Behaviour of Cross-laminated Timber Panels under Cyclic Loads

Igor Gavric
Department of Architecture, Design and Urban Planning, University of Sassari, Italy
Email: gavric.igor@gmail.com

Massimo Fragiacomo
Department of Architecture, Design and Urban Planning, University of Sassari, Italy
Email: fragiacomo@uniss.it

Marjan Popovski
FPInnovations, Building Systems Department, Vancouver, BC, Canada
Email: marjan.popovski@fpinnovations.ca

Ario Ceccotti
CNR IVALSA Trees and Timber Institute, S. Michele all'Adige, Italy
Email: ceccotti@ivalsa.cnr.it

ABSTRACT

In this paper, the behaviour of cross-lam (CLT) wall systems under cyclic loads is examined. Experimental investigations of single walls and adjacent wall panels (coupled walls) in terms of cyclic behaviour under lateral loading carried out ìn Italy at IVALSA Trees and Timber Institute and in Canada at FPInnovations are presented. Different classifications of the global behaviour of CLT wall systems are introduced. Typical failure mechanisms are discussed and provisions for a proper CLT wall seismic design are given. The influences of different types of global behaviour on mechanical properties and energy dissipation of the CLT wall systems are critically discussed. The outcomes of this experimental study provides better understanding of the seismic behaviour and energy dissipation capacities of CLT wall systems.

1. Introduction

The key objective of this paper is to understand in detail how lateral systems in cross-laminated (CLT) structures behave under seismic actions, and how they influence the overall performance of CLT buildings in a seismic event. This is an important issue as this technology has become widespread in several regions of the world including earthquake-prone areas such as Italy, Japan and North America. Thus, in the past and recently, several experimental projects were performed on this subject.

A comprehensive study to determine the seismic behaviour of CLT wall panels was conducted at the University of Ljubljana, Slovenia. Numerous quasi-static monotonic and cyclic tests were carried out on walls with lengths of 2.44 m and 3.2 m and a height of 2.44 m or 2.72 m (Dujic et al., 2004, Dujic & Zarnic, 2005). Wall panels were subjected to different levels of vertical loads and monotonic or cyclic horizontal load applied according to different loading protocols. Also, the wall panels were tested at various boundary conditions, which enabled the development of wall deformations from the cantilever type to the pure shear. Influences of boundary conditions, magnitudes of vertical load and type of anchoring systems were evaluated in terms of wall deformation mechanisms and shear strengths of wall segments (Dujic et al., 2006a). Further, the influence of opening on the shear properties of CLT wall panels was investigated (Dujic et al., 2006b, 2007). Numerical models of CLT wall panels were implemented in the software package SAP2000 and verified against test results (Dujic et al., 2008). Results of the parametric study were used to derive analytical formulas describing the relationship between the shear strength and stiffness of CLT wall panels without and with openings.

This paper presents an extensive study of the behaviour of CLT panels subjected to cyclic horizontal loads. In previous years, extended experimental studies on CLT panels were conducted at CNR IVALSA Trees and Timber Institute (San Michele all' Adige, Trentino, Italy) by Ceccotti et al. (2006), and Gavric et al. (2011, 2013) and at FPInnovations in Vancouver, Canada (Popovski et al. 2010). At CNR IVALSA, the first part of cyclic tests on CLT wall panels was performed in 2006, focusing on single wall panel behaviour and behaviour of CLT wall panels with openings (Ceccotti et al., 2006). The walls had different types of connectors (hold-downs, angle brackets) and boundary conditions. A second part of the experimental programme started in 2010 (Gavric et al., 2011, Gavric, 2013), with the aim to test single connectors (hold-downs, angle brackets, screwed joints) and better understand the behaviour of coupled CLT wall panels. Different types of vertical joints were tested (step joint, spline LVL joint) with several different configurations of anchoring connectors. Analysis of seismic performance was done, with detailed investigation of energy dissipation properties and equivalent damping ratio of CLT timber panels.

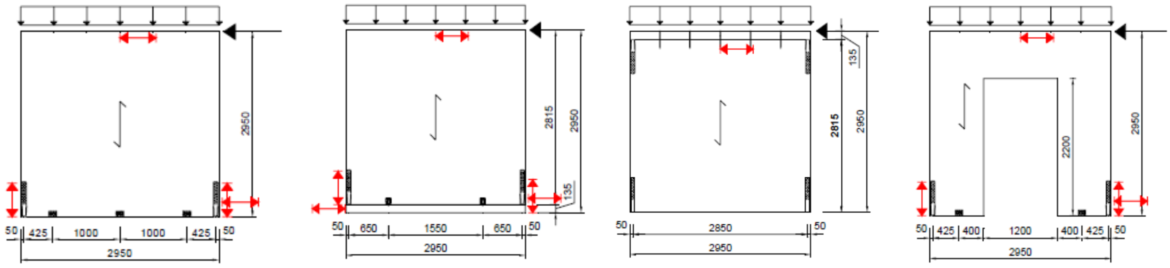
Similarly, an extensive experimental programme on CLT wall panels subjected to cyclic lateral loads was undertaken at FPInnovations in 2010 (Popovski et al., 2010). Wall configurations included single wall panels with three different aspect ratios, multi-panel walls with step joints and different types of screws to connect them, as well as two-storey wall assemblies. Various types of metal connectors were used, as well as several types of fasteners such as annular ring nails, spiral nails, and screws with different diameters and lengths.

