclic conditions.

As mentioned at the beginning of this section, practitioners usually perform simplified simulations to predict the performance of CLT structures. Consequently, linear analyses with the design response spectrum represent the preferred approach in the practice and sometimes are the option prescribed by the standards. In such a situation, it is necessary to pay particular attention to the hold-downs. Due to the contact between the wall and the underlying element (the foundations or an intermediate floor), the hold-downs resist only tension. Consequently, they have a non-linear constitutive law even in the elastic phase, with stiffness under compressive loads much higher than the corresponding value under tension loads. This issue has been extensively addressed in the literature and several approaches have been developed, which rely on iterative procedures [56] or on the assessment of equivalent stiffness values [58].

3.3 Panel-to-panel interaction

The panel-to-panel interaction influences the dynamic behaviour of a CLT structure. As demonstrated via non-linear dynamic simulations of full-scale CLT buildings, the friction at the bottom edge of the walls reduces significantly the inter-storey drifts [57, 58]. Nevertheless, for the sake of simplicity, its effect is usually ignored in the design practice, i.e. when force-based design methods are used.

Depending on the modelling strategy adopted, the friction at the base of the CLT walls is simulated using either gap elements or a surface-to-surface interaction. Generally, the first approach is preferred if truss models or shell models are used [57], while the second approach is adopted in the presence of solid models [60]. Furthermore, the friction in the vertical joints is usually ignored; this simplification is acceptable even under dynamic conditions and does not introduce any evident error in the results.

4. BEHAVIOUR FACTOR OF CLT STRUCTURES

The *q*-behaviour factor is used to perform the seismic design of a building by means of linear analyses with the response spectrum. To this aim, the *q*-factor is necessary to scale down the elastic response spectrum to the design spectrum; it accounts for the non-linear behaviour of the structure, the presence of damping and of any other force-reducing effect.

4.1 Scientific background

The assessment of the *q*-behaviour factor of a CLT structure has been a central topic of many research projects. The investigations have been carried out using both experimental and numerical approaches, and different analysis methods have been proposed. In this context, due to the high costs of testing, numerical methods have played a key role in the assessment of the *q*-factor for CLT structures. Their advantage relies on the possibility of investigating the *q*-behaviour factor by varying the geometry of the buildings (number of storeys, plan dimensions, aspect ratio of the CLT members, and properties of the connections), the applied loads, the regularity in elevation, and the ground motion record.

Among the experimental approaches, the most used method to assess the *q*-behaviour factor of a CLT structure uses the results of full-scale shaking table tests. In particular, the *q*-factor is defined as the ratio of the Peak Ground Acceleration (PGA) at which the near-collapse status is reached to the PGA with which the building was designed elastically [47, 71].

Conversely, when numerical approaches are used, the *q*-factor is assessed using the results of non-linear simulations carried out under static or dynamic loading conditions [53-55, 72-75]. The models are developed using the static ductility and hysteresis cycles resulting from tests of single components (wall systems and mechanical connections). If non-linear static analyses are used, the FE models are subjected to constant vertical loads and the horizontal loads are increased monotonically. The *q*-factor is then evaluated using the base shear versus displacement curve, following the Newmark [76] or the N2 [77, 78] methods. If non-linear dynamic analyses are used, the models are subjected to different accelerograms that simulate the ground motion and the *q*-factor is assessed following either the Peak Ground Acceleration (PGA) or the Base Shear (BS) methods. In the first situation (PGA method), the *q*-factor is defined as the ratio of the PGAs at the yielding and ultimate displacements of the structure, respectively. In the second situation (BS method), the q-factor is defined as the ratio of the base shear at the yielding and ultimate displacements of the structure, respectively.

As shown in Table 1, the highest q-factors are obtained when segmented walls composed of narrow panels and vertical step joints are considered and the lower values are obtained when monolithic walls are adopted. Differences are due to the enhanced energy dissipation in the vertical step joints, which occurs when fasteners with a small diameter are used. Furthermore, Trutalli and Pozza [75] showed that the regularity in elevation influences the q-factor, reducing its value up to 25%.

4.2 Code prescriptions

The current version of Eurocode 8 [10] does not prescribe any specific q-factor for CLT structures; a q-factor equal to 2.0 is prescribed for buildings erected with glued walls and diaphragms that comply with the criterion of regularity in elevation. However, this building typology cannot be intended as a CLT structural system, because the investigations on the seismic performance of panelised buildings have been conducted after the publication of the standard.

Recently, Follesa *et al.* [79] proposed a revised version of Chapter 8 of Eurocode 8 [10] in which CLT buildings are divided into three classes: the first two classes apply if the design is performed in accordance with the principles of the capacity approach (DCM and DCH), while the third class is used when a non-dissipative behaviour is assumed (DCL). The DCM class (i.e. medium capacity to dissipate energy) applies to buildings assembled with monolithic walls without vertical joints, while the DCH class (i.e. high capacity to dissipate energy) applies to structures assembled using segmented walls with vertical step joints. In the first case, a q-factor equal to 2.0 is recommended; in the second case, a q-factor equal to 3.0 is adopted. Finally, if the principles of the capacity-based design are not satisfied, the DCL class (i.e. low capacity to dissipate energy) prescribes a q-factor equal to 1.5.

Specific provisions for the seismic design of CLT buildings have been included in the Canadian CSA O86-14 [62]. In this standard, the q-behaviour factor corresponds to the product of a ductility-related force modification coefficient R_d by an overstrength-related force modification coefficient R_0.

In particular, if the energy is dissipated through connections and wall panels with length-to-height ratio lower than one that act in rocking or in a combination of rocking and sliding, R_d is equal to 2.0 and R_0 is equal to 1.5 (i.e. $R_d R_0 = 3.0$). Furthermore, the standard prescribes a $R_d R_0 \leq 1.3$ for structures with walls that resist in sliding or composed of panels with length-to-height ratio higher than one. In this context, the Canadian standard and the proposal of Follesa *et al.* [79] define the same coefficient for segmented walls, i.e. $q = R_d R_0 = 3.0$. Conversely, a different behaviour is adopted when long walls are considered, because the CSA O86-14 [62] prescribes a low ductility class.

