Quasi-static tests on a two-story CLT building

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Abstract: A two-story full-scale CLT building of 4.5 m x 9.1 m in plan, with a height of 5.04 m, was tested under quasi-static monotonic and cyclic loading for platform-type construction. The main objectives were to evaluate the global response of the structure, the performance of the shear walls, the behaviour of the connectors (hold-downs and angle brackets) and the frequency response of the structure during the tests. Lateral loads were applied on the storeys inducing torsion to the building. Loading procedure, number and disposition of connectors varied between tests. However, it is important to note that, in order to avoid a possible overlap of effects, the metal connectors hold-downs and angle-brackets only have been placed in CLT shear walls in each loading direction. In terms of performance, longitudinal direction presented a stiffer behaviour when compared to the transverse, where it was possible to verify greater sliding in the longitudinal direction and global rocking in the transverse direction. The results of this experimental campaign will be used for further analytical and numerical analyses, in order to help to implement more detailed seismic analysis, namely pushover, of CLT constructions.

Keywords: Cross Laminated Timber; Pushover analysis; Shear walls; Full-scale tests.

1. Introduction

In the search for new solutions based on wood derivatives and with the goal of taking the construction of wood to another level, Cross Laminated Timber (CLT), a competitive replacement for traditional structural materials such as steel, concrete and masonry, was created. It is a multi-layered shell product designed in Switzerland, in the early 1990s. The panels are prefabricated and have many advantages for both wall and floors. Being a relatively recent material, it is completely omitted on current European regulation EC5 [1]. On the other hand, there are already CLT handbooks for the Canadian [2] and US [3] markets [4]. In the last few years, full-scale tests on CLT buildings have been used to assess the performance of these structures for seismic regions [5] with the purpose of analyzing the global behavior of the structure after the tests were performed on individual elements: slabs and, in particular, walls. Nevertheless, it is also pertinent to evaluate the response of the connections materialized by metal devices like angle brackets and hold-downs based on cyclic tests.

Among the tests performed on a shaking table, it is important to point out the SOFIE project, in which a three-story building, with 7 m x 7 m in plan and 10 m of total height, including the roof, was tested with three different configurations (variation of openings). The building was subjected to a series of 26 earthquakes, including the 1995 great Hanshin-Awaji earthquake (in Kobe), at the NIED Laboratory, in Tsukuba, in July 2006. The results showed that the building resisted to 15 destructive earthquakes without any serious damage and no significant torsion was recorded [6].

Another high building with seven stories was tested, in 2007, in the shaking table of the E-Defense laboratory in Miki, Japan. The building with 13.5 m x 7.5 m and a total height of 23.5 m, was submitted to the Hanshin-Awaji earthquake in Kobe, the Italian earthquake of Nocera Umbra and the Kashiwazaki of the Japanese west coast. The walls of the building had 142 mm on the 1st and 2nd storeys, 125 mm on the 3rd and 4th and 85 mm in the others, including the roof. All the floors were 142 mm thick. The tests performed provided excellent results, as the building behaved very well on large-scale earthquakes, with very low structural damage. However, relatively high floor accelerations (maximum acceleration of 3.8 g) were recorded [7].

Two single-stories CLT models were tested in 2006, at the Dynamic Testing Laboratory of the Institute of Earthquake Engineering and Engineering Seismology at the Ss. Cyril and Methodius University, Skopje, Macedonia, using different earthquake records with PGA (Peak ground acceleration) of 0.6 g. As expected, no major damage was documented [8].

More recently, another CLT full-scale building was tested on the shaking table of the National Laboratory for Civil Engineering (LNEC), in Portugal. In the scope of the SERIES project aimed to evaluate multi-stories timber buildings, researchers from Graz University, National Laboratory of Civil Engineering (LNEC), University of Trento and University of Minho, tested a three-story CLT building with 5.17 m x 6.79 m in plan and 7.74 m of total height, including the roof (with 5.36m at the second floor). In terms of CLT components, the walls were of 100 mm (3-layers) panels, the floors had 150 mm (5-layers) and the roof 99 mm (3-layers). The steel connections used were angle brackets (AE116 Simpson Strong-Tie) and hold-downs (HTT22 Simpson Strong-Tie) with the corresponding nails and screws. The building was subjected to 32 seismic tests, in which the maximum ground acceleration was 0.5 g. At the end of these tests, the building presented minor damages (located in some connections and walls) with a decrease of the fundamental frequency from 3.98 Hz to 3.75 Hz [9].

Popovski and Gavric [10, 11] used a different approach, based on quasi-static tests, on a CLT building with 6.0 m x 4.8 m in plan and a height of 4.8 m. Most of the connections used were angle brackets (BMF 116x48x3x116) and hold-downs (HTT4) but their number and location varied on each test performed. The specimen was tested under monotonic and cyclic lateral loading, in five different tests. All the tests showed that the main failure mechanisms were the nails in the brackets at the bottom of the 1st floor story, as a consequence of sliding and rocking (uplift) deformations of the walls. Before the tests, the building registered a 13.5 Hz (E-W) and 11 Hz (N-S) fundamental frequency. After all the tests, the values decreased to 10.13 Hz and 7.63 Hz, respectively.

Two other CLT buildings were analyzed with a different application of CLT panels. In plan and height, both buildings presented 6.0 m x 4.0 m with 5.82 m of height, where the only difference was the CLT panels around the openings. While in one building the openings were cut directly on the CLT panels, in the other the openings were materialized trough segments. It is also important to note that buildings only featured hydraulic jacks on the 2nd floor. The results presented a greater stiffness for the structure without segmentation of the panels, where it was possible to see cracks at the corners of the openings. On the other hand, with segmented walls, the structure presented a high deformation caused by the rotation of each wall panel [12].

Based on these results, it is established that the resistance to lateral loads is mostly related to the behavior of the connections in the shear walls, where it showed high impact on flexibility and therefore greater stiffness, strength and ductility. Accordingly, several configurations of the panels were studied in order to evaluate the response of the panel. In the SOFIE project, 4 different configurations of walls were studied under quasi-static loading, where the influence of the metal connectors (in contact with the foundation and CLT panels), openings and the vertical loads were taken into account. The results showed that connectors have a great influence on the structural response, where ductility and dissipated energy is guaranteed by the metal connectors. Regarding the failure mode, damage was mainly located on metal connectors, where the configuration with door opening showed a local failure of wood in compression [13].

Another study analyzed the influence of openings studied, two different configurations: the opening of a window and door (41% of the entire panel). The results obtained showed a significant reduction of the shear stiffness, but at the level of the load capacity, did not obtain much difference [14].

Similarly, with different walls ratio, a series of 12 CLT wall configurations were tested at Forintek in Vancouver. The results showed that the CLT walls containing angle brackets and hold-downs at each end of the wall, presented an improved performance under lateral loads. On the other hand, the use of diagonal screws to connect CLT floors and CLT walls can reduce the wall ductility [15]. In this context, additional research in this field has been carried out, aiming to increase the knowledge of the real behavior of the shear walls [16-18].