In this paper, the results of 49 cyclic tests (25 conducted at FPInnovations in 2010 and 24 performed at CNR IVALSA in 2006 and 2010) are discussed and compared. Test results were analyzed in terms of mechanical properties and observed failure modes. In addition, evaluation and analysis of energy dissipation properties and equivalent damping was performed. The influence on seismic performance of various parameters, including geometry of panels, vertical loads, connection configuration, number and type of metal connectors, type of fasteners and type of vertical joint between adjacent panels was also studied. Two different classifications of CLT behaviour types under cyclic loads will be introduced, and parameters that are influencing overall performance of CLT walls will be studied.

2. CLT Wall Test Experimental programme

2.1 CLT Wall Tests at CNR IVALSA Research Institute

In 2005, an experimental study to quantify the seismic behaviour of CLT wall panels subjected to lateral loads was performed at CNR-IVALSA Institute (Ceccotti et al., 2006). This experimental research was a first step of the SOFIE project, which included experimental tests of full-scale CLT buildings (Ceccotti, 2008). The CLT wall testing programme involved single wall panels (2.95 m × 2.95 m) with different connection layouts, including walls with openings, subjected to cyclic loading with different levels of vertical loads applied on the wall (Figure 2.1). Wall-foundation and wall-floor connections were tested with different types of metal connectors, which were then used in the SOFIE buildings. Results from the quasi-static tests on CLT wall panels showed that the connection layout and design has strong influence on the overall behaviour of the wall. Hysteresis loops were found to have an equivalent viscous damping of 12% on average which makes the system suitable for implementation in high intensity earthquake-prone areas. The cyclic tests showed that the construction system is very stiff but still ductile (Ceccotti et al., 2006).

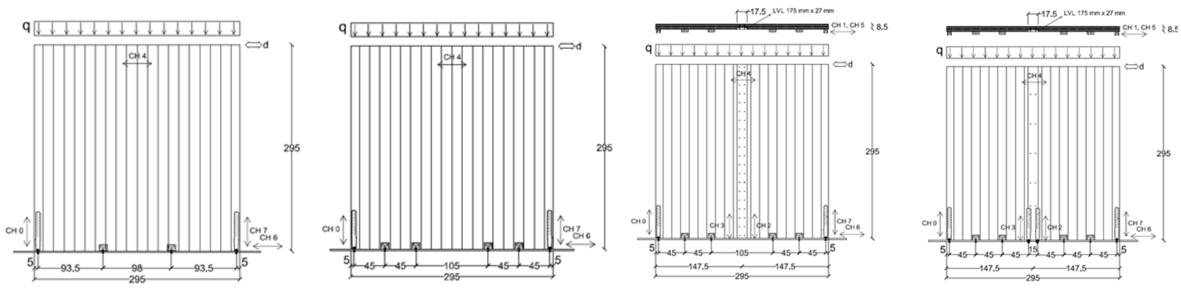


Configuration A Configuration B Configuration C Configuration D

Figure 2.1 *CLT wall test configurations tested at CNR IVALSA by Ceccotti et al. (2006) (dimensions in mm)*

Additional experimental tests were conducted on CLT wall panels in 2010 and 2011. The main goal of this testing programme was to provide a better insight of the seismic performance of single and coupled adjacent CLT wall panels subjected to cyclic loads and to understand the differences in their seismic behaviour. Wall configurations included single panel walls with different connection layouts, coupled wall panels with half-lap joints and different types of screws to connect them. In total 16 cyclic wall tests were performed, including single walls and coupled walls with in-plane screwed connections. Different connection layouts were used, with the aim to investigate how they affect the entire wall behavior and, possibly, optimize the structural performance.

Experimental wall tests were performed on 2.95 m × 2.95 m single walls and on 1.48 m × 2.95 m coupled walls (Figure 2.2). Different connector layouts and applied vertical load were used. The types of metal connectors, screws and nails used were the same as those used in the 3-story SOFIE building tested on a shaking table in Japan (Ceccotti 2008).



Configuration S-I Configuration S-II Configuration C-III Configuration C-IV

Figure 2.2 *CLT wall test configurations tested at CNR IVALSA by Gavric (2013) (dimensions in mm)*

The experimental test results were assessed in terms of strength, stiffness, energy dissipation, equivalent damping ratio, ductility and strength degradation, following the standard procedure from EN12512 (2001). The values of mechanical properties for a single wall specimen were analyzed by taking into account the results from the experimental hysteretic loops. Detailed assessment of these properties is presented in Gavric (2013).