5. DESIGN PRINCIPLES OF CLT STRUCTURES

The development of advanced analytical models capable of predicting the mechanical behaviour and failure mechanisms of laterally loaded wall systems has been the focus of several research efforts. In particular, Casagrande *et al.* proposed a rheological model to predict the elastic performance of single and multiple shear-walls [56], horizontally-aligned and connected with rigid beams, and an analytical approach for segmented wall systems [80]. Reynolds *et al.* [81] published a yield criterion for laterally loaded shear-walls based on acceptable permanent deformations. Tamagnone *et al.* [82] presented a non-linear procedure to design the anchoring connections of typical CLT walls. Finally, Flatscher and Schickhofer [83] developed a displacement-based model to predict the lateral performance of single and segmented CLT wall systems.

In spite of the significant findings obtained in the research field, some specific rules for the seismic design of CLT structures have not yet been included in Eurocode 8 [10]. In this context, the Canadian CSA O86-14 [62] is the first structural code to introduce instructions for the seismic design of CLT buildings.

Nowadays, it is common practice to verify a structural element using capacity-based principles. This approach, originally developed during the 1970s by Paulay [84, 85], is a necessary condition to ensure a global ductile behaviour consistent with the *q*-factor assumed in the design phase. Therefore, it is a part of the force-based method [86] and should not be considered an advanced design procedure.

5.1 Scientific background

The capacity-based approach aims at ensuring ductile failure mechanisms at different structural levels and at preventing undesired brittle collapses. As discussed in Section 2, the seismic behaviour of CLT structures predominantly depends on the performance of the connections, while the timber panels act almost as rigid bodies. This means that, under dynamic conditions, the dissipative connections shall withstand large deformations and provide a stable energy dissipation [69, 87, 88].

At connection level, it is common practice to assume as dissipative components the laterally loaded joints with dowel-type fasteners (e.g. the timber-to-timber and steel-to-timber joints). To this aim, the yielding of the fasteners shall be achieved with at least one plastic hinge, although the mechanism where two hinges are formed is considered as the most desirable. All failure mechanisms localised outside the ductile joints shall be avoided, i.e. those that might occur in the connections metal member (e.g. tensile failure of the net cross-section), at the anchoring bolts to the foundations and in the timber panels (e.g. splitting or plug-shear). In this context, the dowel-type fasteners located in the dissipative zones shall be inserted in perpendicular to the direction of the load being transferred along the joint (Figures 3a-3b), while all connections made of dowel-type fasteners transferring most of the load via axial resistance shall not be considered as dissipative (Figures 3c-3d).

Once inappropriate failures at connection level are prevented, similar provisions are applied at the wall level. Here, the wall is designed for the overstrength of the connections to avoid any local failure that may occur in the CLT member. Hence, it is important to distinguish between dissipative and non-dissipative connections (Figure 4). The first ones are located in the connections against rocking (hold-downs) and sliding (angle brackets), and in the vertical joints between adjacent panels, respectively. The latter ones ensure the stability of the structure, and are located at the floor level (in the floor-to-floor and in the floor-to-wall connections) and in the vertical step joints between perpendicular walls, respectively.

Finally, at the building level, the floor panels shall act as rigid diaphragms, by ensuring a box-type behaviour and by redistributing the torsion between the walls. Furthermore, to prevent any soft-storey mechanism and to distribute the energy dissipation along the height of a building, the lateral resistance of the shear-walls shall be higher at lower storeys and decrease at higher storeys.

5.2 Analytical provisions

According to the concept of capacity-based design discussed in Section 5.1, the energy dissipation shall be localised in selected dissipative zones; all other structural elements shall be designed with an adequate overstrength to behave elastically. Consequently, ductile failures are achieved thanks to the hierarchy of resistance between the structural components, defined as follows:

$$\gamma_{\mathrm{Rd}} R_{\mathrm{d,ductile}} \leq R_{\mathrm{d,brittle}} \tag{1}$$

In Equation 1, γ_{Rd} is the overstrength factor, while $R_{\mathrm{d,ductile}}$ and $R_{\mathrm{d,brittle}}$ are the design strengths of the ductile and brittle components, respectively. The coefficient γ_{Rd} takes into account all the factors that may increase the strength of a ductile element (e.g. higher-than-specified material strength, strain hardening at large deformations and commercial sections larger than what resulting from the design) and ensures that all non-dissipative components activate after the dissipative ones.

Capacity-based principles for timber structures were firstly discussed by Jorissen and Fragiacomo [89], and have been the focus of a large body of research afterwards; in particular, in agreement with Equation 1, those authors have defined the overstrength factor as shown in Equation 2.

$$\gamma_{\mathrm{Rd}} = \frac{R_{95\%}}{R_{\mathrm{d}}} = \frac{R_{95\%}}{R_{5\%}} \cdot \frac{R_{5\%}}{R_{\mathrm{k}}} \cdot \frac{R_{\mathrm{k}}}{R_{\mathrm{d}}} = \gamma_{\mathrm{sc}} \cdot \gamma_{\mathrm{an}} \cdot \gamma_{\mathrm{M}} \tag{2}$$

In the equation above, $R_{5\%}$ and $R_{95\%}$ are the 5^{th} and 95^{th} percentiles of the experimental strength capacity of the ductile component (from monotonic tests), while R_{k} and R_{d} are the characteristic and the design strength values of the same element, determined with analytical methods (e.g. the European Yielding Model), respectively.

According to Equation 2, three coefficients are identified. The coefficient γ_{sc}, equal to the ratio of $R_{95\%}$ to $R_{5\%}$, accounts for the scatter of strength properties in the experimental tests. The coefficient γ_{an}, defined as the $R_{5\%}$ to R_{k} ratio, measures the accuracy of the analytical model to predict the strength property. Finally, γ_M is the partial safety factor for material properties, equal to one in Eurocode 8 [10] for ductile elements designed in accordance with the concept of dissipative behaviour.

Schick *et al.* [90] and Vogt *et al.* [91] have developed a slightly different approach to evaluate the overstrength factor. Relying on experimental data of light-frame shear-walls, they have defined the overstrength factor as the product of three contributions:

$$\gamma_{\mathrm{Rd}} = \frac{R_{\mathrm{exp,95\%}}}{R_{\mathrm{k}}} = \frac{R_{\mathrm{m}}}{R_{\mathrm{k}}} \cdot \frac{R_{\mathrm{exp,m}}}{R_{\mathrm{m}}} \cdot \frac{R_{\mathrm{exp,95\%}}}{R_{\mathrm{exp,m}}} \tag{3}$$

In Equation 3, R_k is the characteristic value according to code provisions, R_m is the analytical value of resistance calculated with the mean values of the material properties (rather than the characteristic ones), $R_{exp,m}$ is the average strength capacity assessed from the tests, and $R_{exp,95\%}$ is the 95th percentile of the experimental strength capacity. Formally, there are some differences between Equations 2 and 3; however, a closer look reveals that those equations differ only on how the coefficients are defined.

