Structural behaviour of log timber walls under lateral in-plane loads

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1. Introduction

The construction of timber houses using logs is an ancient practice in many regions of the globe. Overlapped logs were used, covering the gaps between logs with moss. With the emergence of other construction materials, the use of timber decreased considerably and the log system lost importance. Nevertheless, timber log constructions are still popular in many forest regions of the world, especially in North America and Scandinavia.

One of the main disadvantages of log construction is the lack of sound understanding of the structural behaviour of these structures, in particular, under seismic loads [1], [2]. Log buildings rely on the walls built staking horizontal layers of logs, for resistance to both vertical and horizontal loads. The resistance to vertical loads depends mostly of the contact area between logs and on the compression strength perpendicular to the grain. While, horizontal loads are supported by transverse walls, depending strongly on the friction between slots.

Lateral loads in log shear walls depends on the (1) interlocks between logs, (2) wood or steel dowels, (3) vertical through bolts and anchor-bolts, and, (4) frictions between logs due to vertical loads [1]. However, current codes only consider the influence of dowels and vertical through bolts [3], as a result of the significant variability and inexistence of accurate models for the other resistance mechanisms.

In this paper, the resistance of a standardized log construction technology considering all mentioned mechanisms (Figure 1) is evaluated experimentally. Wall panels under vertical and horizontal loads are tested and, lastly, a timber log house is studied under seismic loading.

2. Timber logs

The basic component of this system is the log obtained from lamellas (40 mm) glued face to face, representing an example of vertical glulam, as defined in EN 386:2001 [4]. Three thicknesses are available for the logs: 80 mm (2 lamellas), 120 mm (3 lamellas) and 160 mm (4 lamellas). Notches are made in the top and bottom surfaces of the logs. Those notches increase the interlock and the friction between horizontal layers of logs. Figure 2 presents log cross-sections available on the Rusticasa system.

Lamellas are made of Scots pine (*Pinus sylvestris* L.), bought from the Scandinavian supplier with the minimum requirement to belong to Quality Class VI (or Class C under the new designation), according to [5]. In other words, lamellas are bought based on a visual classification for non-structural applications.

Using NP EN 1194:1999 [6] it is possible to predict the global behaviour (log) based on the mechanical properties of the lamellas. However, in this case, no reference values were known for the lamellas. Therefore, an experimental analysis of the logs under compression perpendicular to the grain and bending was performed, following [7]. The experimental campaign undertaken, the tests results obtained and their analysis can be found in [8] and [9].

3. Connection between the first log and foundation

In timber log constructions, connections between the first log and the foundation are normally achieved through anchor bolts using holes, spaced 120 cm on average, using oversized to facilitate construction. Anchor bolts lose tightness as the log shrinks due to drying and anchor bolt nuts may be inaccessible, thus they cannot be tightened later in the life of the structure [10]. In the Rusticasa system, the connection between the first log and the foundation is made using an angle connector (BMF 40314), every 150 cm, with three screws (5x50 mm) in the timber side and two metal anchors (M8) fixed to the concrete, as shown in figure 3.

Applying the expressions of Eurocode 5 [11] section 8, a value of 3,57 kN is obtained for the resistance of the connections for both directions (parallel and perpendicular to the log axis). This value refers only to the resistance of the connection on the wood side, assuming that the connection device-foundation must be designed according to an appropriate overstrength.

Two types of cyclic tests were performed to evaluate the behaviour of this connection. Using three specimens for each type, the connection was submitted, in the wall plane, to shear (Figure 4a, loaded in the direction of the log axis) and to tension (Figure 4b, loaded in the direction perpendicular to the log axis).

For both kinds of tests, a quasi static cyclic loading procedure in accordance with EN 12512:2001 [12] was assumed. For the shear tests complete cycles were used (Figure 5) while half cycles (only in the tension side) were adopted in the tension tests (Figure 6).

Figure 7 presents the experimental load-displacement curves obtained in the shear tests. The first shear test was not considered because important rotation of the specimen occurred around the connection axis, due to a misconceived test layout. After the improvement of the test layout (figure 4a), shear tests were carried out applying pure shear to the connection, as required.