2.2 CLT Wall Tests at FPInnovations Research Institute

In 2010 a series of monotonic and cyclic CLT wall tests was conducted at FPInnovations Research Institute, Vancouver, Canada (Popovski et al., 2010). In total 12 different configurations including one- and two-storey walls, single and coupled walls, and different wall-to-floor and wall-to-wall connections details were performed. Different types of connectors (hold-downs, angle brackets) and fasteners (ring nails, spiral nails, self-threaded screws, timber rivets) were used as well as different aspect ratios, levels of vertical loads and loading protocols (Figure 2.3).

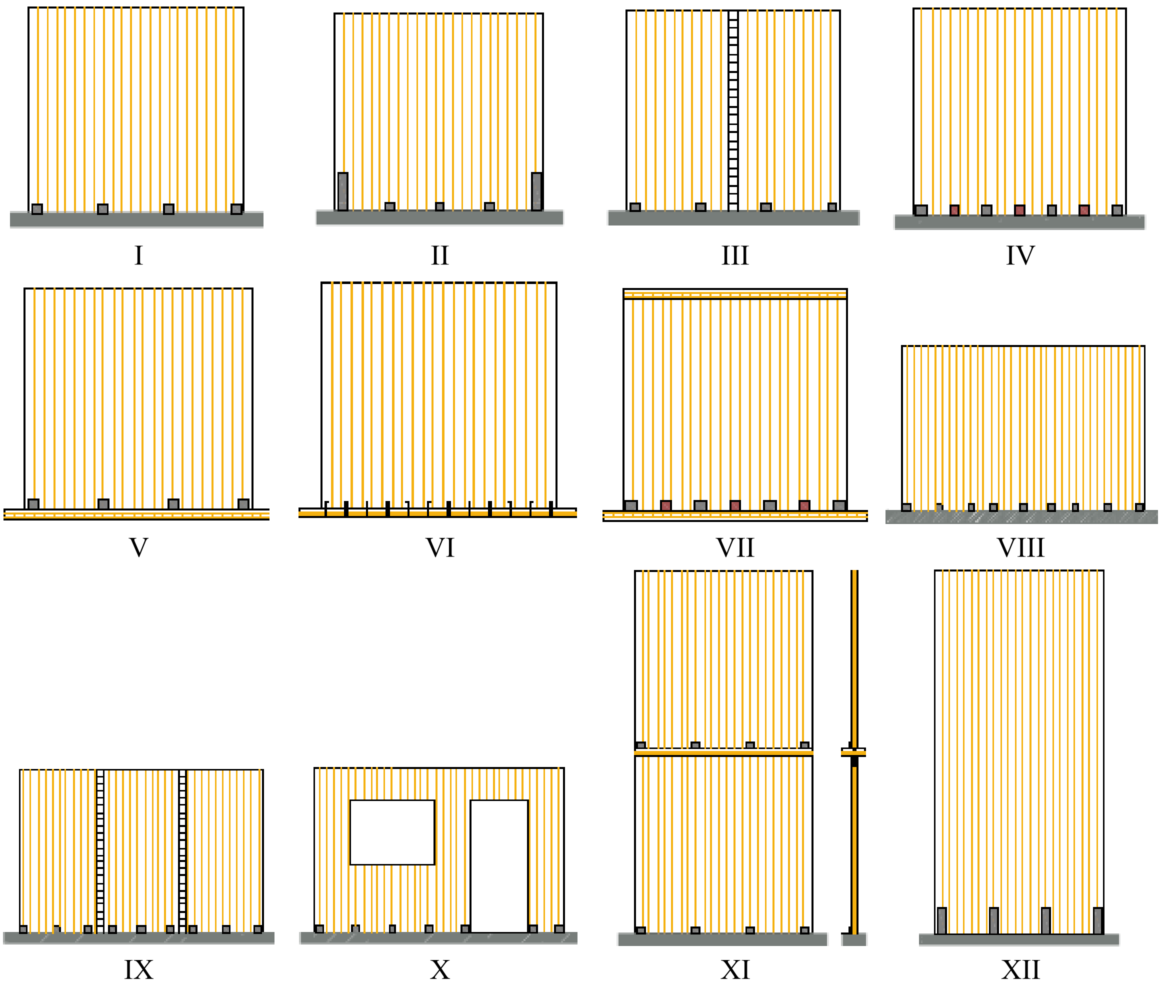


Figure 2.3 *CLT wall test configurations tested at FPInnovations by Popovski et al. (2010)*

Results from quasi-static tests on CLT wall panels showed that CLT walls can have adequate seismic performance when nails or screws are used to connect the steel brackets to the wall. The use of hold-downs with nails on each end of the wall enhanced the seismic performance of the wall panel. The use of inclined screws to connect the CLT walls to the floor panel underneath performed relatively poor in terms of seismic performance. The use of half-lap joints in longer walls proved to be an effective solution to reduce the wall length and consequently to improve the wall deformation capabilities.

