

SHAKING TABLE TESTS OF A TWO-STOREY LOG HOUSE

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Abstract: This paper presents the findings of an experimental campaign that was conducted to investigate the seismic behaviour of log houses. A two-storey log house designed by the Portuguese company Rusticasa® was subjected to a series of shaking table tests at LNEC, Lisbon, Portugal. The paper contains the description of the geometry and construction of the house and all the aspects related to the testing procedure, namely the pre-design, the setup, instrumentation and the testing process itself. The shaking table tests were carried out with a scaled spectrum of the Montenegro (1979) earthquake, at increasing levels of PGA, starting from 0.07g, moving on to 0.28g and finally 0.5g. The log house did not suffer any major damage and remained in working condition throughout the entire process. The preliminary analysis of the overall behaviour of the log house is also discussed.

Keywords: log house, timber, shaking table, seismic performance, numerical modelling

1. Introduction

Log house construction is one of the foremost practices of building with wood that continues to exist even in present times, although to a much lesser extent. In the rudimentary stages, log houses were simple box-like enclosures with minimal openings and consisted of inter-connected walls composed of horizontally stacked logs. With time, notching techniques and log profiles have been refined. There have been advances in carpentry joints and the development of new and enhanced mechanical fasteners has facilitated more efficient adjacent log connections, sill log-foundation connections as well as cross-wall connections. The upshot of these developments is that modern log houses are stronger and more durable structures, with improved stability. However, despite the fact that the practice of building log houses has existed since a very long time, the amount of data available on their seismic behaviour and load resistance mechanisms is limited (Hirai et al., 2004; Vasconcelos et al., 2011). As an example, no specific design guidelines for log houses have been outlined in the Eurocodes. The Log Building Standards published by the International Log Builders Association (2000) only deals with common construction for log walls, notches, joists and roof systems but does not outline any seismic or wind design procedure.

For log shear walls, resistance to lateral loads comes from the interlock between logs, wood or steel dowels, vertical through-bolts or anchor bolts and friction between logs due to vertical loads (National institute for land and infrastructure management et al., 2003). In order to ensure adequate seismic resistance, the sill-log must be firmly connected to the foundation, so that uplift is eliminated and friction and interlocking mechanisms can come into play (Hahney, 2000). Current building codes only consider that the compression perpendicular to the grain and shear stress at intersections between walls are responsible for the in-plane resistance. Since the exact contribution to resistance by friction and interlock between logs cannot be quantified with current knowledge, friction is not considered as a mechanism (Branco and Araujo, 2012). The lateral load transfer mechanism and the energy dissipation characteristics of handcrafted log walls with different configurations has been studied by subjecting them to quasi-static cyclic and non-linear static pushover tests (Popovski, 2002). In the experiments by Graham et al. (2010), the behaviour of log shear walls under racking loads was analyzed and the log walls were found to have a higher resistance than conventional light-frame shear walls. With the view of increasing the understanding of the seismic performance of log houses, a combination of experimental and numerical approaches was adopted in this research campaign.

2. Prior work

A number of experimental tests were carried out as part of the procedure to secure the European Technical Approval (ETA) (2002) of the log house system adopted by Rusticasa®. During this process, the individual structural components of the building system were studied and the lateral load resistance mechanisms seen in Figure 1 were evaluated.

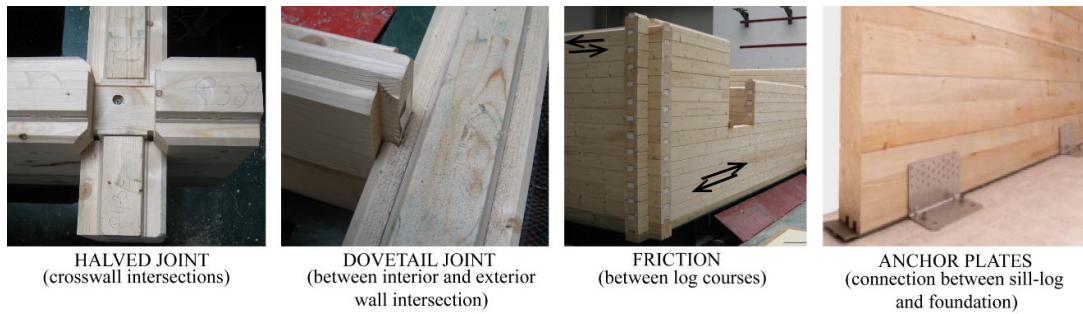


Figure 1: Lateral load resistance mechanisms in log shear walls

The first set of tests was performed on the logs to determine the values of compression strength perpendicular to grain and bending strength (Branco and Cruz, 2008) according to the procedure outlined in EN 408:2010. This was followed by a series of shear and tensile tests under cyclic loading to investigate the behaviour of connections between the sill log and the foundation. In the system angle connectors spaced 1.5m apart are used to connect the sill log with the foundation. Each connector (BMF 40314) has three screws (5x50mm) embedded in the log and two metal anchors (M8) embedded in the foundation. The resistance of the connection was calculated to be 3.57kN as per the expressions given in Eurocode 5 (EN 1995-1-1:2004). The shear tests revealed that the connections possessed good ductility and energy dissipation capacity. From the load-displacement graph obtained during the tensile tests, it was found that the maximum load value increased in accordance with the cyclic value until a displacement of 15mm, after which it decreased.