The results obtained from both tests (Sh1 and Sh2) show a good agreement. They demonstrate good ductility and an important capacity of those connections to dissipate energy under shear. Load increases with the amplitude of the cycle while stiffness decreases. As characteristic of timber joints, considerable pinching is observed. The response is not symmetric. The impairment of the strength is low, under 10% in the compression side (negative) and less than 5 % in the tension side. This difference is due to the fact that the first load step is in tension and therefore the response in the compression side is affected by the pinching effect.

Figure 8 shows the experimental load-displacement curves obtained from the three tension tests performed. Analyzing in detail the results obtained leads to the conclusion that 15 mm of deformation, in agreement with EN 26891:1991 [13], determines the maximum resistance of the connection. The results obtained from the three specimens are consistent, apart from slight variations between the experimental values achieved. In terms of maximum load, this value increased with the cycle amplitude until 15 mm, after which there was a significant impairment of the strength. The same conclusions can be extended to the stiffness experimental values achieved. In particular, a major reduction of the stiffness value was measured in the last cycle amplitude (20 mm), between the first and the third cycles.

4. In-plane behaviour of log-to-log

The in-plane behaviour of timber log walls is ensured by the friction forces developed in the notches existing in the top and bottom of the logs and by the interception between orthogonal walls. The Rusticasa system defines as maximum distance between two consecutive interceptions: 4 meters for 80 mm walls, 6 meters for walls with 120 mm and 8 meters in the case of 160 mm walls. Those interceptions can be of two types: two exterior walls (halved joint) or one exterior wall with an interior wall (dovetail joint), Figure 9.

In accordance with Eurocode 5 [11], friction cannot be regarded as a resistant mechanism, despite its importance in this kind of timber structural system. Therefore, the in-plane resistance of log walls is determined based on the compression perpendicular to the grain and shear stresses developed at the interceptions between walls. In fact, in the halved joints there are both compression stresses, perpendicular and parallel to the grain, but, as the second is higher, it is the former that governs the resistance. Then the resistant capacity offered by the intersection of two walls can be quantified as:

$$R_h = \min\begin{cases} f_{c,90,d} \times A_{r,comp} \\ f_{v,d} \times A_{r,shear} \end{cases} \tag{1}$$

where $f_{c,90,d}$ is the design value of compressive strength perpendicular to the grain, $f_{v,d}$ is the design value for the shear strength, $A_{r,comp}$ and $A_{r,shear}$ represent, respectively, the contact area where strengths of compression perpendicular to the grain and shear can develop.

In addition, with the objective to study the friction forces developed in the notches existing in the top and bottom of the logs, an experimental campaign was carried out using specimens comprising 5 overlapped logs of 120 mm.

Four groups composed by 2 specimens, were tested under different values of the vertical pre-compression (10 kN, 30 kN, 50 kN and 70 kN) while a quasi static cyclic horizontal displacement (Figure 10) was implemented in the top of the specimen, in accordance with the recommendations of EN 12512:2001 [12].

The test setup and the instrumentation used are similar to the ones used to evaluate the in-plane behaviour of full-scale log walls that will be presented in the next section of this paper. The unique difference is that the walls used here did not have interceptions with orthogonal walls, being simply made of 5 overlapped logs.

A summary of the test results is presented in Table 1, namely, in terms of maximum load values in compression (F_{max}^{-}) and tension (F_{max}^{+}) and the equivalent viscous damping ratio (ν_{eq}). The results obtained demonstrate the symmetric response of the walls and the very high values of the equivalent viscous damping ratio that can be achieved. However, it is important to notice that massive dissipation of energy is due to large displacements and based on friction resistance mechanisms. Results obtained for the maximum load applied show a linear correlation (y=0,3389x+2,2685, with a R^2 of 0,9979) with the vertical pre-compression value.