Equation 2 clearly highlights that two situations should be considered, depending on the method used to assess R_k. When R_k is determined based on general rules (e.g. those prescribed in Eurocode 5 [9]), the simplifications introduced in the formulas might underestimate the strength capacity of the ductile component, compromising the hierarchy of resistance planned by the designer. Consequently, γ_{Rd} shall be defined considering the contribution of both γ_{an} and γ_{sc} [89, 92, 93]. Conversely, when R_k is defined based on experimental results or using distinct design rules (e.g. those given in the ETAs), γ_{an} is assumed equal to one and Equation 2 leads to $\gamma_{Rd} = \gamma_{sc}$ [15, 18, 64, 94]. Finally, if no test results are available, another option to assess γ_{Rd} relies on the use of structural reliability methods (e.g. the Monte Carlo simulations [95]).

Recently, Follesa *et al.* [79] proposed a revised version of Chapter 8 of Eurocode 8 [10] where specific design provision for CLT structures in seismic areas are introduced. In particular, a structural element designed in accordance with the concept of dissipative behaviour is verified at the Ultimate Limit State if:

$$E_{\mathrm{d}} \leq \beta_{\mathrm{sd}} \cdot R_{\mathrm{d,ductile}} \tag{4}$$

In the equation above, E_d denotes the design value of the action effects, β_{sd} is a reduction factor that considers the impairment of strength due to cyclic loading and $R_{d,ductile}$ is the design strength of the dissipative element. According to the same authors, β_{sd} is equal to 0.8 for all dissipative systems and higher values may be used if the actual strength degradation is derived from test data.

Once the dissipative elements are verified at Ultimate Limit State, ductile failure mechanisms are ensured by designing the strength of the brittle part $R_{d,brittle}$ as follows:

$$\frac{\gamma_{Rd}}{\beta_{sd}} \cdot R_{d,ductile} \leq R_{d,brittle} \tag{5}$$

Regarding the overstrength factor to be used in Equation 5, Follesa *et al.* [79] recommend a value of 1.3. This coefficient is consistent with the outcomes of the experimental studies conducted during the last years and is obtained by ignoring the contribution of γ_{an}. According to Table 2, values of γ_{an} may vary between 1.0 and 1.4. However, the results included in the table are obtained by considering single joints or connections, and are not necessarily valid for a structural system. In particular, a CLT wall system is usually equipped with several connections and each metal connector is fastened to the CLT wall using many nails (or screws) that carry the load simultaneously. Consequently, it is feasible to assume that both γ_{sc} and γ_{an} may be lower and, according to the authors opinion, this could lead to an overstrength factor close to the one discussed above.

6. SUMMARY AND OUTLOOK

This paper presents a state-of-the-art review of the most important research studies that have focused on the seismic performance of CLT structural systems. The discussion has considered four principal aspects: the experimental testing, the numerical modelling, the assessment of the *q*-behaviour factor, and the seismic design. An in-depth comparison of the proposals made by different research groups has been presented; furthermore, an overview on the prescriptions currently included in the structural design codes has been reported.

Despite of the significant findings on the seismic performance of CLT structures, future research efforts are required to extend the knowledge of this building system. Some specific aspects that have been identified from this research work are listed below:

- the role of CLT diaphragms in multi-storey structures and how they affect the behaviour of the lateral load-resisting systems shall be further examined via testing and numerical modelling;
- great efforts shall be devoted to developing simplified numerical methods for design purposes, capable of predicting the dynamic behaviour of CLT structures;
- the concept of the capacity-based design shall be extended to lateral load-resisting systems and to full-scale structures, by defining specific formulas and procedures for the design practice.

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REFERENCES

1. Filiatrault A, Christovasilis IP, Wanitkorkul A, Van de Lindt JW (2010) Experimental seismic response of a full-scale light-frame wood building. *Journal of Structural Engineering*, **136**(3): 246-254, doi: 10.1061/(ASCE)ST.1943-541X.0000112.
2. Van de Lindt JW, Pei S, Pryor SE, Shimizu H, Isoda H (2010) Experimental seismic response of a full-scale six-story light-frame wood building. *Journal of Structural Engineering*, **136**(10): 1262-1272, doi: 10.1061/(ASCE)ST.1943-541X.0000222.
3. Tomasi R, Sartori T, Casagrande D, Piazza M (2015) Shaking table testing of a full-scale prefabricated three-story timber-frame building. *Journal of Earthquake Engineering*, **19**(3): 505-534, doi: 10.1080/13632469.2014.974291.
4. Tomasi R, Casagrande D, Grossi P, Sartori T (2015) Shaking table tests on a three-storey timber building. *Proceedings of the ICE - Structures and Buildings*, **168**(11): 853-867, doi: 10.1680/jstbu.14.00026.

5. Casagrande D, Grossi P, Tomasi R (2016) Shake table tests on a full-scale timber-frame building with gypsum fibre boards. *European Journal of Wood and Wood Products*, **74**(3): 425-442, doi: 10.1007/s00107-016-1013-6.
6. Brandner R, Flatscher G, Ringhofer A, Schickhofer G, Thiel A (2016) Cross laminated timber (CLT): overview and development. *European Journal of Wood and Wood Products*, **74**(3): 331-351, doi: 10.1007/s00107-015-0999-5.
7. Hristovski V, Dujic B, Stojmanovska M, Mircevska V (2013) Full-scale shaking-table tests of XLam panel systems and numerical verification: Specimen 1. *Journal of Structural Engineering*, **139**(11): 2010-2018, doi: 10.1061/(ASCE)ST.1943-541X.0000754.
8. Hristovski V, Mircevska V, Dujic B, Garevski M (2017) Comparative dynamic investigation of cross-laminated wooden panel systems: Shaking-table tests and analysis. *Advances in Structural Engineering*, doi: 10.1177/1369433217749766.
9. EN 1995-1-1:2004/A2 (2014) Eurocode 5: Design of timber structures. Part 1-1: General. Common rules and rules for buildings. CEN, Brussels, Belgium.
10. EN 1998-1:2004/A1 (2013) Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. CEN, Brussels, Belgium.
11. ÖNORM B 1995-1-1 (2014) Eurocode 5. Bemessung und Konstruktion von Holzbauten. Teil 1-1: Allgemeines. Allgemeine Regeln und Regeln für den Hochbau. Nationale Festlegungen zur Umsetzung der ÖNORM EN 1995-1-1 nationale Erläuterungen und nationale Ergänzungen. ÖN, Wien, Austria.
12. Pei S, van de Kuilen J-WG, Popovski M, Berman JW, Dolan JD, Ricles J, Sause R, Bromgren H-E, Rammer DR (2016) Cross-Laminated Timber for seismic regions: Progress and challenges for research and implementation. *Journal of Structural Engineering*, **142**(4): E2514001, doi: 10.1061/(ASCE)ST.1943-541X.0001192.
13. Polastri A, Giongo I, Piazza M (2017) An innovative connection system for CLT structures. *Structural Engineering International*, **27**(4): 502-511, doi: 10.2749/222137917X14881937844649.
14. Dietsch P, Brandner R (2015) Self-tapping screws and threaded rods as reinforcement for structural timber elements - A state-of-the-art report. *Construction and Building Materials*, **97**: 78-89, doi: 10.1016/j.conbuildmat.2015.04.028.
15. Gavric I, Fragiacomo M, Ceccotti A (2015) Cyclic behaviour of typical metal connectors for cross laminated (CLT) structures. *Materials and Structures*, **48**(6): 1841-1857, doi: 10.1617/s11527-014-0278-7.