In order to study the effect of friction forces between adjacent logs, wall specimens consisting of five overlapped logs of 120mm thickness were used. The specimens were subjected to different values of vertical pre-compression (10, 30, 50 and 70kN). A relationship between the maximum load(y) and pre-compression value(x) was established: $y=0,3389x+2,2685$ (kN) with an R^2 value of 0,998. The high levels of energy dissipation were attributed to the friction resistance.

The next step was to study the in-plane behaviour of log walls subjected to lateral loads. Full-scale wall specimens with cross wall intersections were subjected to monotonic and cyclic tests. The transversal stiffness, vertical pre-compression, slenderness ratio and foundation connection were varied. Based on the results of the monotonic tests, it was found that the transversal stiffness influences the ultimate load and that the level of vertical pre-compression is proportional to the stiffness. While the foundation type had no influence on the results of the monotonic tests, it did have an influence on the results of the cyclic tests. The specimens where the cross walls were fixed to a steel frame performed better than the specimens with the traditional foundation connection. The lateral resistance of the wall specimens increased with the vertical pre-compression value and decreased with an increase in the slenderness ratio.

An overall seismic analysis of a typical Rusticasa® log house was carried out based on the results obtained from the experiments conducted previously. The numerical model that was developed pointed out the need for an improved connection system between the logs as well as the connection between the sill log and foundation. Since seismic forces are highest at the base of the log house, it is essential that the connection between the sill log and the foundation is secure (Scott et al., 2005). While previously, the system under study relied only on the orthogonal connections between logs for resistance to loads, the introduction of metal fasteners connecting the logs was advised. Instead of an anchor plate between the wall and the foundation, the use of anchor bolts was recommended (Branco and Araujo, 2012). Both types of foundation connections can be seen in Figure 2.

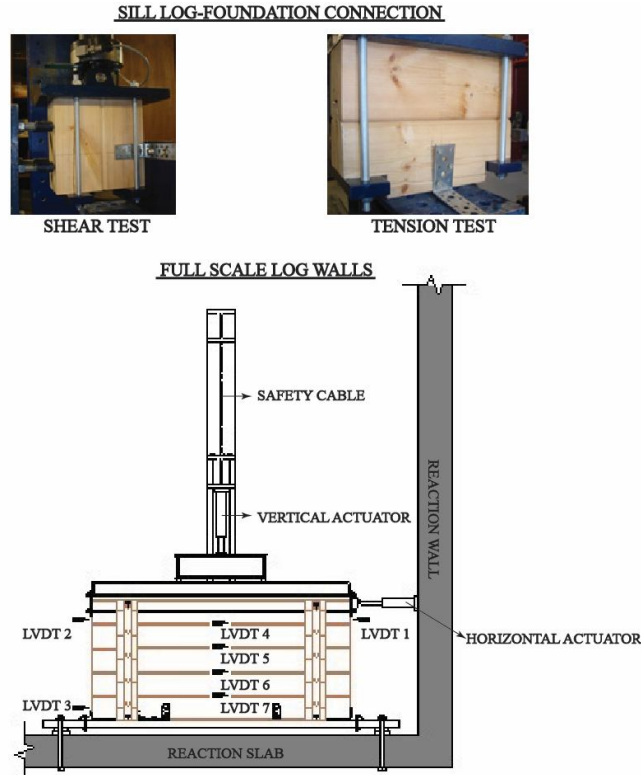


Figure 2: Set-up for tests on individual components of the log house (Branco & Araujo, 2012; adapted)

The present and final research phase addresses the understanding of the global behaviour of the structure when subjected to seismic loads, its dynamic properties and the interaction of the individual components inside the real structure during an earthquake. The test campaign consists of a series of shaking table tests conducted on a full-scale two-storey log house at LNEC (National Laboratory of Civil Engineering) in Lisbon, Portugal.

3. Experimental campaign

3.1 Description of the test building

The log house used in the current research experiment has been designed by the Portuguese company Rusticasa®. The house is characterized by a rectangular plan of size 5.64m x 7.3m and is symmetrical in the longitudinal direction but asymmetrical in the transverse direction. Its height measures 5.28m at the ridge and 4.4m at the edges, forming a gable roof. The logs used in the construction of the walls are made from glued laminated timber of Scots Pine trees (*Pinus sylvestris* L.) and belong to the C24 class of resistance as per EN 388:2009. The mechanical properties of the lamellas were obtained using

EN 1194:1999. The logs form a perfect fit with each other as they are grooved both at the top and bottom. The dimensions of the cross sections of the logs used are 160mm x 160mm and 80mm x 160mm in the outer and inner walls respectively. The floor and the roof beams are superposed by 22m thick Oriented Strand Board (OSB) studded panels in order to ensure rigid in-plane behaviour. Figure 3 shows the 3D structure of the house.