5. In-plane behaviour of log walls

The main objective of this work is to evaluate the in-plane behaviour of timber log walls subjected to lateral (horizontal) actions. To achieve that, and as conclusion of the several precedent studies presented, an experimental campaign composed of full-scale log walls was performed. Two distinct transversal stiffness (wall type 1 and type 2), two vertical pre-compression values (10,1 kN and 48 kN) and the influence of the slenderness of the wall (6,25 and 11,25) were studied. The difference between wall type 1 and wall type 2 is that, in the first angle connectors were used to fix the first log to the foundation. Wall type 2 did not have this connection, but the two short orthogonal walls used to simulate connection between exterior log walls were fixed to the steel frame located in the base of the specimen (log wall). The values of pre-compression adopted correspond to the quasi-permanent value of loads acting in a wall of a house with ground-floor and ground-floor plus one floor, respectively.

For each possible combination of the variables under study, one monotonic and two cyclic loading tests were performed. In the total 13 walls were tested, 4 under monotonic and 9 under quasi-static cyclic loading (Table 2).

Monotonic tests aimed to analyze the failure modes and defining the limits of the elastic displacement needed to define the cyclic procedure according to EN 12512:2001 [12]. Cyclic tests permit the quantification of resistance and its reduction after loading cycles. In addition, they allow the capacity to dissipate energy to be assessed and ductility to be quantified.

In the first test carried out, (W1_1), monotonic loading of a wall type 1 under a vertical compression of 10,1 kN, a displacement of 50 mm at the top of the wall was applied with a constant movement of the hydraulic head of 0,03 mm/s. In the next tests, and because in those conditions tests take too much time, it was decided to apply 100 mm displacement on top of the wall through a constant rate of 0,06 mm/s.

All tests performed, monotonic and cyclic, were composed of a preliminary step aimed at ensuring the adequate contact between logs and removing eventual voids. This step consists in applying the vertical compression level in 3 minutes (56,1 N/s and 266,67 N/s), keeping then the load value for 3 minutes after which the wall was unloaded within 3 minutes. This process was repeated 4 times for each wall. Total vertical displacement of the wall and relative vertical displacement between logs were recorded during this preliminary step for further analysis. After that, the vertical compression level was applied in 3 minutes and then kept constant during the implementation of the horizontal displacement history on the top of the wall. This horizontal displacement history was defined according to [12] using the elastic limit displacement obtained in the corresponding monotonic test, performed previously.

Figure 11 shows the test setup and instrumentation adopted in the case of the walls with 75 cm height. Seven displacement transducers were used to measure: the horizontal slip between each log (4), the horizontal displacement on the top (1) and bottom (1) in the front of the wall and the horizontal displacement on the top of the back of the wall, near the hydraulic jack in charge to implement the displacement history.

In the case of the 135 cm height wall test (Test W1_7), a different configuration of the transducers responsible for registering the horizontal slip between logs had to be adopted.

5.1.Analysis of the tests results

The results obtained in the monotonic tests (Figure 12), shows that there is no difference between wall type 1 (Test W1_1 and Test W1_4) and wall type 2 (Test W2_1 and Test W2_4). In other words, whether or not the wall is fixed to the foundation through an angle connector (BMF 40314), influence on the global behaviour of the wall is not significant.

Experimental results are expressed through horizontal load (F) *versus* shear strain (ϕ) curves of the wall. Here, the shear strain is given by the ratio of top displacement to the wall heigh. Results show that the behaviour of the walls tested under monotonic loading depends on the level of vertical compression. A higher level of vertical pre-compression causes an increase of stiffness and in the horizontal load corresponding to the initial displacement as consequence of the enhancement of the initial friction (Figure 12). However, the maximum load is fairly constant demonstrating that the ultimate resistance of the walls is depending on the transversal stiffness.

The same conclusions cannot be extended to the results obtained in cyclic tests. The walls of type 2, with orthogonal short walls fixed to the foundation, showed a better performance under cyclic horizontal displacement. In both cases, walls types 1 and 2, the value of vertical pre-compression is reflected in the resistance to lateral wall (Figures 13, 14, 15 and 16). The response obtained on the cyclic tests follows the behaviour registered under the monotonic loading. In the tests on walls fixed to the foundation through the first log (type 1), maximum load values of F_{max}^{+}=24,49 kN and F_{max}^{-}=-31,81 kN were obtained for a vertical pre-compression of 10,1 kN, and F_{max}^{+}=44,67 kN and F_{max}^{-}=-48,83 kN were registered for a vertical pre-compression of 48 kN. In the case of the walls type 2, maximum force values of F_{max}^{+}=28,82 kN and F_{max}^{-}=-42,81 kN were measured for a vertical pre-compression of 10,1 kN and F_{max}^{+}=50,84 kN and F_{max}^{-}=-54,72 kN were obtained for a vertical pre-compression of 48 kN.