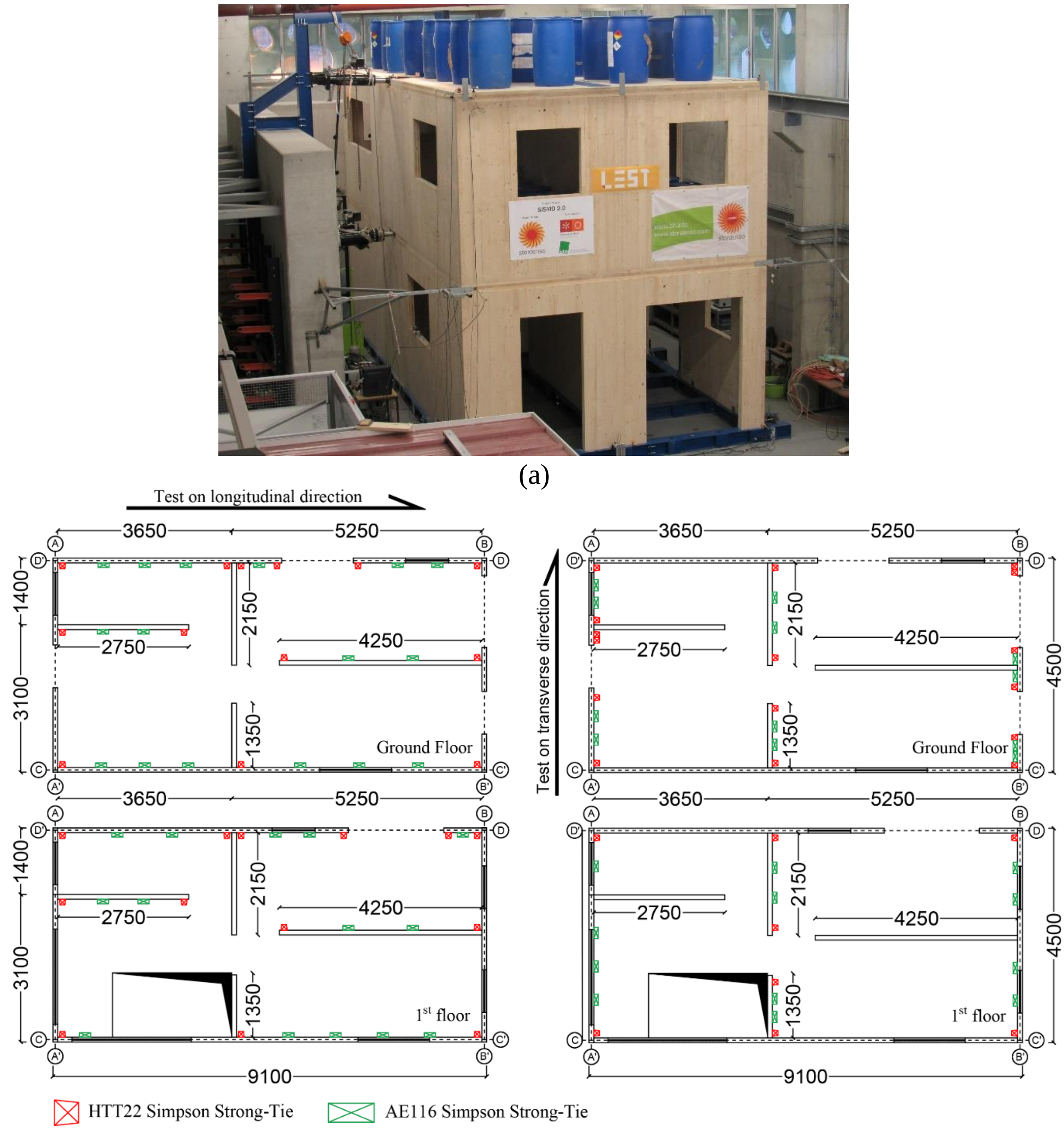
Finally, with focus on the seismic performance of timber structures, two state-of-the-art reviews were performed. One with the purpose of discussing displacement-based seismic design and their applications to timber buildings [19] and, on the other hand, only for CLT structures, the discussion was conducted mainly for experimental tests, numerical models, q-behavior factor and seismic design [20]. Given the facts mentioned, even with the research carried out, the regulations still do not provide a reliable method of seismic design, so there is still a need for adjustments between reality and design.

In this way, an experimental program based on quasi-static tests was planned at the University of Minho, Portugal, using a 2-story building, aiming the analysis of the 3-D system performance when subjected to lateral loads. The main variables for the experimental program were the analysis of lateral resistance and deformability capacity of the structure, frequency response and the performance of connectors (mainly AE116 and HTT22 from Simpson Strong-tie). The building was designed to obtain a non-symmetric response, with a clear distinction between the longitudinal (stiffer) direction and the transverse one and assuming that the center of mass had to be different from the center of stiffness. However, particularly in this experimental campaign, it was assumed that the metal connectors would be placed only in the CLT shear walls in each loading direction. The simplification was carried out looking at the numerical prediction of the experimental tests in a finite element model, aiming a representation with greater accuracy of the in-plane behavior of the metal connectors. In fact, the out-of-plane connectors have low resistance values, and therefore, can be considered as a safety factor in the seismic design. Nevertheless, it is important to note that due to the technical limitations of the hydraulic jacks, the loading and displacement were not sufficient to reach failure of the building. Apart from these, only a cyclic test was performed, where the reservation of all instrumentation and space were the main reasons. The following sections present and discuss the preparation works, the tests performed and the results obtained.

2. Experimental Program

2.1. Building description

The building had a plan of 4.5 m x 9.1 m, with two floors, with a total height of 5.04 m. Several partition walls and openings were included (a staircase on the 1st floor and on the external walls), with the purpose of creating an asymmetric structure prone to torsion. The CLT panels, were produced by Stora Enso Wood Products Ltd. These panels were made of spruce, with an approximate density of 470 kg/m^3. In terms of thickness, the CLT panels for the walls had 100 mm (5-layers of 20 mm) and the floors' CLT panels had 120 mm (3-layers with 40 mm). Several metal connectors were installed on the structure, mainly the angle bracket (AE116 - shear resistance) and the hold-down (HTT22 - uplift resistance). However, as they play an important role in final results and to avoid a possible overlapping of effects, the connectors were applied only to the shear walls where the test were performed. A panoramic image and plans of the building, with the location of the main connectors inserted in the tests are presented in Figure 1.



(a)

(b) (c)

Figure 1. Panoramic image (a) and building plans, with location of main connectors in the tests under longitudinal (a) and transversal (b) direction. (dimensions in mm).

The connections between the CLT wall panels were connected with LVL (laminated veneer lumber) spline joints, with the introduction of screws to ensure the continuity of the wall. The same connection method was used on floors. Regarding the openings in the walls, several windows and doors were included, as depicted in Figure 2. However, knowing that the openings can result in structural disorders, the percentage of openings in each façade is shown in Figure 2.