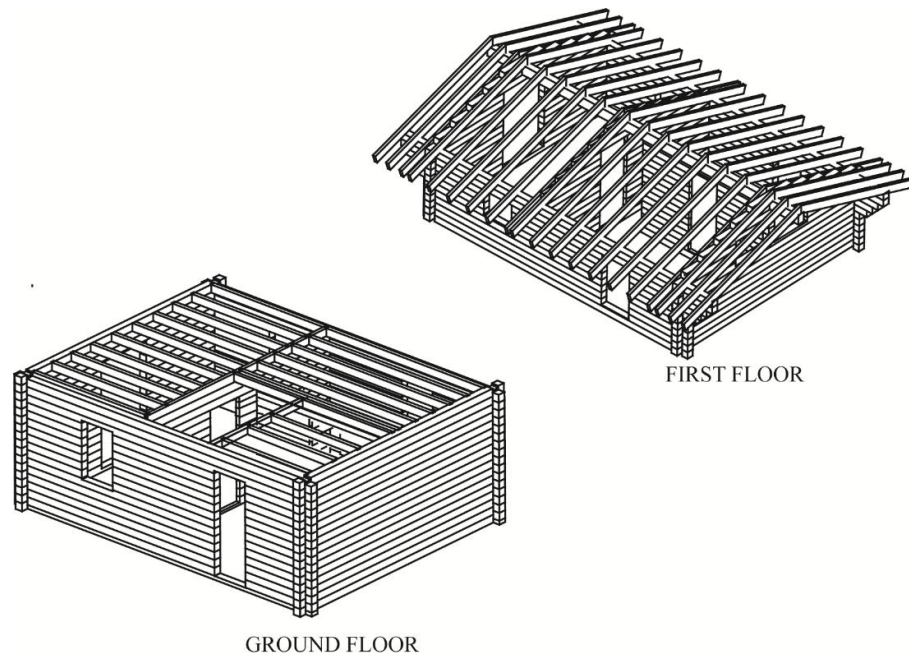


Figure 3: 3-dimensional structure of the log house showing the ground floor and first floor

All the walls of the building have been provided with openings to study their influence on the seismic behaviour of the house. In the case of walls provided with openings on both floors, they are aligned one above the other, either as doors or windows. The ground floor plan can be seen in Figure 4. The external walls in the longitudinal direction (W2 and W4) have identical openings, with a window and a door on the ground floor and two windows in the upper storey. One of the external walls in the transverse direction (W1) has a door provided on each floor while the other one (W3) has two windows provided on the upper floor. Both internal walls (W5 and W6) are provided with a door on each floor. The inner wall in the longitudinal direction is connected with the exterior wall (W3) only along one edge to minimize the effect of wall linkages.

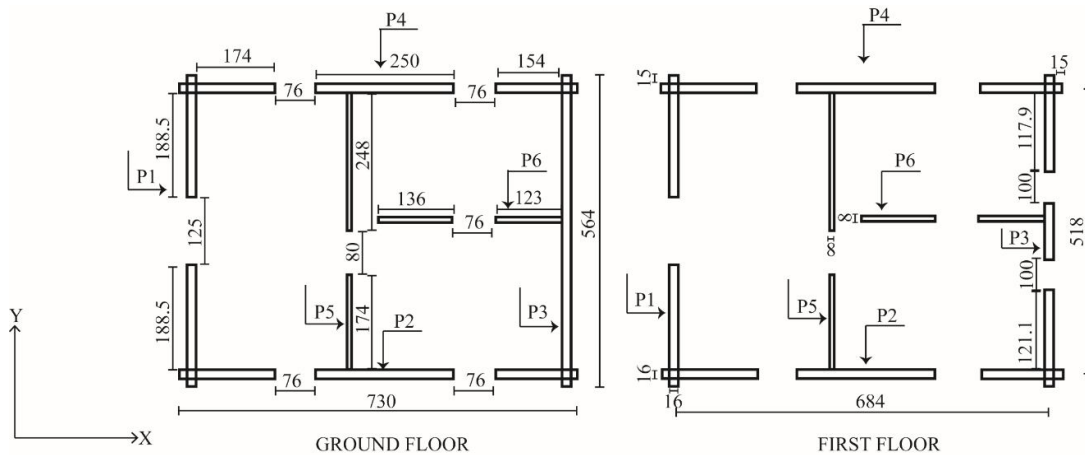


Figure 4: Floor plans of the log house (all dimensions in cm)

Since a typical log-house does not dissipate a large amount of energy, the value of the behaviour factor q was assumed to be 2 taking into account the dissipative action contributed by friction as well (Toratti, 2001). In order to simulate real-life loading conditions, additional masses in the form of four steel platforms weighing 600kg each and 398 steel plates, weighing 7.1kg each, were placed on the inter-storey floor and the roof, respectively.