16. Flatscher G, Bratulic K, Schickhofer G (2015) Experimental tests on cross-laminated timber joints and walls. *Proceedings of the ICE - Structures and Buildings*, **168**(11): 868-877, doi: 10.1680/stbu.13.00085.

17. Tomasi R, Sartori T (2013) Mechanical behaviour of connections between wood framed shear walls and foundations under monotonic and cyclic load. *Construction and Building Materials*, **44**: 682-690, doi: 10.1016/j.conbuildmat.2013.02.055.

18. Gavric I, Fragiacomo M, Ceccotti A (2015) Cyclic behavior of typical screwed connections for cross-laminated (CLT) structures. *European Journal of Wood and Wood Products*, **73**(2): 179-191, doi: 10.1007/s00107-014-0877-6.

19. Hossain A, Danzig I, Tannert T (2016) Cross-Laminated Timber Shear Connections with Double-Angled Self-Tapping Screw Assemblies. *Journal of Structural Engineering*, **142**(11): 04016099, doi: 10.1061/(ASCE)ST.1943-541X.0001572.

20. Popovski M, Schneider J, Schweinsteiger M (2010) Lateral load resistance of Cross-Laminated Wood panels. *World Conference on Timber Engineering (WCTE)*, Riva del Garda, Italy.

21. Schneider J, Karacabeyli E, Popovski M, Stiemer SF, Tesfamariam S (2014) Damage assessment of connections used in Cross-Laminated Timber subject to cyclic loads. *Journal of Performance of Constructed Facilities*, **28**(6): A4014008, doi: 10.1061/(ASCE)CF.1943-5509.0000528.

22. Tomasi R, Smith I (2015) Experimental characterization of monotonic and cyclic loading responses of CLT panel-to-foundation angle bracket connections. *Journal of Materials in Civil Engineering*, **27**(6): 04014189, doi: 10.1061/(ASCE)MT.1943-5533.0001144.

23. Latour M, Rizzano G (2017) Seismic behavior of cross-laminated timber panel buildings equipped with traditional and innovative connectors. *Archives of Civil and Mechanical Engineering*, **17**(2): 382-399, doi: 10.1016/j.acme.2016.11.008.

24. Pozza L, Ferracuti B, Massari M, Savoia M (2018) Axial-shear interaction on CLT hold-down connections - Experimental investigation. *Engineering Structures*, **160**: 95-110, doi: 10.1016/j.engstruct.2018.01.021.

25. Liu J, Lam F (2016) Experimental test of Cross Laminated Timber connections under bi-directional loading. *World Conference on Timber Engineering (WCTE)*, Vienna, Austria.

26. Laggner TM, Flatscher G, Schickhofer G (2016) Combined loading of self-tapping screws. *World Conference on Timber Engineering (WCTE)*, Vienna, Austria.

27. Polastri A, Giongo I, Angeli A, Brandner R (2017) Mechanical characterization of a pre-fabricated connection system for Cross Laminated Timber structures in seismic regions. *Engineering Structures*, doi: 10.1016/j.engstruct.2017.12.022.

28. Loo WY, Quenneville P, Chouw N (2014) A new type of symmetric slip-friction connector. *Journal of Constructional Steel Research*, **94**: 11-22, doi: 10.1016/j.jcsr.2013.11.005.
29. Kramer A, Barbosa AR, Sinha A (2015) Performance of steel energy dissipators connected to Cross-Laminated Timber wall panels subjected to tension and cyclic loading. *Journal of Structural Engineering*, **142**(4): E4015013, doi: 10.1061/(ASCE)ST.1943-541X.0001410.
30. Sarti F, Palermo A, Pampanin S (2016) Fuse-type external replaceable dissipaters: experimental program and numerical modeling. *Journal of Structural Engineering*, **142**(12): 04016134, doi: 10.1061/(ASCE)ST.1943-541X.0001606.
31. Hashemi A, Zarnani P, Masoudnia R, Quenneville P (2017) Seismic resistant rocking coupled walls with innovative Resilient Slip Friction (RSF) joints. *Journal of Constructional Steel Research*, **129**: 215-226, doi: 10.1016/j.jcsr.2016.11.016.
32. Gavric I, Fragiacomo M, Ceccotti A (2015) Cyclic behavior of CLT wall systems: Experimental tests and analytical prediction models. *Journal of Structural Engineering*, **141**(11): 04015034, doi: 10.1061/(ASCE)ST.1943-541X.0001246.
33. Hummel J, Flatscher G, Seim W, Schickhofer G (2013) CLT wall elements under cyclic loading - Details for anchorage and connection. *COST Action FP1004, Focus solid timber solutions - European Conference on Cross Laminated Timber (CLT)*, pp: 152-165, Graz, Austria.
34. Dujic B, Aicher S, Zarnic R (2005) Investigation on in-plane loaded wooden elements - Influence of loading on boundary conditions. *Otto-Graf-Journal*, **16**: 259-272.
35. Okabe M, Yasumura M, Kobayashi K, Haramiishi T, Nakashima Y, Fujita K (2012) Effect of vertical load under cyclic lateral load test for evaluating Sugi CLT wall panel. *World Conference on Timber Engineering (WCTE)*, Auckland, New Zealand.
36. Yasumura M (2012) Determination of failure mechanism of CLT shear walls subjected to seismic action. *45th CIB-W18 Meeting*, Växjö, Sweden, Paper 45-15-3.
37. Loo WY, Kun C, Quenneville P, Chouw N (2014) Experimental testing of a rocking timber shear wall with slip-friction connectors. *Earthquake Engineering & Structural Dynamics*, **43**(11): 1621-1639, doi: 10.1002/eqe.2413.
38. Sarti F, Palermo A, Pampanin S (2016) Quasi-static cyclic testing of two-thirds scale unbonded posttensioned rocking dissipative timber walls. *Journal of Structural Engineering*, **142**(4): E4015005, doi: 10.1061/(ASCE)ST.1943-541X.0001291.
39. Hashemi A, Zarnani P, Masoudnia R, Quenneville P (2018) Experimental testing of rocking Cross-Laminated Timber walls with resilient slip friction joints. *Journal of Structural Engineering*, **144**(1): 04017180, doi: 10.1016/j.jcsr.2016.11.016.