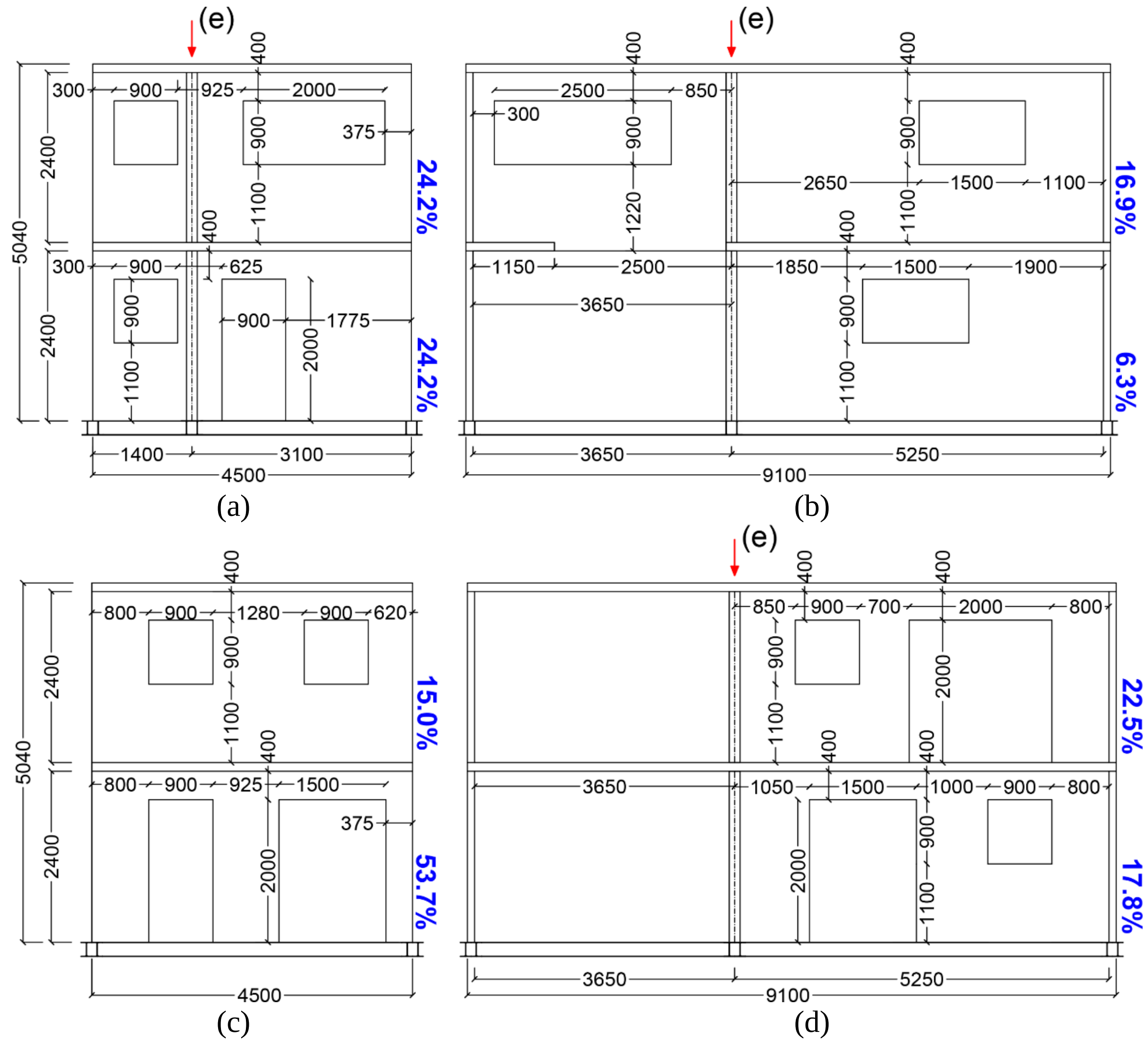


Figure 2. Building facades (dimensions in mm): (a) façade A-A’; (b) façade C-C’; (c) façade B-B’; (d) façade D-D’; (e) spline joint. (note that the plotted percentage values concern the relative area of the openings within each façade).

In terms of vertical loads, for representation of a real building, in addition to own weight, the remaining dead loads and the live-loads [21] (combinations of the seismic action of Eurocode 8 [22]) were placed over the building as additional masses, by distributing drums of water over the floors. A total of 2 kN/m^2 and a 1.7 kN/m^2 were applied for the first and second floors, respectively.

2.2. Setup and Instrumentation

The test setup was based on the need to have two lateral load additions in both directions of the CLT building, one in each floor. Thus, in order to achieve accurate experimental results, the main concerns of the test setup were: i) to have a rigid steel base to ensure an adequate fixation of the building to the reaction floor of the lab, including the fixation of the CLT panels of the first floor to the base with angle-brackets (AE116) and hold-downs (HTT22), as discussed above (see Figure 3a); ii) steel structure to place and fix two hydraulic jacks responsible for applying the lateral loads in both axes of the building (see Figure 3b); the hydraulic jacks, placed in the middle of the façades, included one hinge in each extremity, to avoid other deformations and stresses (see Figure 3c); iii) steel plate to ensure that the load applied by the hydraulic jacks on the CLT floors is distributed (see Figure 3d).

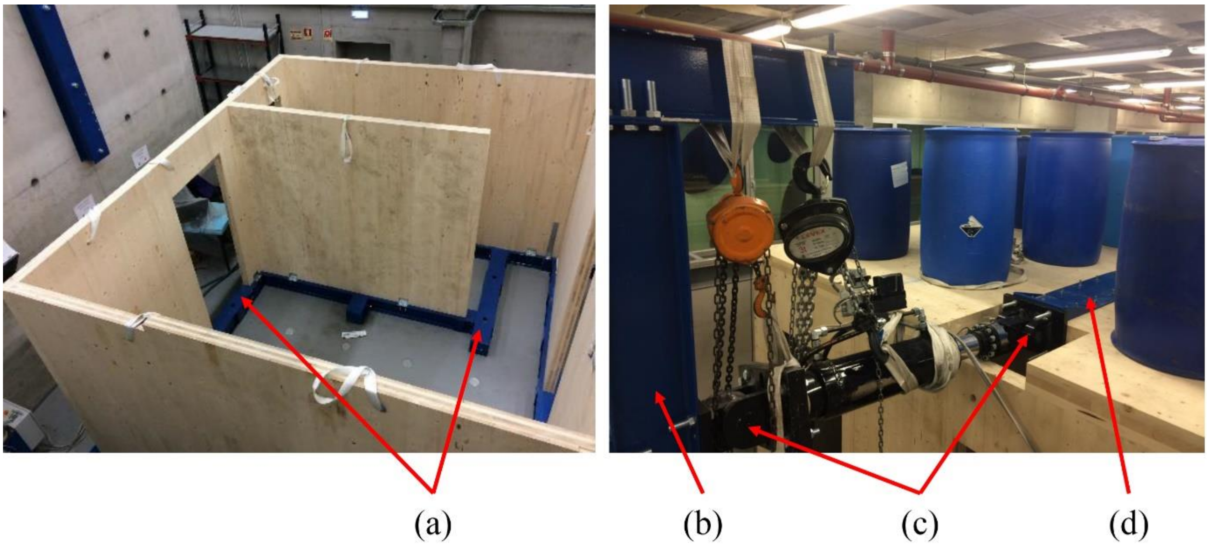


Figure 3. Setup used in the tests: (a) Steel base structure; (b) Steel structure to fix the hydraulic jacks and (c) respective hinges; (d) Steel plate placed on the floors.