As in other timber constructions, connections are important in log houses to dissipate energy. Mechanical connectors in the form of screws were provided around the openings, at cross-wall intersections and at the ends of floor and roof beams. The sill logs are connected to the foundation (a steel lattice in this case) through M16 bolts of class 8.8 that are spaced 0.5m apart. The sill log-foundation connection is rigid to ensure that sliding is minimal. Apart from mechanical connections, there are halved joints between two intersecting orthogonal logs in the exterior walls and dovetail joints at intersections between logs of exterior walls with interior walls. Ends of the floor joists are also appropriately notched to form dovetail joints with the log walls. These connections can be seen in Figure 5.

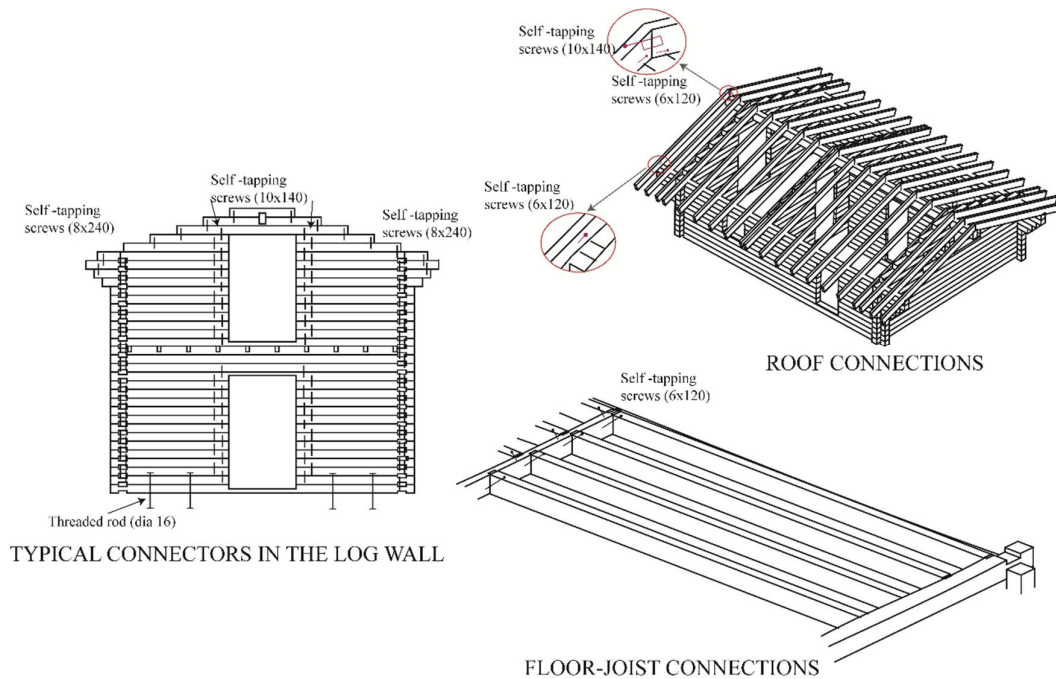


Figure 5: Mechanical connections in the log house (all dimensions in mm)

3.2 Frequency estimation

Prior to the seismic test, it was necessary to make an analytical prediction of the expected behaviour of the building during the tests. The design of the test is the key step in the experimental campaign because the seismic input and the parameters required for phase tuning the shaking table are determined based on the model. The test required an accelerogram that would contain frequencies in the vicinity of the frequencies of interest and the control system of the shaking table had to be able to reproduce the motion of the chosen reference in the range of the frequencies of interest.

A numerical model of the building (seen in Figure 6) was developed in SAP2000 to obtain information on the dynamic properties of the structure. One of the challenges faced when modelling timber structures is the definition of the material. With properties differing in the longitudinal, tangential and radial directions, wood is best defined as an orthotropic material. Assuming the material properties obtained from previous numerical and experimental studies performed by the University of Minho (Branco and Araujo, 2012; Branco and Cruz, 2008), wood was modelled as an orthotropic material with different properties along the three principal axes.