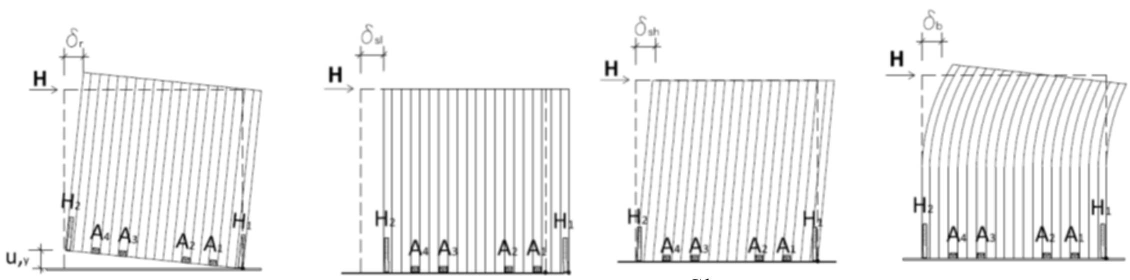
3. Classification of CLT Walls Behaviour Types

Understanding the behaviour of cross-laminated timber (CLT) wall systems subjected to lateral loads is crucial for a reliable seismic design of CLT buildings. In order to understand better what affects CLT walls seismic performance in terms of energy dissipation, hysteretic damping, ductility and displacement capacity (drifts), different classifications of wall panel behaviour under cyclic loading is presented in this section.

3.1 Classification based on predominant type of wall deformation during cyclic loading

The total top horizontal displacement δ_{tot} (interstory drift) of a CLT wall panel is a sum of four components: (i) rocking δ_r; (ii) slip δ_{sl}; (iii) shear deformation δ_{sh}; and (iv) bending deformation δ_b (Figure 3.1):

$$\delta_{tot} = \delta_r + \delta_{sl} + \delta_{sh} + \delta_b \tag{3.1}$$



B

Rocking Slip Shear ending

Figure 3.1 *Deflection components of a CLT wall panel (Gavric et al., 2011)*

In nearly all tests, most of the total top horizontal displacement was a consequence of panel rocking and sliding. Shear and bending deformation of the panels were generally negligible as, on average, their contribution on the total deflection was only 2.77%. The only case where shear and bending deformations became more important was CLT wall panels with large openings. A correlation between type of CLT wall panel and predominant type of deformation was found. Based on the results of numerous CLT wall cyclic tests, the following classification based on predominant type of wall deformation during cyclic loading can be proposed:

i) Rocking behaviour
ii) Combined Rocking-Sliding behaviour
iii) Sliding behaviour

While shear and bending contributions were almost negligible (except for walls with large openings, see Dujic 2007), rocking deformation was governing the wall behaviour in the cases of coupled wall panels and tall wall panels. It was found that the geometry (aspect ratio) of the CLT panels influences significantly the rocking and sliding contributions to the total lateral displacement of the panels. In case of single wall panels with aspect ratios 1:1, panels with the same number of metal connectors but using angle brackets at the ends instead of hold-downs exhibited a significantly higher rocking behaviour. The higher proportion of sliding in case of walls with hold-downs is due to the lower shear stiffness of hold-downs in comparison with angle brackets. Furthermore, higher levels of vertical load decreased the rocking contribution, thus increasing the sliding contribution of the panel. Long wall panels had combined rocking-sliding behaviour, again due to the panel aspect ratio. Walls with relatively low number of metal connectors behaved predominantly with sliding as the resistance to overturning moments was higher than the panel shear resistance. Figure 3.2 presents the percentages of the rocking contribution to the total ultimate top horizontal displacement of the CLT wall panels tested at IVALSA and FPInnovations.

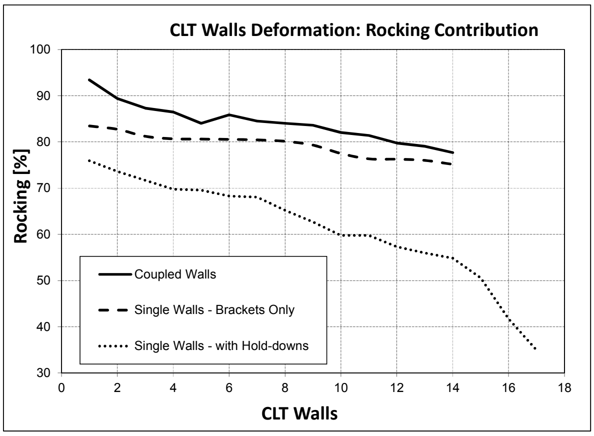


Figure 3.2 *Percentages of rocking contribution to the total CLT wall deformation at ultimate displacement under cyclic loading*

As can be seen in Figure 3.2, the rocking contribution was the highest in coupled CLT walls, followed by single walls with brackets only. The lowest rocking contribution was observed in the case of single CLT walls with hold-downs and angle brackets. On average, the rocking contribution for coupled walls was 84.2% (N = 14, COV = 5.0%), for single walls with only angle brackets was 79.3% (N = 14, COV = 11.1%) and for single walls with hold-downs and brackets was 61.5% (N= 17, COV = 18.3%).