The instrumentation system included 12 accelerometers, 4 on each level, in order to determine the natural frequencies of the CLT building. On the ground floor, the accelerometers were placed in each corner of the building, while, on the 1st and 2nd floor they were located only in two corners, at the intersection of facades A-A' and D-D' (see Figure 2) and at the intersection of facades B-B' and C-C' (see Figure 2). This information was crucial to analyze the behavior of the structure and to recognize if the damage in the building was induced by the tests performed. For the measurement of the displacement during each test, 24 LVDTs (Linear Variable Differential Transformer) were placed in demarcated positions, ensuring that not only the global deformation of the building, in each direction, was measured but also that the in-plane deformation, rotation of the floors, uplift of the walls panels and sliding were accurately registered. Figure 4 shows the location of: LVDTs; hydraulic jacks; and accelerometers, applied at different levels of the building.

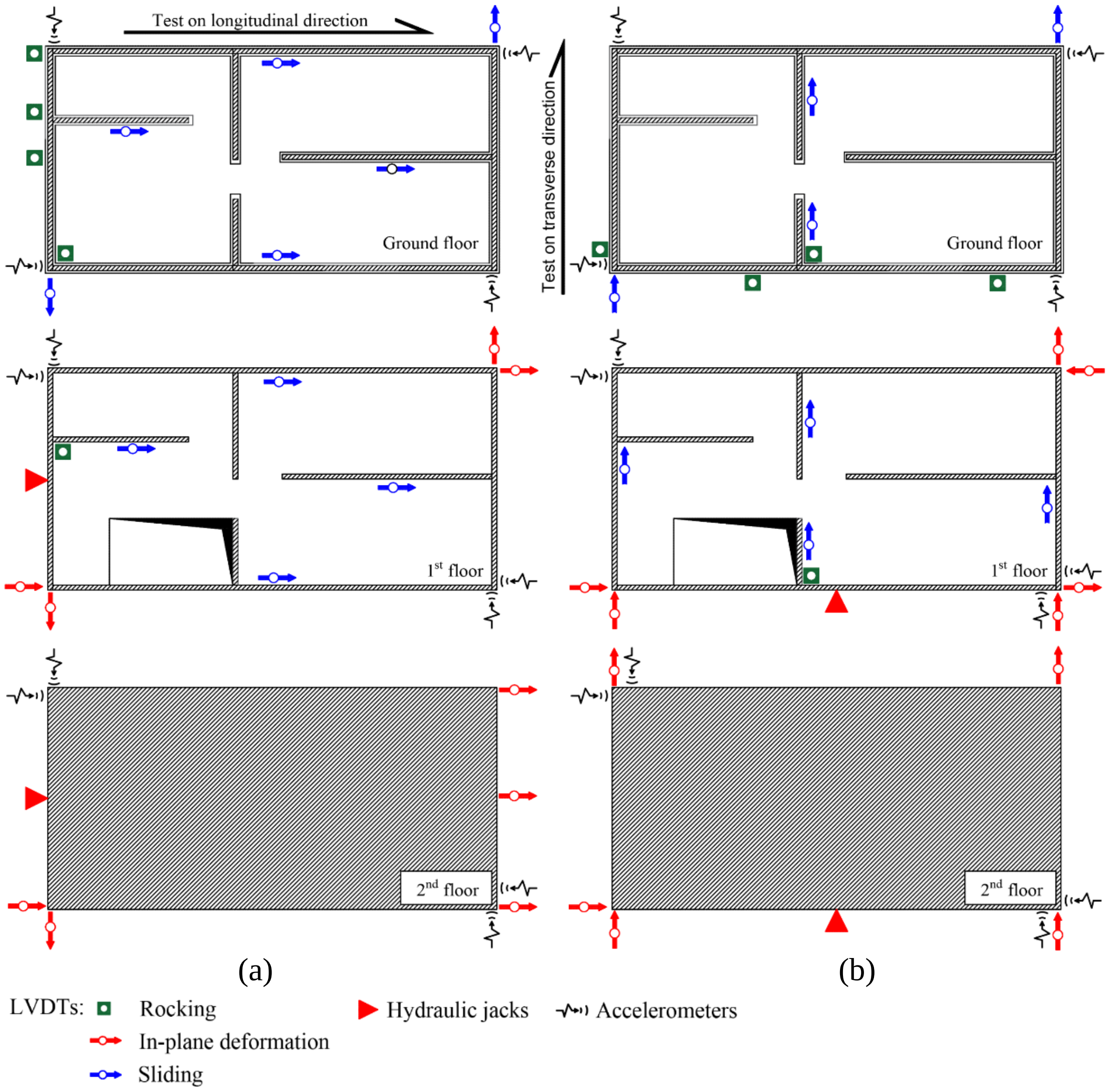


Figure 4. Instrumentation used in the tests under longitudinal (a) and transverse (b) direction.

2.3. Frequencies estimation and definition of connectors

Connections play an important role in the performance of CLT buildings and this case is no exception. The connections between the different CLT panels are crucial to ensure an adequate overall behavior of the system, keeping the different structural elements connected, while the local behavior of joints is fundamental to assure the deformability, ductility, and energy dissipation capacities needed. The connections used represented the common techniques used in practice, based on the use of angle brackets as shear connectors, hold-downs taking the uplift forces (tension) and adding screws to increase the stiffness of the connections. The metal connectors used, angle brackets and hold-downs, were supplied by Simpson Strong-tie, while the screws were from Rothoblaas (see Figure 5). To ensure a perfect distribution of the forces introduced by the hydraulic jacks at the floors level, steel plates, screwed to the CLT panels, were placed in both floors. Table 1 and Table 2 summarizes the different types of connections used and their locations.