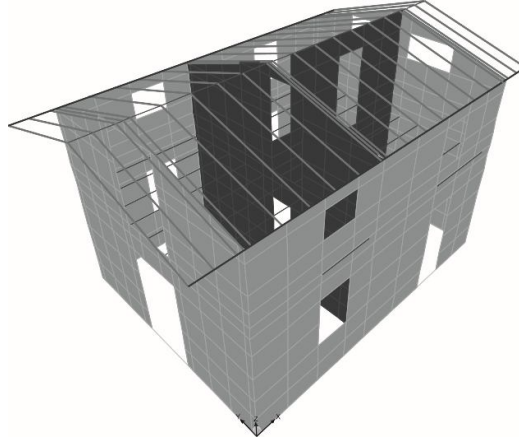


Figure 6: Preliminary numerical model of the log house developed in SAP2000

The logs were represented by shell elements while the floor and roof joists were represented by beam elements. In order to ensure the rigid in-plane behaviour of the floor and roof, diaphragm constraints were assigned to the beams elements existing in those two structural systems. Since the main objective of the model was to get a rough estimate of the modal frequencies, the model was not a sophisticated one. This model neither took into account the effects due to friction nor contribution of the mechanical connections. So the behaviour of the log house model is mainly dependent on the compression stress perpendicular to grain and shear stresses arising at the connections between orthogonal cross walls. The mass of the OSB planks at the inter-storey floor and roof level has been distributed as a uniform load acting on the floor and roof beams, respectively. The additional masses on the roof have been distributed uniformly along the rafters and the additional masses on the inter-storey floor have been uniformly assigned to the floor beams. The value of the modulus of elasticity, E , was assumed to be $2 \times 10^6 \text{MPa}$ in all directions. Moreover, the values of shear modulus G_{13} and G_{23} were assumed to be 20MPa . This was done in order to induce a shear type of behaviour in the house. The value of shear modulus G_{12} was calculated based on the value of the in-plane stiffness (K) obtained in (Branco and Araujo, 2012) and in the geometry (h , L and t) of the wall specimen tested there.

$$G_{12} = \frac{K \cdot h}{L \cdot t} = \frac{1289 \cdot 0,75}{1,54 \cdot 0,12} = 5231 \text{ kN/m}^2 \quad (1)$$

Here, K is the in-plane stiffness (kN/m), h is the height of the specimen (m), L is the length of the specimen (m) and t is the thickness of the specimen (m).

The values of Poisson's ratio suggested by Bodig and Jayne (1982) for the case of softwoods, have been assumed. The material properties assumed for the wood in the numerical model are presented in Table 1 and 2. The mechanical properties assumed for the floor and roof beams are seen in Table 3.

Table 1: Values of mechanical properties differing in the orthogonal directions

Property	12	13	23
Poisson's ratio, ν	0.37	0.42	0.47
Shear Modulus, G (MPa)	5.321	20	20

Table 2: Values of mechanical properties assumed to be constant in the orthogonal directions

Property	Value
Modulus of Elasticity, E (MPa)	2000000
Density, ρ (kg/m ³)	340

Table 3: Values of mechanical properties of the roof and floor beams

Property	Value
Modulus of Elasticity, E (MPa)	11000
Density, ρ (kg/m ³)	420
Poisson's ratio, ν	0.37
Shear Modulus, G (kN/m ²)	690

According to the model, the fundamental frequency of the log house was estimated to be 5.04Hz and corresponded to translational movement in the x-direction (longitudinal axis). The second mode was very close to the first mode with a natural frequency of 5.08Hz (Testoni, 2012). The first five mode shapes can be seen later in Section 3.6, Figure 12, where a comparison is made with the experimental mode shapes.

3.3 Construction of the test building

As the shaking table at LNEC was not large enough to directly accommodate the building, a lattice of steel beams (Figure 7) was used to connect the structure to the table and provide a wider base. The steel lattice consisted of two identical sections of in-plane length 7m and in-plane depth 2.65m firmly connected to the table by means of steel bars of 36mm diameter, making a base of 7m x 5.3m for the building. Each section has two HEA 400 profile beams in the longitudinal direction and three HEA 400 beam profiles

along with two IPE 400 profile beams in the transverse direction. The building was mounted and assembled directly over the steel lattice. All primary structural elements of the building were prefabricated in a factory at Vila Nova de Cerveira and transported by truck over a distance of about 400km to Lisbon. The construction was carried out by three workers and took four days. The stages of assembly can be seen in Figure 8.