3.2 Classification based on panels interaction during cyclic loading

Theoretically, there are three possible scenarios for the behaviour of adjacent CLT wall panels subjected to cyclic lateral loads:

i) Coupled wall panels behaving as independent, individual panels (Coupled wall behaviour)
ii) Coupled wall panels behaving as partly fixed panels with semi-rigid screw connection (Combined Single-Coupled wall behaviour)
iii) Coupled wall panels behaving as a single wall panel with rigid screw connection (Single wall behaviour)

In the first case, the vertical joint between wall panels is relatively weak in comparison to the anchoring connections, thus providing low level of stiffness between individual wall panels. While being loaded with lateral forces, connected panels behave as individual panels, rocking around each individual lower corner (Figure 3.3a). Conversely, if the vertical connection between coupled wall panels is very stiff, the behaviour of coupled walls is to the same as the behaviour of a single wall panel, as shown in Figure 3.3c. In this case, the vertical connection has higher resistance than shear forces between wall panels, and is very stiff. The third possibility is an intermediate, combined behaviour between individual wall behaviour and fully connected walls behaviour. As vertical connections between coupled wall panels are semi-rigid, slight deformations (slip) of the vertical connection can take place (Figure 3.3b).

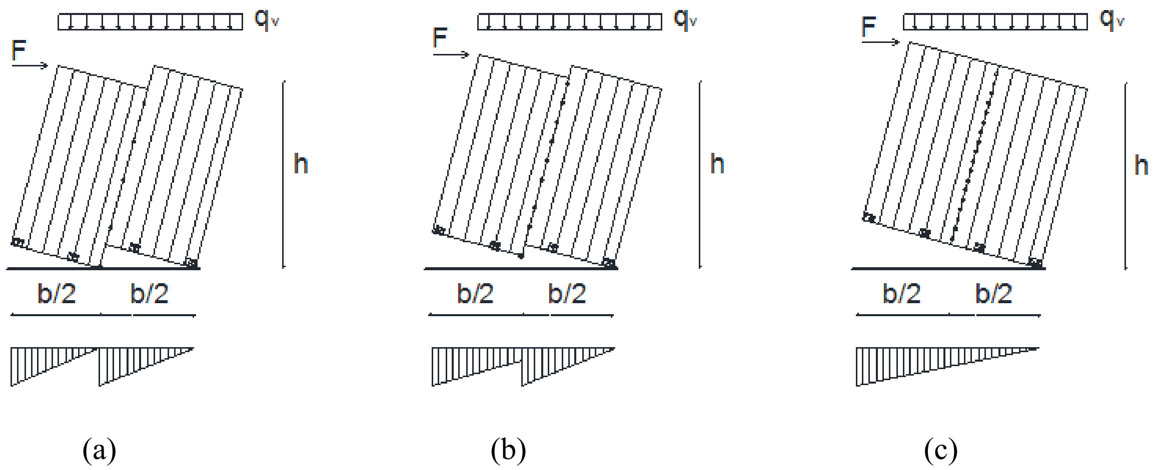


Figure 3.3 *Types of behaviour of adjacent wall panels: (a) Coupled wall behaviour; (b) Combined Single-Coupled wall behaviour; (c) Single wall behaviour*

Thus, special attention in coupled walls design should be given to the vertical connection between adjacent panels. By over-sizing the vertical connection may result in a completely different behaviour of the coupled wall panels. For example, if the stiffness of the vertical screwed connection is very high because large diameter screws are used at a small spacing, the behaviour of adjacent wall panels will become similar to that of a single wall panel. Thus, special care should be given when designing the vertical joint to achieve the desired wall behaviour. The "Single wall behaviour" results in higher strength capacity whereas the "Coupled wall behaviour" has lower elastic stiffness but attains larger ultimate displacements, which are also very important in earthquake design. Hence, the type of behaviour of wall subassemblies should be always decided a priori and then implemented into a proper design of the vertical joints. An example can be given by comparing wall tests 1.2, 2.1 and 2.3 (Gavric, 2013). Wall test 1.2 was a single wall (Figure 2.2, Configuration S-II), while walls 2.1 and 2.3 were coupled walls with screwed half-lap joint (Figure 2.2, Configuration S-III). As there were 20 screws connecting adjacent panels in case of Wall 2.1, the coupled panels behaved virtually as a single panel, exhibiting a very similar behaviour in terms of mechanical properties and energy dissipation capacity as wall panel 1.2. Wall 2.3 had only 50% screws in the vertical joint compared to Wall 2.1. This resulted in the so called "coupled wall behaviour", as both panels were rocking separately and not any more as one single panel. Here, the total dissipated energy was 35%-40% lower than in Walls 1.2 and 2.1. However, Wall panel 2.3 exhibited 33% higher ultimate displacement in comparison to single wall panel 1.2, showing higher level of flexibility. The reasons for the higher energy dissipation capacity of panels with "single wall panel behaviour" in comparison to wall panels with "coupled wall behaviour" up to a certain level of top horizontal displacement (interstory drift) is due to the aspect ratio of the panels.