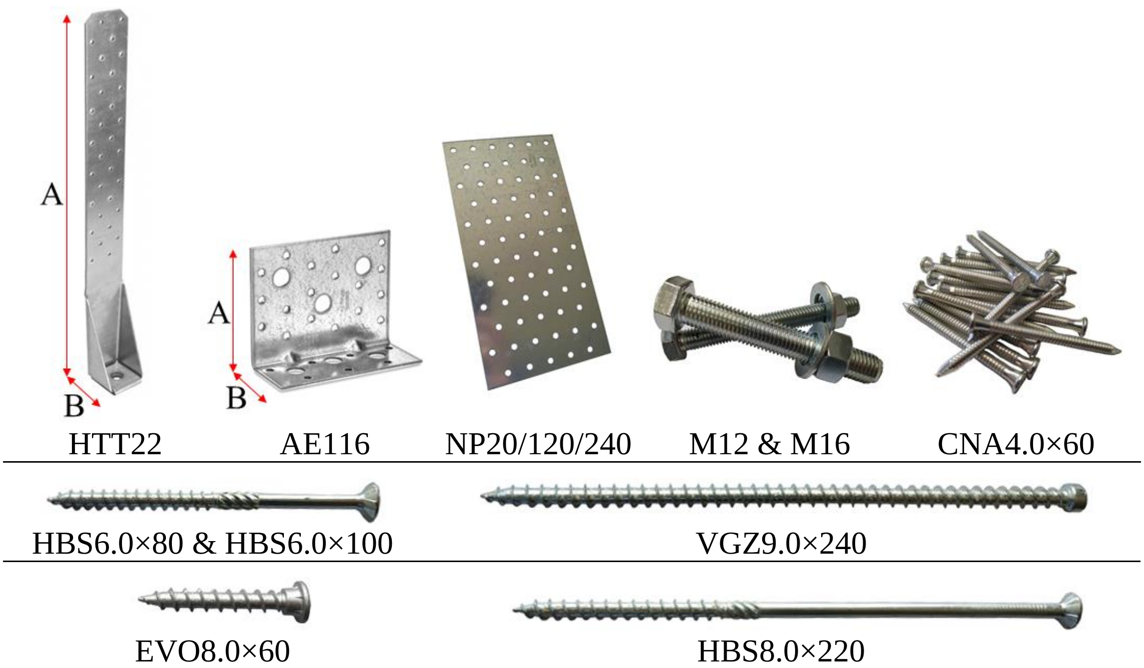


Figure 5. Connectors used in the CLT building.

Table 1. Main connectors used in the CLT building.

Location	**Type**	**Reference**	**Description**
Ground floor [CLT-to-Steel]	Angle bracket	AE116	14 × CNA4.0×60 (A) 2 × M12 (B)
	Hold-down	HTT22	14 × CNA4.0×60 (A) 1 × M16 (B)
1st floor [CLT-to-CLT]	Angle bracket	AE116	14 × CNA4.0×60 (A) 7 × CNA4.0×60 (B)
	Hold-down	HTT22	14 × CNA4.0×60 (A) 1 × M16 (B)
	Perforated plate	NP20/120/240	14 × CNA4.0×60 (staircase)

M12 - Threaded road Ø12 (8.8 Grade); M16 - Threaded road Ø16 (8.8 Grade)

Table 2. General fasteners used in the CLT building.

Quantity	**Location**
EVO8.0×60 + M12	Steel plate-floors
2 × (HBS6.0×80) spaced to 150 mm	Wall-to-wall (spline joints)
2 × (HBS6.0×100) spaced to 150 mm	Floor-to-floor (spline joints)
HBS8.0×220 spaced to 150 mm	Wall-to-wall
VGZ9.0×240 spaced to 150 mm	Floor-to-wall

M12 - Threaded road Ø12 (8.8 Grade)

In the definition and design of the AE116 shear connections used in the CLT building, the methodology proposed by Eurocode 8 [22] was adopted. In this method, the horizontal forces are determined from the total mass of the building and the spectral acceleration of the building for the respective period. Horizontal forces were applied independently in longitudinal and transverse directions, where two separate analyses were carried out with the same seismic demand. The total mass of the building admitted was 27 tons and, as the EC8 does not provide a simplified method to define the period for CLT structures, the Rayleigh method was applied with help of a finite element software RFEM [23] to the quantification of relative stiffness. Periods of 0.277 seconds (frequency of 3.60 Hz) and 0.385 seconds (frequency of 2.60 Hz) were obtained, for the longitudinal and transverse direction, respectively. In terms of seismic demand, the response spectrum was defined by NTC 2008 [24]. The location defined was the south of Italy (Calabria), with the goal of obtaining a spectrum with high seismic action. Regarding the behavior factor used, a value of 2 (ductility class medium) was assumed, according to working documents aimed to prepare a new version of Eurocode 8, chapter 8 [25, 26]. Under these circumstances, a peak ground acceleration of 0.42 g was found. Thus, as both periods were in the area of constant spectral acceleration (horizontal behavior), the seismic base shear force used for the design was 138 kN. On the other hand, the connectors HTT22, were the main responsible for the uplift resistance. In order to improve the performance under lateral loads [15], connectors HTT22 were introduced near all openings and at all corners of the shear walls (see Figure 1b and Figure 1c).

2.4. Monotonic Tests

The quasi-static monotonic tests carried out consisted on the application of a displacement under a constant rate, on each floor, respecting the ISO/FDIS 21581:2010 [27]. Two hydraulic jacks were used, one in each floor, to apply the displacements under a constant rate of 0.08 mm/s and 0.04 mm/s on the second and first floor, respectively. Due to technical limitations, namely the load capacity of the hydraulic jack installed on the second floor, the criterion adopted to stop the tests was a load value of 300 kN in that hydraulic jack. Two tests were performed: one for each direction, longitudinal and transverse.

2.5. Cyclic Test

The cyclic test, was also based on the loading procedure standardized by ISO/FDIS 21581:2010 [27], where the analysis was only in the transverse direction. Therefore, contrary to what happened with the monotonic tests, the loading procedure was, here, performed by force control, in which 0.90kN/s was admitted on the 1st floor and 1.80kN/s on the 2nd floor. Consequently, on the cyclic test, when the need for greater displacement of the hydraulic jacks occurred, the limitation was given by the maximum displacement of the 1st hydraulic floor of 100mm (50mm positive and 50mm negative). Concerning the values to be reached for each step, this was achieved based on the ultimate displacement (*lu*). Due to the lack of definition of this value, a final displacement equal to the total height of the building divided by fifteen (*H/15*), according to the standard, was admitted. Since this factor is quite conservative, the number of cycles of the fourth and fifth steps of the loading procedure was changed to three (see Figure 6). In relation to the inserted connections, they were equal to the ones in the monotonic test in the transverse direction, although all connections AE116 and HTT22 used were removed and new ones were introduced.

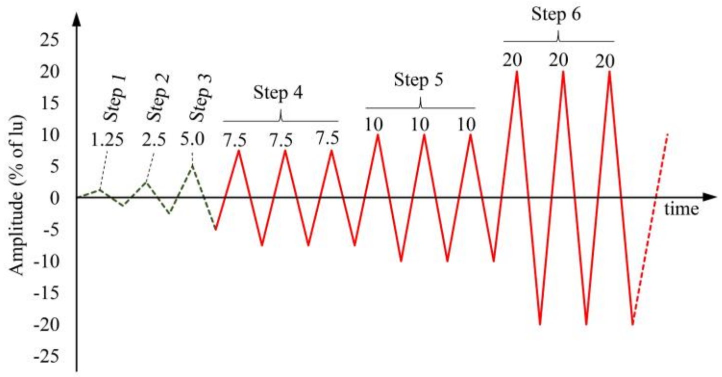


Figure 6. Loading procedure defined by ISO/FDIS 21581:2010 for cyclic tests [27].

3. Results and discussion

The main results obtained in the experimental program are described and discussed. Two experiments have been performed under monotonic loading and one with cyclic loading, and the results were separated in three groups: load-deformation response, dynamic analysis and damages observed.