40. Dunbar AJM, Moroder D, Pampanin S, Buchanan AH (2014) Timber core-walls for lateral load resistance of multi-storey timber buildings. *New Zealand Timber Design Journal*, **22**(3): 11-19.

41. Moroder D, Smith T, Dunbar AJM, Pampanin S, Buchanan AH (2018) Seismic testing of post-tensioned Pres-Lam core walls using cross laminated timber. *Engineering Structures*, doi: 10.1016/j.engstruct.2018.02.075.

42. Dujic B, Klobcar S, Zarnic R (2007) Influence of openings on shear capacity of wooden walls. *40th CIB-W18 Meeting*, Bled, Slovenia, Paper 40-15-6.

43. Yasumura M, Kobayashi K, Okabe M (2016) Failure analysis of CLT shear walls with openings subjected to horizontal and vertical loads. *World Conference on Timber Engineering (WCTE)*, Vienna, Austria.

44. Shahnewaz M, Tannert T, Alam MS, Popovski M (2017) In-plane stiffness of Cross-Laminated Timber panels with openings. *Structural Engineering International*, **27**(2): 217-223, doi: 10.2749/101686617X14881932436131.

45. Lauriola MP, Sandhaas C (2006) Quasi-static and pseudo-dynamic tests on XLAM walls and buildings. *COST Action E29, International Workshop - Earthquake Engineering on Timber Structures*, pp: 119-133, Coimbra, Portugal.

46. Ceccotti A, Follesa M (2006) Seismic behaviour of multi-storey XLam buildings. *COST Action E29, International Workshop - Earthquake Engineering on Timber Structures*, pp: 81-95, Coimbra, Portugal.

47. Ceccotti A, Sandhaas C, Okabe M, Yasumura M, Minowa C, Kawai N (2013) SOFIE project - 3D shaking table test on a seven-storey full-scale cross-laminated building. *Earthquake Engineering & Structural Dynamics*, **42**(13): 2003-2021, doi: 10.1002/eqe.2309.

48. Flatscher G, Schickhofer G (2015) Shaking-table test of a cross-laminated timber structure. *Proceedings of the ICE - Structures and Buildings*, **168**(11): 878-888, doi: 10.1680/stbu.13.00086.

49. Popovski M, Gavric I (2016) Performance of a 2-story CLT house subjected to lateral loads. *Journal of Structural Engineering*, **142**(4): E4015006, doi: 10.1061/(ASCE)ST.1943-541X.0001315.

50. Tsuchimoto T, Kawai N, Yasumura M, Miyake T, Isoda H, Tsuda C, Miura S, Murakami S, Nakagawa T (2014) Dynamic and static lateral load tests on full-sized 3-story CLT construction for seismic design. *World Conference on Timber Engineering (WCTE)*, Quebec City, Quebec, Canada.

51. Kawai N, Miyake T, Yasumura M, Isoda H, Koshihara M, Nakajima S, Araki Y, Nakagawa T, Sato M (2016) Full scale shake table tests of five story and three story CLT building structures. *World Conference on Timber Engineering (WCTE)*, Vienna, Austria.

52. Yasumura M, Kobayashi K, Okabe M, Miyake T, Matsumoto K (2016) Full-scale tests and numerical analysis of low-rise CLT structures under lateral loading. *Journal of Structural Engineering*, **142**(4): E4015007, doi: 10.1061/(ASCE)ST.1943-541X.0001348.

53. Ceccotti A (2008) New technologies for construction of medium-rise buildings in seismic regions: The XLAM case. *Structural Engineering International*, **18**(2): 156-165, doi: 10.2749/101686608784218680.

54. Pozza L, Trutalli D (2017) An analytical formulation of *q*-factor for mid-rise CLT buildings based on parametric numerical analyses. *Bulletin of Earthquake Engineering*, **15**(5): 2015-2033, doi: 10.1007/s10518-016-0047-9.

55. Pozza L, Scotta R, Trutalli D, Polastri A (2015) Behaviour factor for innovative massive timber shear walls. *Bulletin of Earthquake Engineering*, **13**(11): 3449-3469, doi: 10.1007/s10518-015-9765-7.

56. Casagrande D, Rossi S, Sartori T, Tomasi R (2016) Proposal of an analytical procedure and a simplified numerical model for elastic response of single-storey timber shear-walls. *Construction and Building Materials*, **102**: 1101-1112, doi: 10.1016/j.conbuildmat.2014.12.114.

57. Rinaldin G, Fragiacomo M (2016) Non-linear simulation of shaking-table tests on 3- and 7-storey X-Lam timber buildings. *Engineering Structures*, **113**: 133-148, doi: 10.1016/j.engstruct.2016.01.055.

58. Sustersic I, Fragiacomo M, Dujic B (2016) Seismic analysis of Cross-Laminated multistory timber buildings using code-drescribed methods: Influence of panel size, connection ductility, and schematization. *Journal of Structural Engineering*, **142**(4): E4015012, doi: 10.1061/(ASCE)ST.1943-541X.0001344.

59. Blaß HJ, Fellmoser P (2004) Design of solid wood panels with cross layers. *World Conference on Timber Engineering (WCTE)*, Lathi, Finland.

60. Izzi M, Polastri A, Fragiacomo M (2018) Investigating the hysteretic behavior of Cross-Laminated Timber wall systems due to connections. *Journal of Structural Engineering*, **144**(5): 04018035, doi: 10.1061/(ASCE)ST.1943-541X.0002022.