Similarly, coupled wall CA-SN-11 (Figure 2.3 Configuration III) (Popovski et al., 2010) exhibited 43% lower energy dissipation in comparison with single wall CA-SN-03 (Figure 2.3 Configuration I), both having equal connection layout. Coupled wall CB-SN-16 with three adjacent panels (Figure 2.3 Configuration IX) had 37% lower total dissipated energy in comparison with a single long wall (3.45 m × 2.3 m) CB-SN-14 (Figure 2.3 Configuration VIII). The difference in unexpected higher level of energy dissipation is attributable to the aspect ratio of the panels; long single wall had aspect ratio 1.5:1, while each of the three adjacent panels in the long coupled wall had dimensions 1.15 m × 2.30 m (aspect ratio 1:2). The long single wall had three times higher elastic stiffness and 46% higher lateral load resistance F_{max}, but the long coupled wall exhibited 59% higher ultimate displacement (4.66% vs. 2.94% interstory drift). On the other hand, the single wall (CB-SN-14, Figure 2.3 Configuration VII) had a ductility ratio 26% higher than in the case of the coupled wall (CB-SN-16, Figure 2.3 Configuration IX), even if the ultimate displacement was significantly lower. The reason for that is the almost twice larger yielding displacement in case of the coupled wall (18.3 mm vs 9.5 mm) compared to the single wall. In terms of global behaviour, the failure mechanism was similar in both walls, namely: exceeded vertical uplifts in the corners of walls, while the horizontal displacements were relatively small.

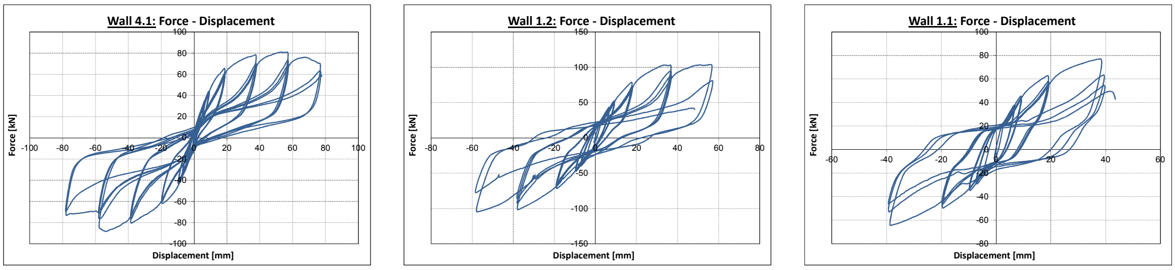
A practical design rule is that all brackets in each individual wall panel should be placed symmetrically with respect to both panel edges. This provides symmetrical wall behaviour in both directions, which is needed as horizontal earthquake and wind loads alternate their direction.

4. Analysis of CLT Walls Behaviour Under Cyclic Loads

Each CLT wall behaves under cyclic loading as a combination of the two classifications introduced in the previous section. How do these different combinations affect the walls performance under cyclic loads? The answer is provided in the following sections.

4.1 Hysteresis loops and failure mechanisms

Figure 4.1 displays the typical force-displacement hysteresis loops of CLT walls with different predominant types of behaviour.



(a) (b) (c)

Figure 4.1 *Typical force-displacement hysteresis loops: (a) Rocking behaviour; (b) Combined Rocking-Sliding behaviour; (c) Sliding behaviour*

In the first case (rocking mechanism), the CLT wall panel under horizontal cyclic loading failed due to exceeded uplifts as a consequence of rocking of the panel over the lower corners of the panel and crushing of the wood under the compressed corner. On average, the compression deformation of the lower corners of tested CLT panel was 2.6 mm. This value was higher for the walls with high height-to-width ratio (tall walls) and for the walls with higher vertical loads. Metal connectors (hold-downs, brackets) progressively yield, starting from the outer connectors, and progressing to the brackets installed in the central part of the wall. While in vertical (tension) direction metal connectors yielded and eventually failed, no significant deformation of connectors were observed in horizontal direction, thus no significant panel sliding occurred (Figure 4.1a). In cases of combined rocking-sliding mechanisms, the metal connectors failed due to combined shear-uplift displacements, thus yielding of the connectors occurred due to exceeded shear forces and the overturning moment (Figure 4.1b). In the last case, the sliding mechanism, the CLT wall failed due to exceeded shear resistance. Shear forces are concentrated in the angle bracket connectors, which consequently failed in shear (Figure 4.1c). Hold-downs do not have significant shear stiffness and shear resistance and typically fail due to buckling of the steel part when subjected to high shear forces.