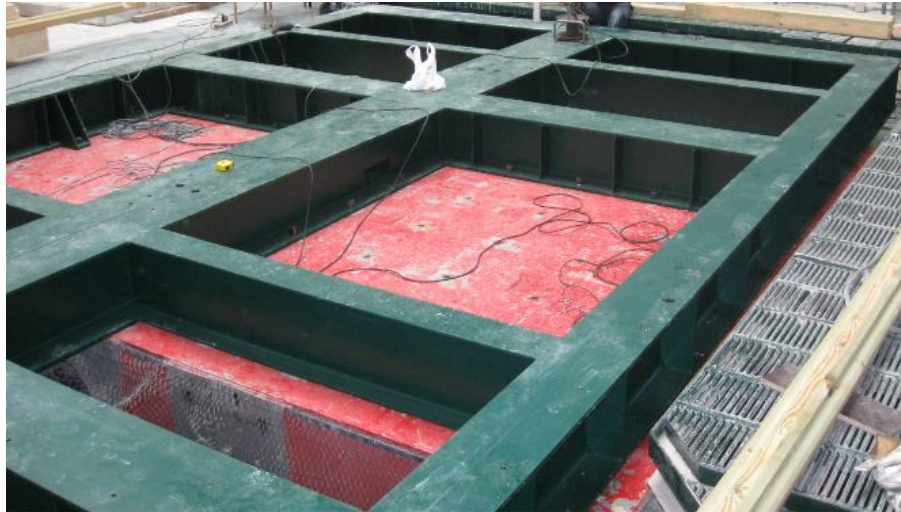


Figure 7: The steel lattice connected to the shaking table

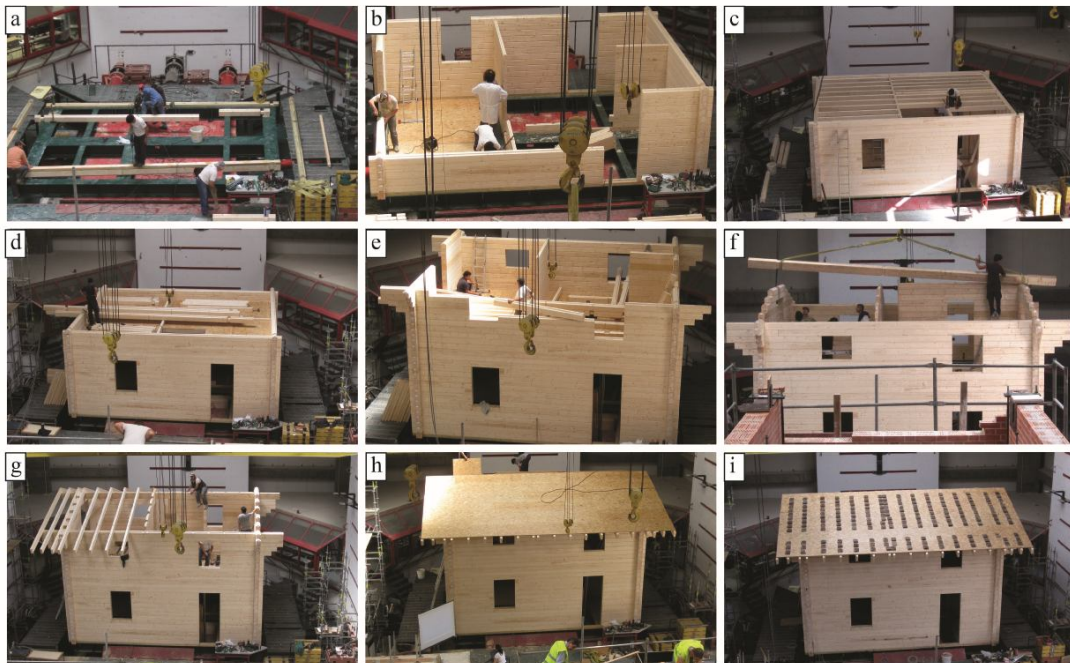


Figure 8: Stages of construction: (a) connecting the sill logs with the steel lattice; (b) laying the ground floor walls; (c) placing the inter-storey floor beams; (d) nailing OSB planks to the floor joists; (e) laying the first floor walls; (f) positioning the ridge board; (g) connecting the rafters; (h) placing the OSB sheaths over the roof; (i) completed house with additional weights

3.4 Instrumentation

Once the house was constructed, the next step was to place the instruments to monitor the accelerations, wall slippage and uplift, shear deformations, inter-storey displacements and forces in the building components. A total of 80 instruments were placed at different points of interest in the building. The instruments were arranged to assess the behaviour of the building in the longitudinal and transverse directions, and focused on observing the behaviour of five wall sections on the ground floor, two in the longitudinal direction and three in the transverse direction. The different instruments used to collect information on the forces and deformations of the building are seen in Figure 9. The data obtained during all the stages of testing right from the tuning of the shaking table to the dynamic identification of the building and the seismic tests were sampled at a sampling frequency of 125Hz.