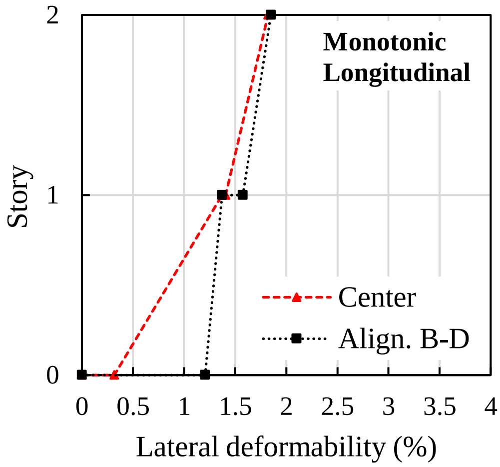
3.1 Load-deformation response

Table 3 and Figure 7 show the deformability of the building at different levels, concerning the results of the measurement of displacement (LVDTs) for the center of the facades A-A’ and C-C’ (location of the hydraulic jacks) and the farthest point in relation to the hydraulic jacks, region where greater displacements were obtained (intersection of facade B-B' and D-D'). It is important to note that the monotonic tests were stopped when the criterion of the limitation for the load applied by the hydraulic jack of the second floor (300 kN) was reached. Contrarily to the monotonic tests, for the cyclic test, due to the greater need of displacement, the hydraulic jacks of the first floor determined the stopping criterion, only having 100 mm of maximum displacement (50 mm for each direction). However, due to loss of displacements in the introduced hinges, the maximum displacement reached in the 1st floor was 30 mm.

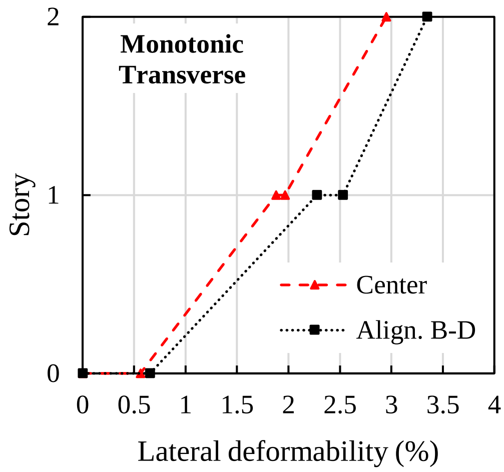
Table 3. Lateral deflection (mm) measured during the tests performed

Test		Monotonic Longitudinal	Monotonic Transverse	Cyclic Transverse
Location		**Hydraulic jacks**		
1st story	Sliding	8.0 (0.32% h)	14.1 (0.56% h)	7.4 (0.30% h)
	In-plan deformation	34.5 (1.37% h)	47.4 (1.88% h)	30.0 (1.19% h)
	Rocking	7.9 (0.31% h)	16.6 (0.66% h)	15.9 (0.63% h)
2nd story	Sliding	0.9 (0.03% h)	2.2 (0.09% h)	2.9 (0.12% h)
	In-plan deformation	45.9 (1.82% h)	74.3 (2.95% h)	52.4 (2.08% h)
	Rocking	1.6 (0.06% h)	2.7 (0.11% h)	2.3 (0.09% h)
Location		**intersection of facades B-B 'and D-D'**		
1st story	Sliding	30.4 (1.21% h)	16.5 (0.66% h)	12.7 (0.50% h)
	In-plan deformation	34.5 (1.37% h)	57.4 (2.28% h)	37.3 (1.48% h)
2nd story	Sliding	5.1 (0.20% h)	6.4 (0.25% h)	5.5 (0.22% h)
	In-plan deformation	46.6 (1.85% h)	84.4 (3.35% h)	57.8 (2.29% h)

h – Story height



(a)



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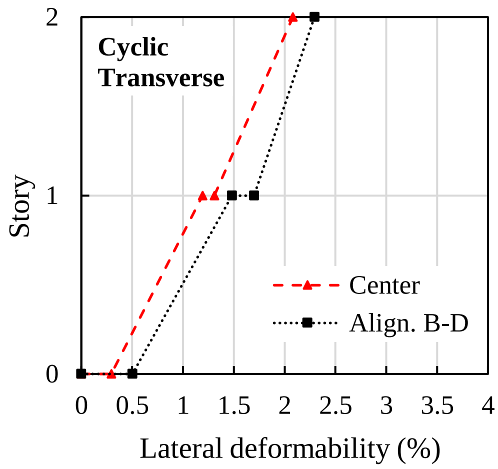


(c)

Figure 7. Lateral deformability of the building at different levels: (a) Monotonic test under longitudinal direction; (b) Monotonic test under transverse direction; (c) Cyclic test under transverse direction.

By analyzing Table 3 and Figure 7, transverse direction generally obtained greater sliding, rocking and in-plane wall deformation. It is important to note that, for the, stiffer, longitudinal direction, it presented greater slip of the base. On the other hand, the smaller values of displacement in cyclic test when compared to the monotonic test in the same direction, resulted from the lower load reached during the test. In these circumstances, it is possible to conclude that the longitudinal direction obtained a greater sliding behavior, and with this, more friction between the panels and the steel base was verified. In relation to transverse direction, as a consequence of a less in-plane stiffness, the global rotation was more evident. To better understand the behavior of the building (and as happened in the tests performed by Popovski [10]) it is important to point out the sliding occurred at the ground floor and 1st floor. This slip results from the stiffness differences between the steel base and the 1st floor walls. Although on a smaller scale, it also happens between the 1st and 2nd floor walls. On the other hand, the in-plane deformation, where it represents most of lateral deformation, occurred due to the shear and flexural deformation and to the global rocking of the CLT panels. Consequently, overlapping force-displacement graphs (see Figure 8) of the tests performed, the standard ASTM-E2126:2012 [28] has been applied to quantify the parameters of elastic shear stiffness (Ke), yield load (P_{yield}) and yield displacement (Δ_{yield}), as can be seen in the Table 4. However, in the cyclic test, for the application of the standard, the positive envelope curve has been used, as can be seen in the Figure 8. The load values reached in each hydraulic jack are also listed. In addition, the results of the comparison between the monotonic and cyclic test in transverse direction were added in Table 4 for the same load magnitude.

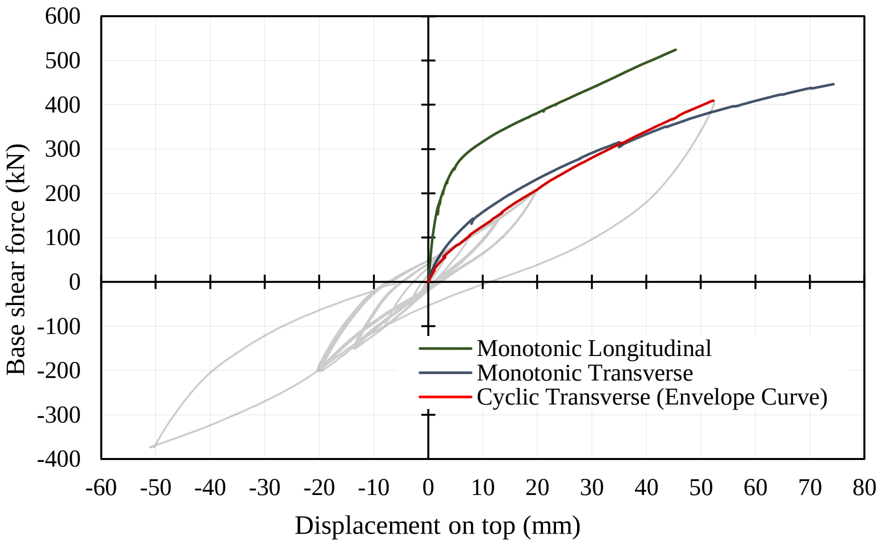


Figure 8. Force-displacement on top of the CLT building registered during the tests.