61. Moroder D, Sarti F, Smith T, Pampanin S, Buchanan AH (2016) The influence of diaphgram stiffness on the dynamic behaviour of multi-storey timber buildings. *World Conference on Timber Engineering (WCTE)*, Vienna, Austria.

62. CSA O86-14 (2016) Engineering design in wood. CSA Group, Mississauga, Ontario, Canada.
63. Pozza L, Scotta R, Trutalli D, Polastri A, Smith I (2016) Experimentally based *q*-factor estimation of cross-laminated timber walls. *Proceedings of the ICE - Structures and Buildings,* **169**(7): 492-507, doi: 10.1680/jstbu.15.00009.
64. Fragiacomo M, Dujic B, Sustersic I (2011) Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions. *Engineering Structures,* **33**(11): 3043-3053, doi: 10.1016/j.engstruct.2011.05.020.
65. Shen Y-L, Schneider J, Tesfamariam S, Stiemer SF, Mu Z-G (2013) Hysteresis behavior of bracket connection in cross-laminated-timber shear walls. *Construction and Building Materials,* **48**: 980-991, doi: 10.1016/j.conbuildmat.2013.07.050.
66. Rinaldin G, Amadio C, Fragiacomo M (2013) A component approach for the hysteretic behaviour of connections in cross-laminated wooden structures. *Earthquake Engineering & Structural Dynamics,* **42**(13): 2023-2042, doi: 10.1002/eqe.2310.
67. Pozza L, Saetta A, Savoia M, Talledo D (2017) Coupled axial-shear numerical model for CLT connections. *Construction and Building Materials,* **150**: 568-582, doi: 10.1016/j.conbuildmat.2017.05.141.
68. Izzi M, Polastri A, Fragiacomo M (2018) Modelling the mechanical behaviour of typical wall-to-floor connection systems for Cross-Laminated Timber structures. *Engineering Structures,* **162**: 270-282, doi: 10.1016/j.engstruct.2018.02.045.
69. Follesa M, Christovasilis IP, Vassallo D, Fragiacomo M, Ceccotti A (2013) Seismic design of multi-storey CLT buildings according to Eurocode 8. *Ingegneria Sismica: International Journal Earthquake Engineering,* **30**(4): 27-53.
70. Pozza L, Scotta R (2013) Influence of wall assembly on behaviour of cross-laminated timber buildings. *Proceedings of the ICE - Structures and Buildings,* **168**(4): 275-286, doi: 10.1680/stbu.13.00081.
71. Ceccotti A, Sandhaas C (2010) A Proposal for a standard procedure to establish the seismic behaviour factor q of timber buildings. *World Conference on Timber Engineering (WCTE),* Riva del Garda, Italy.
72. Popovski M, Karacabeyli E (2011) Seismic performance of Cross-Laminated Wood panels. *44th CIB-W18 Meeting*, Alghero, Italy, Paper 44-15-7.
73. Pei S, Van de Lindt JW, Popovski M (2013) Approximate R-factor for Cross-Laminated Timber walls in multistory buildings. *Journal of Architectural Engineering,* **19**(4): 245-255, doi: 10.1061/(ASCE) AE.1943-5568.0000117.

74. Popovski M, Pei S, Van de Lindt JW, Karacabeyli E (2014) Force modification factors for CLT structures for NBCC. *Materials and Joints in Timber Structures,* **9**, pp: 543-553, Springer, New York, USA, doi: 10.1007/978-94-007-7811-5_50.

75. Trutalli D, Pozza L (2018) Seismic design of floor-wall joints of multi-storey CLT buildings to comply with regularity in elevation. *Bulletin of Earthquake Engineering*, **16**(1): 183-201, doi: 10.1007/s10518-017-0193-8.

76. Newmark NM, Hall WJ (1982) *Earthquake spectra and design*. Earthquake Engineering Research Institute, California.

77. Fajfar P (1999) Capacity spectrum method based on inelastic demand spectra. *Earthquake Engineering & Structural Dynamics,* **28**(9): 979-994, doi: 10.1002/(SICI)1096-9845(199909)28:9<979::AID-EQE850>3.0.CO;2-1.

78. Fajfar P (2000) A nonlinear analysis method for performance-based seismic design. *Earthquake Spectra,* **16**(3): 573-592, doi: 10.1193/1.1586128.

79. Follesa M, Fragiacomo M, Casagrande D, Tomasi R, Piazza M, Rossi S, Vassallo D, Canetti D (2017) The new provisions for the seismic design of timber buildings in Europe. *Engineering Structures (under review)*.

80. Casagrande D, Doudak G, Mauro L, Polastri A (2018) Analytical approach to establish the elastic behaviour of multi-panel CLT shear-walls subjected to lateral loads. *Journal of Structural Engineering*, **144**(2): 04017193, doi: 10.1061/(ASCE)ST.1943-541X.0001948.

81. Reynolds T, Foster R, Bregulla J, Chang W-S, Harris R, Ramage M (2017) Lateral load resistance of Cross-Laminated Timber shear walls. *Journal of Structural Engineering,* **143**(12): 06017006, doi: 10.1061/(ASCE)ST.1943-541X.0001912.

82. Tamagnone G, Rinaldin G, Fragiacomo M (2017) A novel method for non-linear design of CLT wall systems. *Engineering Structures*, doi: 10.1016/j.engstruct.2017.09.010.

83. Flatscher G, Schickhofer G (2016) Displacement-based determination of laterally loaded Cross Laminated Timber (CLT) wall systems. *INTER 2016 Meeting*, Graz, Austria, Paper 49-12-1.

84. Park R, Paulay T (1975) *Reinforced Concrete Structures*. Wiley, New York, USA.

85. Paulay T, Priestley MJN (1992) *Seismic design of reinforced concrete and masonry building*. Wiley, New York, USA.

86. Chopra AK (2007) *Dynamics of Structures - Theory and Applications to Earthquake Engineering*. Prentice-Hall, Upper Saddle River, New York, USA.

87. Gavric I, Fragiacomo M, Ceccotti A (2013) Capacity seismic design of X-LAM wall systems based on connection mechanical properties. *46th CIB-W18 Meeting*, Vancouver, Canada, Paper 46-15-2.

88. Seim W, Hummel J, Vogt T (2014) Earthquake design of timber structures - Remarks on force-based design procedures for different wall systems. *Engineering Structures*, **76**: 124-137, doi: 10.1016/j.engstruct.2014.06.037.

89. Jorissen A, Fragiacomo M (2011) General notes on ductility in timber structures. *Engineering Structures*, **33**(11): 2987-2997, doi: 10.1016/j.engstruct.2011.07.024.