Walls with rocking behaviour have the so called self-centering ability after being subjected to horizontal loads, due to the effect of vertical loads and the axial resistance of metal connectors. This means that after a seismic event, this type of wall system returns to the initial vertical position without any significant residual displacements. This kind of behaviour is very desirable in terms of seismic performance of lateral load resisting systems as even for high level of damages in the connectors, the CLT building can snap back to the initial vertical position. The only residual damage will be localized in the nailed connections between the steel brackets/hold-downs and CLT panel, where some timber crushing at the CLT panel-fastener connection will occur together with the plasticization of some fasteners. However, there will be little or no permanent deformation of the building, as the shaking table tests have clearly demonstrated (Ceccotti 2008). The damaged components can be easily replaced at the end of an earthquake, in this way minimizing the disruption caused by the seismic event, and enabling a prompt reuse of the building.

On the other hand, combined rocking-sliding behaviour has some non-reversible deformations after cyclic loading, while sliding mechanisms are not able to return by themself to the original position. Full scale building tests (Ceccotti 2008) showed that sliding of walls resulted in withdrawal of fasteners in walls perpendicular to them. That caused a reduction of strength and stiffness of the wall system in the opposite direction, thus reducing the overall strength and stiffness capacity of the entire building.

A suggestion is therefore given that at the wall level, plasticization should preferably occur in the hold-downs and angle brackets loaded in tension, whereas the angle bracket should ideally remain elastic in shear so that there is no residual slip in the wall at the end of the seismic event. This condition means that the angle brackets should be overdesigned with respect to the hold-downs. To achieve this result, capacity based design should be applied, in order to ensure that ductile modes of failure should precede the brittle modes of failure with sufficient reliability.

4.2 Mechanical properties

In terms of strength and stiffness capacity of CLT walls, the vertical load has beneficial impact. For example, Wall 3.6 (Figure 2.2 Configuration C-III) (Gavric, 2013) without any vertical load exhibited 45% lower lateral resistance and 24% lower initial stiffness in comparison with Wall 3.2 (Figure 2.2 Configuration C-III...) with 18.5kN/m vertical load. Similarly, wall CA-SN-00 (Figure 2.3 Configuration I) (Popovski et al., 2010) without any vertical load had 10% lower strength and 28% lower initial stiffness in comparison to wall CA-SN-03 (Figure 2.3 Configuration I) with 20 kN/m vertical load.

Higher elastic stiffness and strength of CLT walls can be achieved by increasing the number of metal connectors (wall CA-SN-20 vs CA-SN-03, Figure 2.3 Configuration IV), where 75% increased number of brackets resulted in 35% higher elastic stiffness and 55% higher strength. Moreover, presence of hold-downs in CLT walls increases the initial stiffness (80% higher k_{el} in case of wall CA-SNH-08A compared to CA-SN-03 without hold-downs, Figure 2.3 Configuration II).

As discussed in the previous section, coupled walls usually have lower elastic stiffness and strength capacity in comparison to single walls, while their ultimate displacement capacity is higher. Walls with aspect ratio b/h < 1 (tall walls), result in lower initial stiffness but higher displacement capacity in comparison with walls with ratio b/h > 1 (long walls), if the connection layout in both cases has the same distribution (Popovski et al., 2010).

CLT walls anchored to CLT floor panels have lower lateral stiffness and resistance capacity in comparison with CLT walls which are anchored to the foundation. This is mostly due to the larger flexibility of the CLT floor panel compared to a reinforced concrete foundation, and to the reduced load-carrying capacity of fasteners used in timber (usually self-threaded screws) compared to fasteners used for concrete (usually bolts). Thus, CLT wall-CLT floor panel connections are in general weaker in comparison with CLT-foundation connections (Ceccotti et al., 2006, Popovski et al., 2010).

Higher ductility ratios did not always correspond to better performance of CLT walls in terms of being able to dissipate energy in non-linear range with larger ultimate displacement capacity. For example, walls CS-WT-22 (Figure 2.3 Configuration VI) and CS-WT-22B (Figure 2.3 Configuration VI) had relatively high ductility ratios (7.54 and 4.97, respectively) in comparison with walls which were able to dissipate larger amount of energy and underwent larger interstory drifts, like for example wall CA-SN-20 (Figure 2.3 Configuration IV) (ductility ratio 3.65) with almost three times larger interstory drift capacity and five times more dissipated energy. A study of parameters which could possibly influence drift capacity of CLT wall was done; relatively good correlation between absolute values of rocking deformations and interstory drifts was found ($R^2 = 0.91$). Rocking deformations are directly related with CLT wall uplifts, which are usually measured during CLT wall experimental tests. Thus, instead of relative ductility definition (EN12512, 2001), emphasis should be given to total displacement capacity of CLT walls (Jorissen & Fragiacomo, 2011), as this quantity better represents the capacity of CLT walls to undergo large displacements during seismic events.

4.3 Energy dissipation capacity

In the previous sections, differences in total dissipated energy among different wall types were presented. In addition, equivalent viscous damping ratios were calculated for all walls (N = 49) at the displacement rate where the maximum force was attained.