Table 4. Mechanical parameters with application of the ASTM-E2126:2012.

Tests	*Force* (kN) 1st story/2nd story	P_{peak} (kN)	Δ_{peak} (mm)	P_{yield} (kN)	Δ_{yield} (mm)	K_e (N/mm)
Monotonic Longitudinal	228.4/300.0	528.4	45.9	408.8	5.6	63241
Monotonic Transverse	147.7/300.0	447.7	74.3	347.5	19.2	14911
Monotonic Transverse[(a)]	142.2/266.8	409.0	60.1	328.3	21.4	15312
Cyclic Transverse	136.3/272.7	409.0	52.3	328.8	29.1	9910

P_{peak} - Maximum load; Δ_{peak} - Maximum displacement; Δ_{yield} - Yield displacement; P_{yield} - Yield load; K_e - Elastic shear stiffness; [(a)] Load magnitude of the cyclic test.

By looking at Figure 8 and Table 4, one can demonstrate that the CLT building is stiffer in the longitudinal direction when compared to the transverse direction, with a significant increase of the load capacity of the structure in that direction. On the other hand, when analyzing the tests in the transversal direction, the cyclic test presents lower values of resistance. Regarding the comparison between the monotonic and cyclic test in the transverse direction with same load magnitude, the results demonstrates the decrease of the resistance of the cyclic test. This decrease can be considered normal given that the cyclic test is more aggressive to the structure, in which there occurred a decrease in maximum displacement (around 13%), yielding displacement (around 36%) and elastic shear stiffness (around 35%). However, it is important to note that the value of yielding load is close.

3.2 Dynamic analysis

In relation to the results of the dynamic identification, Table 5 shows the natural frequencies for the cases with and without additional masses and before and after each test performed.

Table 5. Natural frequencies obtained during the tests.

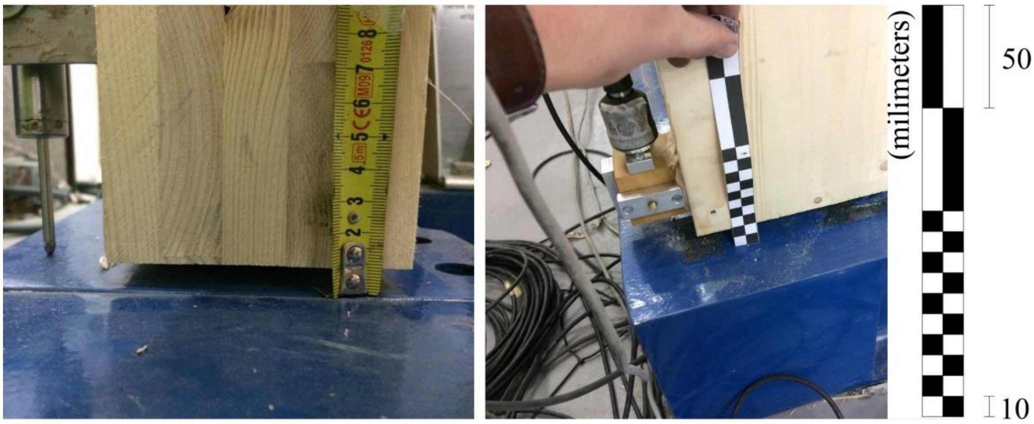
Test	**Natural frequency (Hz)**					
	Transverse direction (mode 1)			**Longitudinal direction (mode 2)**		
	before	**after**	**Δ (%)**	**before**	**after**	**Δ (%)**
Identification*	**8.2**	**5.0**	**38.6%**	**19.2**	**12.5**	**34.9%**
Mono. Longitudinal	5.0	4.9	2.4%	**12.5**	**11.0**	**12.2%**
Mono. Transverse	**6.0**	**4.9**	**18.5%**	6.4	5.8	9.8%
Cyclic Transverse	**5.6**	**4.6**	**18.1%**	5.8	4.9	15.8%

*before and after the introduction of additional masses

Analyzing the values presented in Table 5, for the direction in which the tests were performed, the transverse tests obtained greater damage (reduction of 18.5% and 18.1%) when compared to longitudinal test (reduction of 12.2%). In relation to the additional masses inserted in the building, the natural frequency decreased on 38.6% and 34.9% for the transverse and longitudinal direction, respectively.

3.3 Damages observed

The damages observed during the tests were very similar for all the tests performed, where the difference was given by the level of damage imposed on the building. In this way, and as expected, the damages observed during the test in the transverse direction, were more severe, due to the fact that this loading direction is the one with less stiffness. On the other hand, with the longitudinal direction being the stiffest, practically insignificant damages were found between the walls of the 2nd floor and the 1st floor. In this context, as the building suffered global rotation, the first visible damages concentrated at the base, where the hydraulic jacks were located (see Figure 9).



(a) (b)

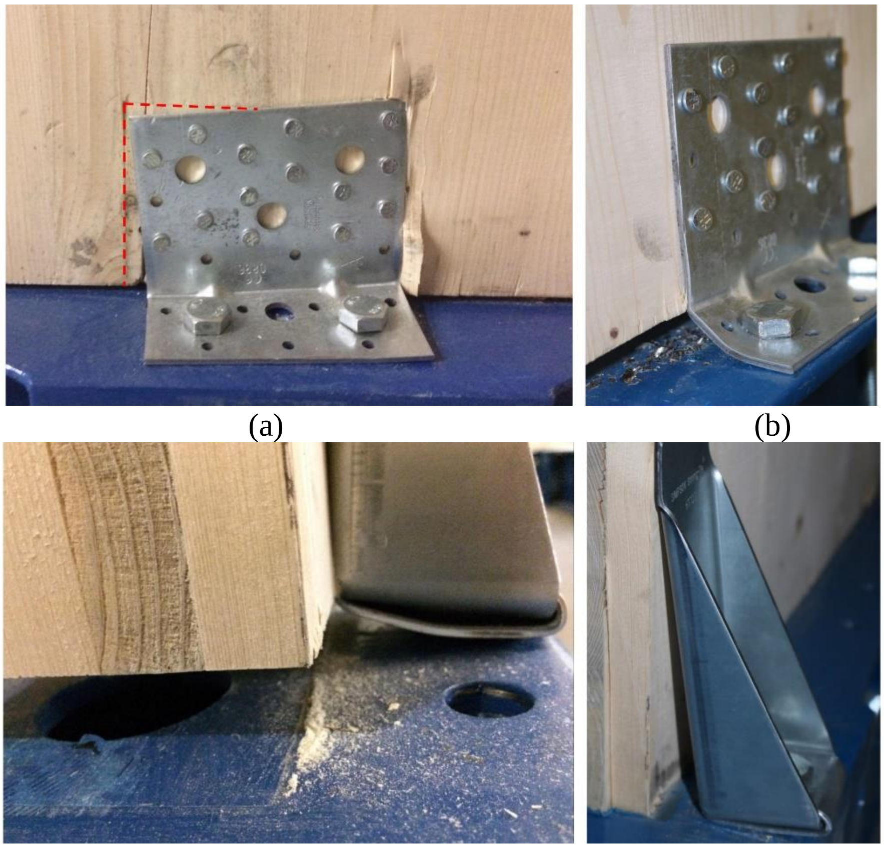
Figure 9. Rocking of the building on longitudinal (a) and transverse (b) monotonic tests.