90. Schick M, Vogt T, Seim W (2013) Connections and anchoring for wall and slab elements in seismic design. *46th CIB-W18 Meeting*, Vancouver, Canada, Paper 46-15-4.

91. Vogt T, Hummel J, Schick M, Seim W (2013) Experimentelle Untersuchungen für innovative erdbebensichere Konstruktionen im Holzbau. *Bautechnik*, **91**(1): 1-14, doi: 10.1002/bate.201300083.

92. Izzi M, Flatscher G, Fragiacomo M, Schickhofer G (2016) Experimental investigations and design provisions of steel-to-timber joints with annular-ringed shank nails for Cross-Laminated Timber structures. *Construction and Building Materials*, **122**: 446-457, doi: 10.1016/j.conbuildmat.2016.06.072.

93. Ottenhaus L-M, Li M, Smith T, Quenneville P (2017) Overstrength of dowelled CLT connections under monotonic and cyclic loading. *Bulletin of Earthquake Engineering*, **16**(2): 753-773, doi: 10.1007/s10518-017-0221-8.

94. Scotta R, Trutalli D, Marchi L, Pozza L, Ceccotti A (2017) Capacity design of CLT structures with traditional or innovative seismic-resistant brackets *INTER 2017 Meeting*, Kyoto, Japan, Paper 50-15-5.

95. Brühl F, Kuhlmann U, Jorissen A (2011) Consideration of plasticity within the design of timber structures due to connection ductility. *Engineering Structures*, **33**(11): 3007-3017, doi: 10.1016/j.engstruct.2011.08.013.

Table 1. Values of the *q*-behaviour factor recommended in the literature.

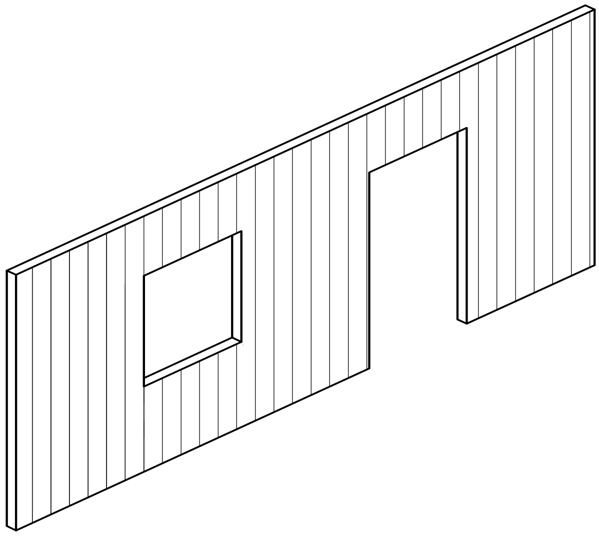
Reference	Method	Value
Ceccotti and Follesa [46]	Shaking table test of a 3-storey structure with narrow walls	3.4
Ceccotti *et al.* [47]	Shaking table test of a 7-storey structure with narrow walls	3.0
Flatscher and Schickhofer [48]	Shaking table test of a 3-storey structure with large walls	2.8
Pei *et al.* [73]	Simulations of full-scale structures with narrow walls	4.5
Pozza and Trutalli [54]	Simulations of full-scale structures with large walls	2.0
Popovski and Karacabeyli [72]	Single components tests (connections and CLT wall systems)	3.0
Popovski *et al.* [74]	Simulations of full-scale structures with narrow walls	3.0

Table 2. Values of the overstrength factor recommended in the literature.

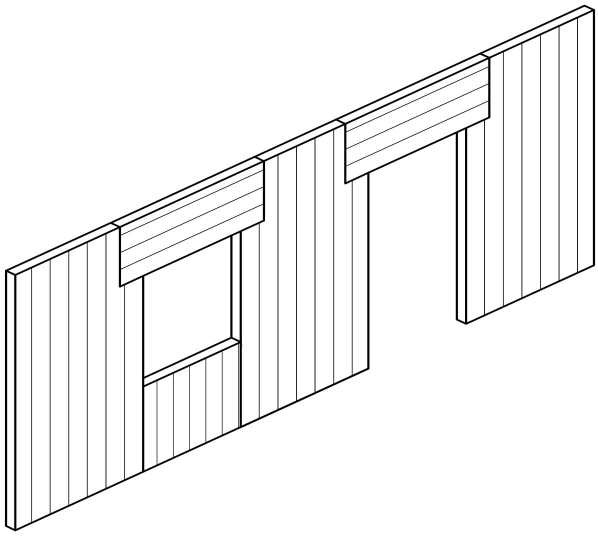
Reference	Connection type	$\gamma_{\mathrm{Rd}} = \gamma_{\mathrm{sc}}$	$\gamma_{\mathrm{Rd}} = \gamma_{\mathrm{an}} \cdot \gamma_{\mathrm{sc}}$
Jorissen and Fragiacomo [89]	Dowelled timber-to-timber joint (shear)	1.4	1.6
Gavric *et al.* [15]	Hold-down (shear and tension)	1.3	-
	Angle bracket (shear and tension)	1.2	-
Gavric *et al.* [18]	Screwed joint (withdrawal)	1.4	-
	Screwed timber-to-timber joint (shear)	1.7	-
Izzi *et al.* [92]	Nailed joint (withdrawal)	1.8	2.0
	Nailed steel-to-timber joint (shear)	1.4	2.0
Ottenhaus *et al.* [93]	Dowelled steel-to-timber joint (shear)	-	1.5

a. b. c. d.

Figure 1. Inappropriate failure mechanisms at the connection level: (a) withdrawal of the nails connected to the CLT floor panel; (b) pull-through of the anchoring bolt, (c) tensile failure in the net cross-section of the metal flange, and (d) buckling of the anchoring to the foundations.



a.



b.

Figure 2. Schematics of a CLT wall with openings obtained (a) by the cutting of the panel and (b) by the assemblage of multiple elements.