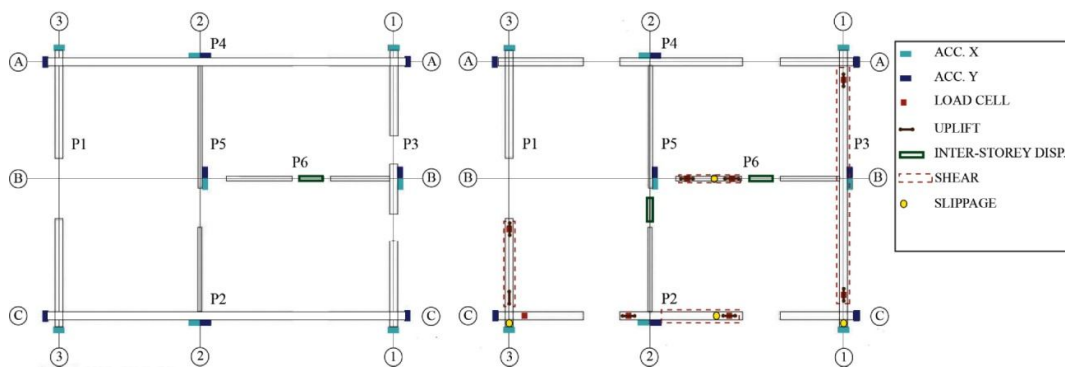


Figure 9: Instrumentation layout

3.5 Shaking table test procedure

Once all measuring instruments were set-up, the following tasks were performed: calibration of the instruments, dynamic characterization tests and shaking table tests. The dynamic characterization tests were of low intensity, while the seismic tests were carried out with increasing levels of Peak Ground Acceleration (PGA). White noise was used as input in the case of the characterization tests. The acceleration time history was characterized by a Gaussian distribution of RMS level of 0.05g and the frequency ranged between 0.1Hz and 30Hz. The purpose of these tests was to calibrate the parameters necessary to control the shaking table and to conduct the dynamic identification of the structure. A first frequency evaluation test was performed to identify the fundamental period, mode shapes and damping of the structure. In this stage, a

fundamental period of $T=0.185s$ ($f=5.389Hz$) was obtained. The mode was a mixed mode, with movement along the longitudinal as well as the transverse directions.

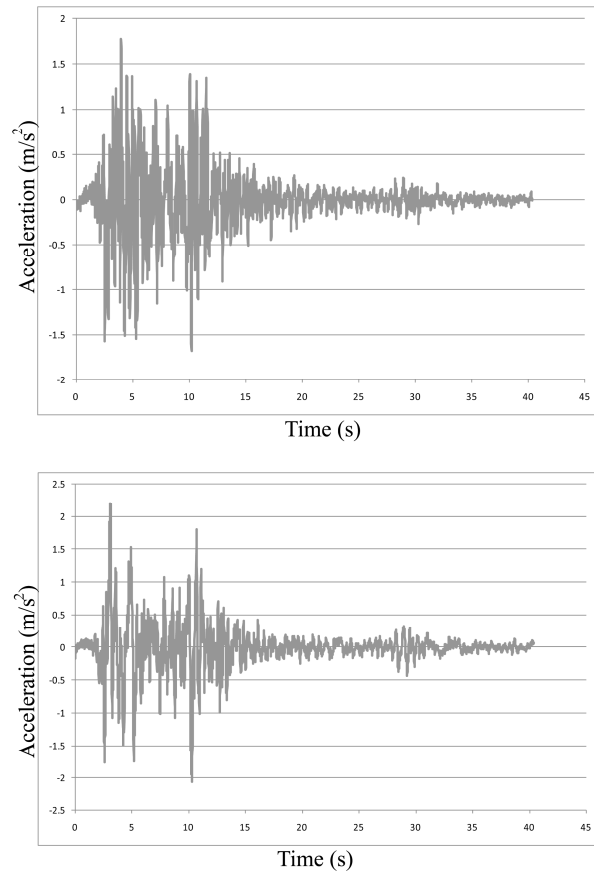


Figure 10: The unscaled accelerograms of the Montenegro earthquake recorded at the Ulcinj- Hotel Albatros station used in the transverse (top) and longitudinal (bottom) directions

The seismic testing procedure that followed was carried out with three different PGA values - 0.07g, 0.28g and 0.5g. To replicate the effect of an earthquake of a longer duration (the Montenegro earthquake has an intense phase of only about 10s), the seismic tests of different intensities were carried out multiple times. Despite this, there was no major damage observed in the structure. After the first dynamic identification test was performed, four shaking table tests were carried out with a PGA of 0.07g. This was followed by another dynamic identification test to determine any changes in the natural frequency of the structure. The PGA was then increased to 0.28g and three seismic tests were carried out until a pronounced sliding of logs was seen in the window of one of the walls of the house. After this, a dynamic identification test was done and the building was subjected to two more seismic tests of 0.28g PGA. Finally two tests

with a PGA of 0.05g were conducted on the log house, which were preceded and followed by dynamic identification tests.

A comparison between the mode shapes and frequencies is seen in Figure 11 as well as Table 4. The fundamental frequency of the log house was determined to be 5.39Hz while the numerical model predicted a value of 5.04Hz for the first mode and 5.08Hz for the second mode.