For walls with single wall panel behaviour type with predominant rocking behaviour, the average hysteretic damping value was 14.50% (COV = 9.32%) for the 1^{st} cycles and 9.86% for the 3^{rd} cycles (COV = 13.97%), N = 25. For walls with coupled wall panel behaviour, the damping value was 14.25% (COV = 8.24%) for the 1^{st} cycles and 9.76% for the 3^{rd} cycles (COV = 9.90%), N = 14. No significant differences in hysteretic damping ratios were observed for walls with single wall panel behaviour with predominant rocking mechanism and walls with coupled wall panel behaviour, even though significant differences in total dissipated energy were calculated (Section 3.2).

Walls with predominant sliding behaviour exhibited higher equivalent viscous damping values, namely on average 17.50% for the 1^{st} cycles (COV = 7.57%) and 14.71% for the 3rd cycles (COV = 13.29%), N = 6. The higher values of hysteretic damping ratios in this case are due to the additional friction between CLT wall and floor panels, as the wall behaviour is predominantly sliding.

Tall walls had the lowest damping values; on average 10.02% for the 1st cycles (COV = 16.48%) and 5.55% for the 3rd cycles (COV = 14.72%), N = 5.

5. Conclusions

The study in this paper focused on the determination of different types of CLT walls behaviour, and how these types affect properties such as strength and stiffness capacity, displacement capacity (drift capacity), ductility ratios, energy dissipation capacity and hysteretic damping values, which are all important in seismic design of CLT systems. In addition, observations on the failure modes of connections provided an insight on how a proper design of typical CLT connections should be carried out. Different type of global panel behaviour resulted in different level of energy dissipation capacity. CLT wall panels with single wall panel behaviour performed better than walls with coupled wall behaviour in terms of total dissipated energy until a certain level of interstory drift. On the other hand, panels with coupled wall behaviour can exhibit higher ultimate displacements. Special attention should be given to the design of vertical joints between adjacent wall panels, as the global behaviour of wall panels can change dramatically and, consequently, the wall performance in terms of mechanical properties, energy dissipation capacity and displacement capacity can be significantly different.

For classification of CLT wall panels in terms of performance under cyclic loads, instead of relative ductility ratio definition, different quantities should be used. CLT wall properties such as interstory drift capacity and energy dissipation capacity can be evaluated more precisely using: (i) CLT wall uplifts, as a consequence of wall rocking; and (ii) types of wall global behaviour, namely: rocking behaviour vs sliding behaviour, and single wall panel behaviour vs coupled wall panel behaviour.

6. References

Ceccotti A., Lauriola M., Pinna M., Sandhaas C. (2006). *SOFIE Project - Cyclic Tests on Cross-Laminated Wooden Panels*, Proceedings of the 9th World conference on timber engineering, Portland, Oregon (USA).

Ceccotti A. (2008). *New technologies for Construction of Medium-Rise buildings in Seismic Regions: The XLAM case*. IABSE Structural Engineering International, 18(2), 156-165.

Dujic B., Pucelj J., Zarnic R. (2004). *Testing of Racking Behavior of Massive Wooden Wall Panels.* Proceedings of the 37th CIB-W18 Meeting, paper 37-15-2, Edinburgh, Scotland.

Dujic B., Zarnic R. (2005). Report on evaluation of racking strength of KLH system. University of Ljubljana, Faculty of civil and geodetical engineering, Slovenia.

Dujic B., Aicher S., Zarnic R. (2006a). *Testing of Wooden Wall Panels Applying Realistic Boundary Conditions.* Proceedings of the 9th World Conference on Timber Engineering, Portland, Oregon, USA.

Dujic B., Klobcar S., Zarnic, R. (2006b). *Influence of Openings on Shear Capacity of Massive Cross-Laminated Wooden Walls.* COST E29 International Workshop on Earthquake Engineering on Timber Structures, pages 105-118, Coimbra, Portugal.

Dujic B., Klobcar S., Zarnic, R. (2007). *Influence of Openings on Shear Capacity of Wooden Walls.* Proceedings of the 40th CIB-W18 Meeting, paper 40-15-6, Bled, Slovenia.

Dujic B., Klobcar S., Zarnic, R. (2008). *Shear Capacity of Cross-Laminated Wooden Walls.* Proceedings of the 10th World Conference on Timber Engineering, Miyazaki, Japan.

EN 12512 (2001). Timber structures - Test methods - Cyclic testing of joints made with mechanical fasteners. CEN, Brussels, Belgium.

Gavric I., Ceccotti A., Fragiacomo M. (2011). *Experimental cyclic tests on cross-laminated timber panels and typical connections*. Proceedings of the 14th ANIDIS Conference, Bari (Italy).

Gavric I. (2013). *Seismic Behaviour of Cross-Laminated Timber Buildings*, Ph.D. Thesis, University of Trieste, Italy. http://hdl.handle.net/10077/8638.

Jorissen, A., Fragiacomo, M. (2011). *General notes on ductility in timber structures.* Engineering Structures, Special Issue on Timber Structures, 33(11), 2987-2997.

Popovski M., Schneider J., Schweinsteiger M. (2010). *Lateral load resistance of cross-laminated wood panels.* Proceedings of the 11th World conference on timber engineering, Riva del Garda, Italy.