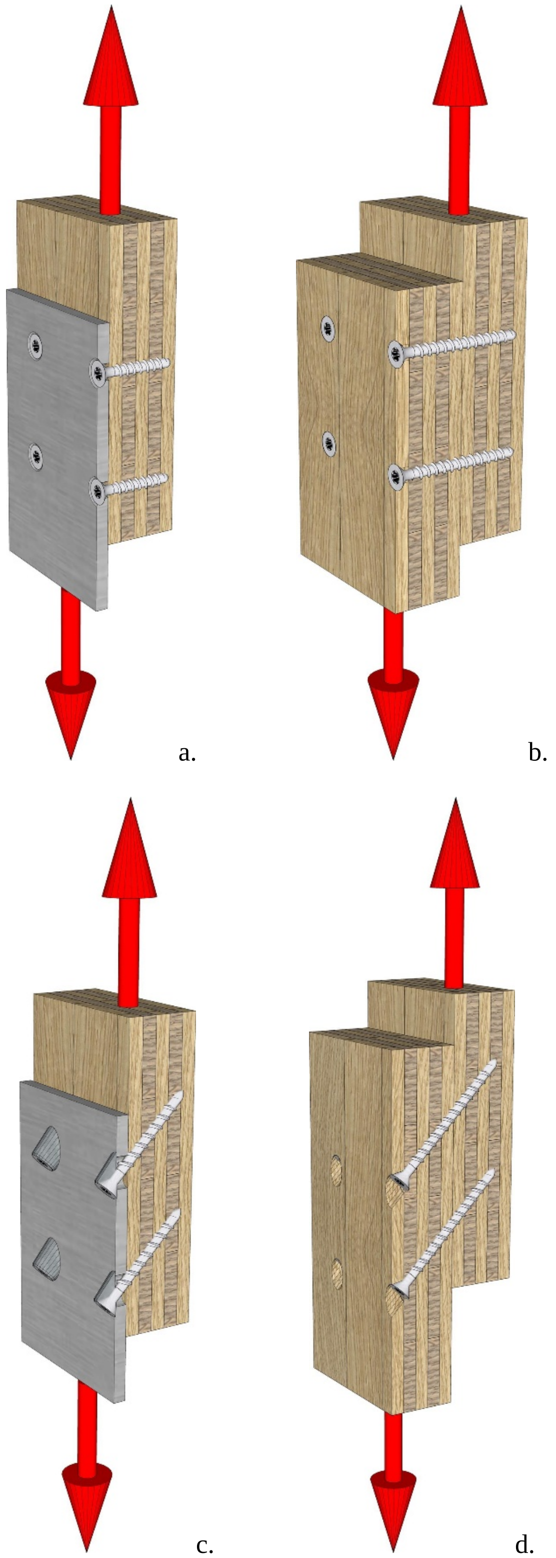
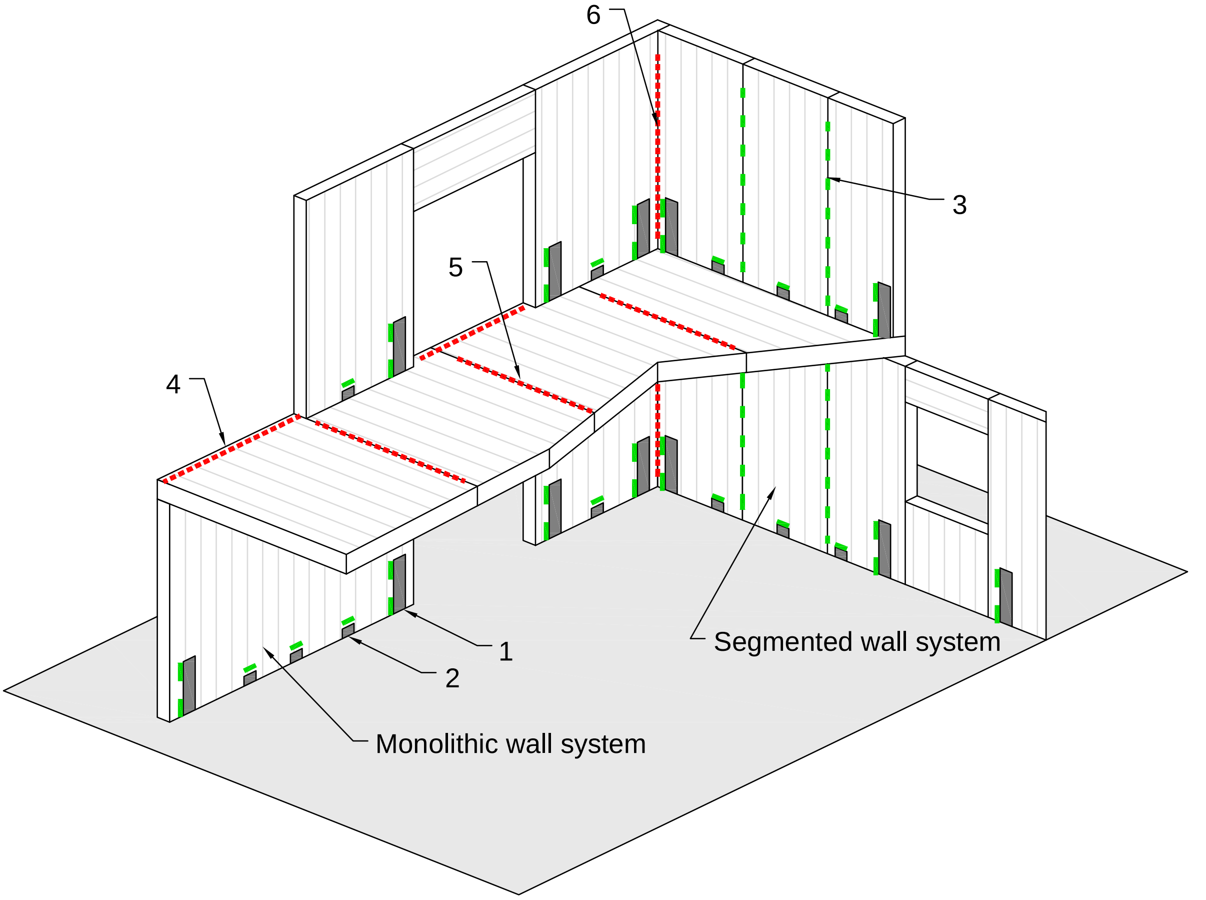


Figure 3. Schematic of (a) a dissipative steel-to-timber connection, (b) a dissipative timber-to-timber connection, (c) a non-dissipative steel-to-timber connection, and (d) a non-dissipative timber-to-timber connection.



Dissipative connections

1. Wall-to-floor connection against rocking
2. Wall-to-floor connection against sliding
3. Vertical joints between adjacent panels

Non-dissipative connections

4. Floor-to-wall panel connection
5. Floor-to-floor panel connection
6. Vertical joints between orthogonal panels

Figure 4. Schematics of a CLT structure, with indication of the dissipative and non-dissipative connections.