The experimental horizontal load-shear strain curves obtained in the cyclic tests demonstrate the capacity of the timber logs walls to dissipate energy in both directions.

Increasing the slenderness of the timber log walls, from 6,25 to 11,25 (Test W1_7), results in a significant reduction of the wall lateral resistance, Figure 17. In this last test the maximum force values recorded were F_{max}^{+}=19,63 kN and F_{max}^{-}=-21,69 kN while the similar wall with a slenderness of 6,25 presented F_{max}^{+}=44,67 kN and F_{max}^{-}=-48,83 kN. Nevertheless, experimental results obtained indicate a good dissipative behaviour of the slenderer wall, characterized by horizontal load-shear strain curves symmetrical and quite large.

Based on the experimental results obtained on the cyclic tests, and following EN 12512:2001 [12], equivalent viscous damping ratio by hysteresis (ν_{eq}) was calculated, Table 3. Comparing the experimental ν_{eq} obtained, it is possible to conclude that wall type 2 dissipates more energy than wall type 1. Moreover, this dissipation increases with the vertical pre-compression and slenderness of the wall.

In terms of failure mode, no significant difference exists between both wall types. Neither the value of the vertical pre-compression nor slenderness seems to present any significant influence. Analyzing in detail the horizontal slip between the logs measured during all tests performed, it is obvious a linear variation on that value with the height of the logs.

Finally, the stiffness of the walls was calculated using a methodology adapted from the one suggested by ISO/FDIS 21581:2010 [14]. For that, it was assumed that the slope of the third envelope in the elastic range corresponds to the stiffness value (K) according to Figure 18 and expression (2).

$$K = \frac{F_B - F_A}{D_B - D_A} \tag{2}$$

The results obtained for the stiffness of the walls are presented in Table 4. It is thus possible to conclude that the stiffness increases with the vertical pre-compression and decreases with the slenderness of the wall.

6. Case study

Aiming to analyze the seismic behaviour of timber log constructions, a typical log house using the construction system studied above was used as case study. The seismic performance of the timber log house was investigated using two simplified methods. The first method, according to [15], distributed the seismic forces by the walls depending on their area of influence, while the second method distributed the seismic forces over the walls in proportion to its stiffness.

The building was considered located in Portimão (Portugal), on a ground type A, characterized through the plants and section presented in Figure 19.

6.1.Distribution of the seismic forces according to the area of influence

The utilization of this method requires the verification of the criteria for regularity in plan and height defined in Part 1-1 of Eurocode 8 [16]. The analysis of the conditions listed concludes that it is possible to classify the building as "regular in plan and height", stressing, however, the non-continuity of wall 6 and part of wall 8 in the floor, existing only at the level the ground-floor.

The weight transmitted by the building (W=362,43kN) would include the self-weight of the structure and the imposed loads, corresponding to the quasi-permanent value of loads defined by [17]. As the natural period of the structure is unknown, because no preliminary dynamic analysis was performed, and the Eurocode 8 [16] does not propose any expression to predict it, the maximum acceleration value of the response spectrum ($S_d(T) = 3{,}125\ m/s^2$) would be assumed. In consequence, a value of 115,45 kN was obtained for the base shear force (F_b) acting independently in each horizontal direction.

This force will be distributed over several floors, constituting the seismic forces acting on various levels and, consequently, shear plans, through the expression:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \tag{3}$$

where F_b is the seismic base shear force, and z_i and z_j correspond to the heights of the application of horizontal forces regarding masses m_i and m_j, relatively to the level of application of the seismic action (foundation).

Applying (3), values of 73,13 kN and 42,32 kN were obtained for the roof and floor level, respectively. Therefore, the total seismic force (T_d) to be applied in the roof level is 73,13 kN and a total of 115,45 kN must be applied at the ground floor level.

The distribution of those seismic forces through the walls following the method of distribution according to the area of influence is presented in Table 5.