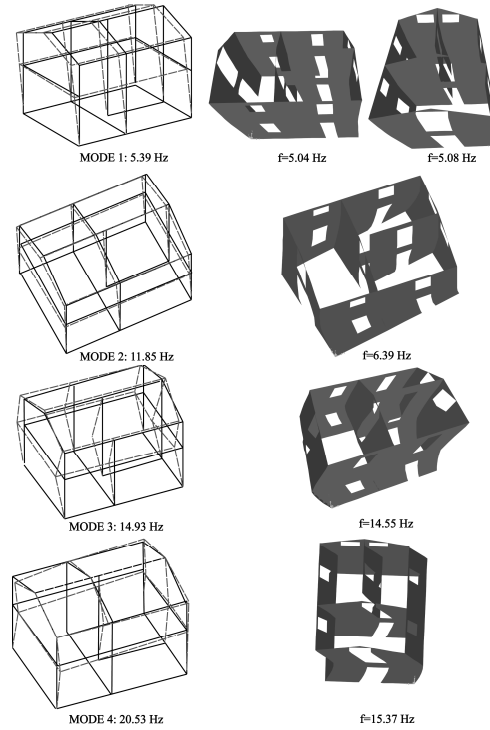


Figure 11: Outlines of the mode shapes obtained during the experiments (left) and the predicted mode shapes (right)

Table 4: Comparison between the experimental and predicted modal properties of the log house

No.	Experimental modes			Predicted modes		
	Description	Frequency (Hz)	Period (s)	Description	Frequency (Hz)	Period (s)
1	Mixed mode	5.39	0.186	Translation-Y	5.04	0.198
				Translation-X	5.08	0.197
2	Translation-X	11.85	0.084	Torsional	6.39	0.156
3	Torsional	14.93	0.067	Translation -X	14.55	0.069
4	Translation-Y	20.53	0.049	Translation- Y	15.37	0.065

3.5 Damages observed

There was no damage observed for the tests carried out with the lowest scaled spectrum of the Montenegro (1979) earthquake (0.07g PGA). The only damage that was noticed after the 0.28g tests was the sliding of the logs in the North Western corner of the house, located near the window in wall W3. There was some more minor damage observed after the PGA was increased to 0.5g. No kind of repair intervention was carried out between the successive stages of the experiment. The following damage was observed at the end of all the tests: fracture along the grain of the log due to out-of-plane flexure, fracture along the grain at connections between orthogonal walls due to shear, internal cracks in the log section and fracture at the top and bottom notches of the logs. The typical damages can be seen in Figure 12.

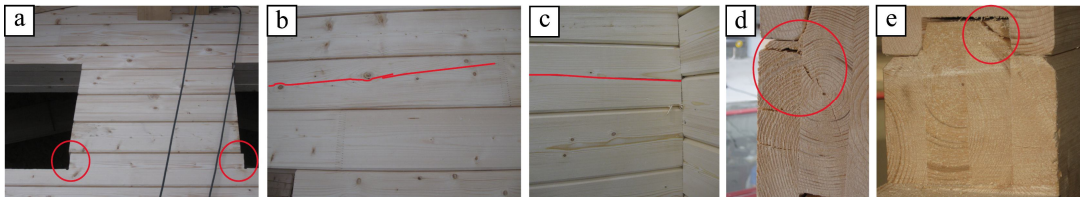


Figure 12: Damage observed after completion of all tests: (a) slipping of logs in the NW part of the house; (b) fracture along the grain of the logs; (c) splitting of logs at cross wall intersections; (d) internal cracks seen in the log cross section; (e) damage seen at the top notch of a log

5. CONCLUSIONS

The above tests were performed within the framework of the SERIES Project ‘Multi-storey timber buildings’, that involved research groups from the University of Trento, Italy, the University of Minho, Portugal and TU Graz, Austria, in collaboration with LNEC, Portugal. In spite of repeated shaking table tests and a strong input of 0.5g, the integrity of the log house was hardly affected. The good seismic performance can be attributed to the carpentry joints, the positioning of the mechanical fasteners around the openings and at the cross-wall corners, the contact between log courses facilitated by the grooves in the logs and the anchor bolts connecting the sill-log and foundation.

The fast pace and ease of construction and the absence of major damage in the building indicate that this form of log construction would be a viable option in areas that have high seismic hazard. The fundamental frequency obtained was 5.39Hz during the initial characterization test and reduced to 5.109Hz [T=0.196s] during the final characterization test. The small difference in the magnitude reinforces the observation that the damages suffered by the log house were minor. The proximity of the values of the first two predicted modes can be attributed to what was seen during the shaking table tests – a combination of

longitudinal and transverse movements of the entire house during the first mode. Apart from the fundamental mode, there was a discrepancy in the frequency values obtained for the next three modes. This can be attributed to the fact that the numerical model was very simple. The seismic response of the model was mainly due to the compression stresses perpendicular to grain and the shear stresses at the cross wall connections as the model did not account for the contribution of friction and the self-tapping screws. A detailed analysis of the dynamic properties, forces and deformations in the log house will be furnished in an ensuing paper.

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