In terms of in-plane walls deformation, as non-metal connectors (angle-brackets and hold-downs) were placed just in relation to the load application, the building suffered a significant lateral translation in internal walls (see Figure 10).

(a) (b)

Figure 10. Translation of the internal walls on (a) longitudinal and (b) transverse tests.

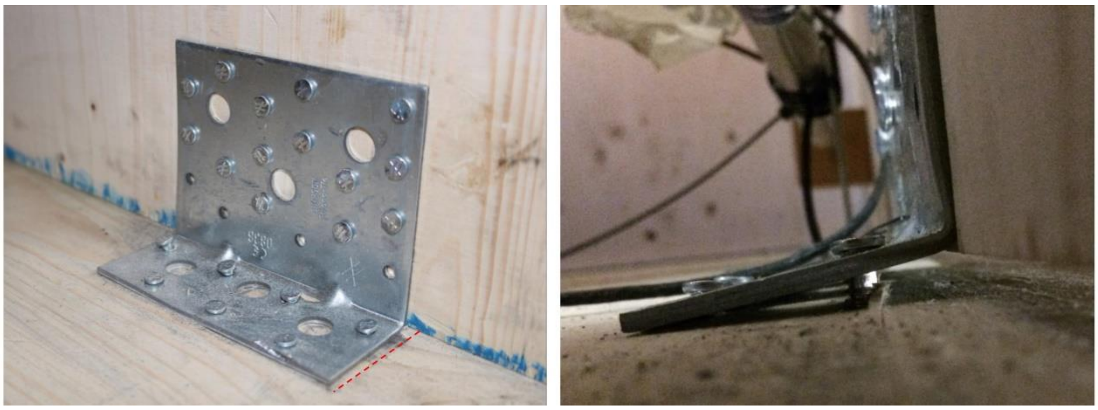
The highest damage observed was located in the metal connectors. For the most part, the connectors have been damaged as a consequence of sliding and rotation (see Figure 11a) and uplift (see Figure 11b and Figure 11c). Moreover, in some cases, AE116 connectors underwent a small uplift, in which the screws that connect the steel structure of the base were virtually undamaged. On the other hand, because the center of the mass is different from the center of stiffness, the hold-downs presented out-of-plane rotation (see Figure 11d). In addition, through the monotonic tests, it was possible to observe damage (plasticization) of the metal connectors on the ground floor. However, the same behavior did not happen on the connectors of the 1st floor due to the limitation of the hydraulic jack of the 2nd floor.



(a) (b) (c) (d)

Figure 11. Damages of the metal connectors: sliding and rotation (a), uplift (b and c) and out-of-plane rotation (d).

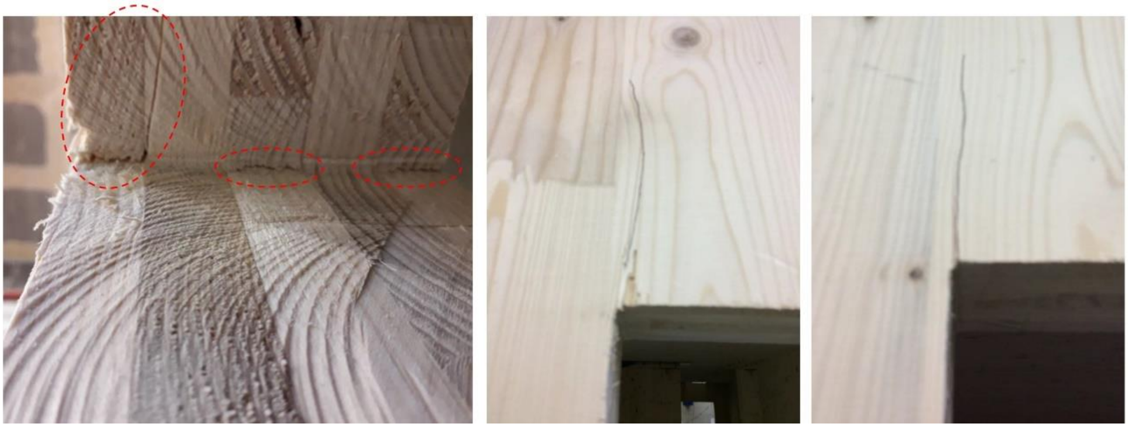
In the case of damages between floors, in transverse direction, the uplift (see Figure 12a) and sliding (see Figure 12b) of the CLT panels of the second floor, in relation to the ones on the first floor, were visible on angle-bracket connectors.



(a) (b)

Figure 12. Sliding (a) and uplift (b) of nails in the connectors AE116.

Finally, as a consequence of a less in-plane stiffness of the transverse direction, in the monotonic test, the lintels over the openings on the ground floor wall in the façade B-B' (see Figure 13) cracked by tension perpendicular to the grain.



(a) (b) (c)

Figure 13. Cracks on top left corner of the openings 1500x2000 (a and b) and 900x2000 (c) on the ground floor wall in the facade B-B' during the monotonic transverse test

4. Conclusions

A non-symmetric 2-storey full-scale structure with large openings was tested for platform-type buildings. The performance, at global and local levels, in each loading direction, was analyzed. In the longitudinal direction, since the structure is stiffer, no significant damage was registered. This can be explained by the technical limitation of the hydraulic jack used in the second floor. In the first monotonic test, under a lateral load in the longitudinal direction of the building, the damage observed was concentrated in the metal connectors (angle-brackets and hold-downs), with signs of sliding, rotation and uplift on the ground floor. In the transverse direction, with short shear walls, more damages were observed. The rotation of the overall structure was visible and the lintels over the larger openings cracked by tension perpendicular to the grain. Finally, yet significantly, in the cyclic test in the transverse direction, the damages were very similar to the monotonic test in the same direction, but in the former, more severe damages were observed in the angle brackets on the first floor. The fundamental frequencies of the CLT building were measured through dynamic identification. Measurements were done, with and without additional masses, before any load tests, and always before and after each test, monotonic and cyclic, performed. However, although the building's load capacity was not reached in the tests performed, it was possible to verify accumulated damage to the building. Under those circumstances, the prediction will be essential in the implementation of the pushover method in CLT structures, where the goal is related to the application of the N2 method to several study cases.

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