For each wall there are *n* interceptions with orthogonal walls, and consequently, the design value of the acting stresses of compression perpendicular to grain ($\sigma_{c,90,d}$) and shear (τ_d) will be, correspondingly:

$$\sigma_{c,90,d} = \frac{T_d}{n \cdot A_{r,comp}} \tag{4}$$

$$\tau_d = \frac{1{,}5 \cdot T_d}{n \cdot A_{r,shear} \cdot 2} \tag{5}$$

where T_d represents the amounts of the seismic load acting in the wall, *n* is the number of interceptions that the wall has with orthogonal walls, $A_{r,comp}$ is the area under compression perpendicular to the grain and $A_{r,shear}$ is the area subjected to shear.

The safety verification imposes that the following conditions must be verified:

$$f_{c,90,d} \geq \sigma_{c,90,d} \tag{6}$$

$$f_{v,d} \geq \tau_d \tag{7}$$

where $f_{c,90,d}$ and $f_{v,d}$ are the design values of the compression strength perpendicular to grain and shear strength, in that order.

The design value of the acting stresses in compression perpendicular to the grain and shear for each wall are presented in Table 6. On the other hand, the shear forces at the foundation level will be resisted by the anchors. In the system under study, the shear resistance of each connection between the first log and the foundation ($F_{v,Rd}$) is 3,57 kN. Considering the anchors represented in Figure 19(a), it is possible to quantify the shear resistance guaranteed by the connection between each wall and the foundation. As shown by Table 7, only wall 5 presents safe connections to the foundation.

Finally, the overall stability of the building must be investigated comparing the acting (M_{Sd}) and the resistant (M_{Rd}) values of the moment with respect to point P (Figure 19(d)), corresponding to the seismic loads applied (T_{Roof} = 73,13 kN and T_{Floor} = 115,45 kN) and to the weight (W=362,43 kN), respectively. It is possible to conclude that safety is largely verified, as the resistant moment is 1395,36 kN.m and the acting value is only 693,66 kN.m.

6.2.Distribution of the seismic forces according to the stiffness

Considering that the distribution of the seismic forces according to the area of influence of each wall is a simplistic methodology, it was decided to repeat the previous analysis assuming that the distribution of the seismic forces over the walls is a function of their in-plane stiffness.

In order to determine the in-plane stiffness of each wall, a numeric modelling was performed applying the finite element method, thus using the commercial package SAP2000 [18]. In a first step, the test results presented in section 5 were used to calibrate the numeric models. Then, every wall of the structure under analysis was modelled to quantify its in-plane stiffness.

In the modelling, and in accordance with the provisions of Eurocodes, the contribution of friction between the logs to resistance was not considered. Therefore, the main resistant mechanism of those log walls is the confinement afforded by the perpendicular walls. For this reason, the connection between the logs was simulated through *NLLink* elements, of which only the ones located at the interceptions with perpendicular walls (interceptions through halved joints) presented stiffness in the axis U2, K_{U2}, (shear in the plane of the wall), having the remaining the unique function to fix the logs vertically (only axial stiffness in the vertical direction of the wall), Figure 20.

In the calibration process, the results of the tests W1_3 (F_v=10,1kN and slenderness=6,25), W1_6 (F_v=48kN and slenderness=6,25) and W1_7 (F_v=48kN and slenderness=11,25), were used. From the K_{U2} values obtained, and considering that they vary linearly with vertical force (F_v) and slenderness (λ), the following expression was established:

$$K_{U2} = 35{,}303\, F_v - 62\, \lambda + 1343{,}94 \tag{8}$$

Using the calibrated models, different horizontal load values were applied to obtain the corresponding load-displacement curve features of each wall, thus quantifying their in-plane stiffness through the slope of those curves (Table 5). The seismic forces applied at each level (ground-floor and roof) were then divided by each wall in proportion to their in-plane stiffness, in both seismic directions (direction X and Y), Table 5.

The safety verification to compression perpendicular to grain ($\sigma_{c,90,d}$) and shear (τ_d) of the walls shows that only wall 7a is not safe with respect to the first stress, and walls 3b, 7a and 15a are unsafe regarding shear stresses (Table 6). As previously done, the safety of the shear connection between each wall and the foundation was verified (Table 7). Employing this methodology to distribute the seismic forces over the resistant walls, according to their in-plane stiffness, lead to the conclusion that six walls are unsafe in terms of their connection with the foundation.

7. Analysis and improvement

The numeric analysis performed, considering a case study representing a typical log-house built according to the system evaluated, showed that walls are unsafe with respect to compression perpendicular to the grain and shear under seismic loads. The reason is that, unlike other systems, the system marketed by Rusticasa does not provide the connection between different logs, easily achieved through the introduction of metal rods. Therefore, in the Rusticasa system, the entire resistance of the log wall is ensured by the connections between orthogonal walls (crossings).

In order to improve the building system under study, the introduction of metal rods drilling at least three logs each is suggested. For example, if metal rods of 10 mm drilling three logs were used, a shear resistance value of 5,95 kN would be introduced by each bolt per shear plane, value sufficient to verify the safety against compression perpendicular to grain and shear in terms of the in-plane behaviour of the log walls.

On the other hand, the connection between the wall and the foundation must also be improved. Despite the methodology that considers the distribution of the seismic forces according to the in-plane stiffness of the walls to lead to more favourable results, the numeric analysis performed reveals the lack of resistance of this connection.

In alternative to the anchor plate employed by the system analyzed, an anchor bolt could be used to provide the connection between the first log and the foundation (Figure 21). As an example, an anchor bolt with 12 mm diameter, in each position of the anchors presented in Figure 19(a), would be sufficient to establish safety of every wall under the seismic loads considered.

8. Conclusions

Despite this being a traditional system used in timber constructions, Rusticasa produces a construction system based on timber log that, stemming from certain particularities, requires a series of experimental and numeric studies to apply to the European Technical Approval [19].

In this work, the main resistant mechanism of the timber log walls was analyzed, in particular the ones concerned with the in-plane resistance to horizontal loading. Timber logs used to make the walls were characterized and both connections between logs and also between walls were studied by means of numerical and experimental studies. Considerable friction stresses are developed in the connection between logs, which are, as expected, function of the vertical pre-compression level.

Special attention was paid to the connection of the walls with the foundation since such connection is manufactured by Rusticasa in a quite unusual manner. This connection was tested and its influence on the global behaviour of walls subjected to in-plane displacement was assessed.

The connection between orthogonal walls, namely the interlock between the logs of exterior walls is the main resistant mechanism of timber log walls under in-plane horizontal loads. Inside the halved joint used to materialize this intersection, shear stresses as well as compression stresses perpendicular and parallel to the grain occur. In the tests performed on full-scale walls, the localized failure was obtained always by compression perpendicular to the grain. Such tests aimed at evaluating full-scale timber log walls under different vertical pre-compression levels, in addition to distinct connection between the first log and the foundation, and also two types of stiffness of the orthogonal walls, besides assessing the effect of the slenderness of the wall.

The experimental results obtained show a good capacity of these walls to dissipate energy, without any impairment of strength being the monotonic response normally enveloped to the behaviour obtained on the cyclic tests. The connection between the first log and the foundation, as far as the wall geometry evaluated is concerned, is not important to the global behaviour, which is function of: a) the stiffness of the orthogonal walls; b) vertical pre-compression value; and, c) wall slenderness.

Assuming a case study, the effectiveness of the simplistic method of distribution of the horizontal loads (seismic) over the walls according to their area of influence was assessed. The numeric analysis performed shows that this distribution must be based on the in-plane stiffness of the walls. Moreover, this analysis also showed that an inter-connection between logs should be implemented. According to actual standards and codes, friction can not be considered as a resistant mechanism and therefore, the in-plane behavior of the system analyzed is totally ensured by the connection between orthogonal walls, where compression perpendicular to the grain and shear stresses are developed. Results also indicated that the connection between the first log and the foundation must be improved.

Consequently, improvements to the Rusticasa system were suggested for the inter-connection between logs and also the connection of the first log to the foundation. Those suggestions are based on the test results and attendant to the conclusions drawn and supported by the results of the numeric analysis undertaken.

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10. References

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