Cyclic behaviour of *Pombalino* "frontal" walls

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ABSTRACT:
The mixed wood-masonry XVIII century *Pombalino* buildings of downtown Lisbon have a recognized patrimonial value, both nationally and internationally. These buildings have a three-dimensional timber structure enclosed in surrounding masonry walls aimed at providing increased seismic resistance. This paper describes an experimental campaign to obtain the hysteretic behavior of these interior walls, named "frontal" walls, by static cyclic shear testing with controlled displacements. The vertical load applied assumed that the wall was placed at the first floor. The loading protocol used was the CUREE for ordinary ground motions, tailored specifically for wood structural components. A total series of 3 tests were conducted in 3 identical real size walls. The hysteretic behavior of such walls subjected to cyclic loading exhibit high nonlinear force-displacement responses and high ductility. As previous experimental studies on "frontal" walls are very limited, these results are very useful and essential for further analytical work in modelling the non-linear behaviour of such walls.

Keywords: Pombalino buildings, experimental study, cyclic static shear testing.

1. INTRODUCTION

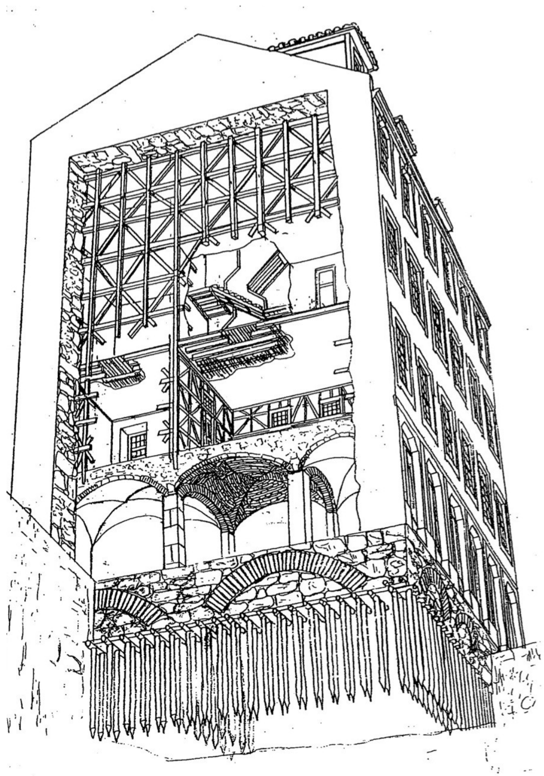

Figure 1: Example of a *Pombalino* building [Jorge Mascarenhas, 2005].

The mixed wood-masonry XVIII century *Pombalino* buildings of downtown Lisbon have a recognized patrimonial value both nationally and internationally. In 1755 a catastrophic earthquake followed by a considerable tsunami stroke the capital of Portugal causing severe damage to the city. The event completely destroyed the heart of the city, which was set on a valley area close to the river Tejo and is composed of a shallow layer of alluvium material. The disaster required an urgent solution. The Prime Minister at the time, Marquis of *Pombal*, was set responsible for the reconstruction of the city and to bring it back to normality as fast as possible. Then, he delegated to a group of engineers the development of a structural solution that would guarantee the required seismic resistance of the buildings. Based on the know-how of that time and on the empirical knowledge gathered from the buildings that survived the earthquake a new construction type was created, this being generally referred to as *Pombalino* construction nowadays. An example of the constructing elements that compose a *Pombalino* building can be seen in figure 1.

This construction type can be summarized as follows, based on Jorge Mascarenhas [2005]. The buildings were built in quarters comprising each block an average of 10 buildings. The foundation system was ingenious; it is found a system of wooden piles over the alluvium layers. The piles are similar and repetitive, on average 15 cm in diameter and 1.5 m in length. These form two parallel rows in the direction of the main walls, which were linked at the top by horizontal cross-members attached by thick iron nails. The construction between the ground and first floors consisted of solid walls and piers linked by a system of arches. In more elaborate cases, thick groined vaults spanned between the arches, which protected the upper floors from the spread of any fire that might start at ground floor level. From the first floor up this building system has a three-dimensional timber structure called "gaiola" (cage), thought to be an improved system based on prior traditional wooden houses. The "gaiola" is composed of traditional timber floors and improved mixed timber-masonry shear walls ("frontal" walls) that would support not only the vertical loads but also act as a restrain for the seismic horizontal loading. These "frontal" walls are one of the main speciousnesses of these buildings. They are made up of a wooden truss system filled with a weak mortar in the empty spaces.

Very few data, analytical and experimental, exists on the behaviour of the "frontal" walls. Such data can be obtained from experiments consisting of physical tests of representative specimen. For this reason it urges to carry out experimental work that can further back up analytical computer models. The experimental activity carried out on these walls so far is limited to Pompeu Santos [1997] and Cruz *et al.* [2001]. In the fist paper referred, three (although not exactly with the same configuration or dimensions) real site specimen were transported to the laboratory and further tested under reversed cyclic loading with imposed displacements. The study was the first to test such walls under static cyclic shear testing but in these tests no specific loading protocol was used and it is important to mention also that no vertical loading was applied to the structure. These two aspects are seen to be downside of this study. Furthermore, although the experimental work has the advantage of using real site specimen, this fact has also disadvantages with regards to the difficulty of transportation of the specimen to the laboratory and reproduction of the real support (boundary) conditions expected in the building. For the transportation a steel cage was built minimizing the deformations and shocks on the specimen; for the support conditions it was built a concrete shoe plate to assure that the lower horizontal beam would remain still with relation to the ground floor (see figure 2).

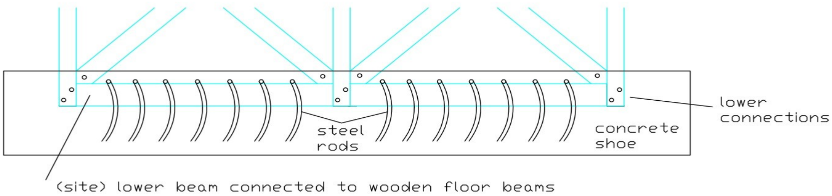

Figure 2: Schematic drawing of concrete shoe for support conditions in experimental set-up of Pompeu Santos [1997].

Even though, when using real site specimen, it seems that this type of support could not be avoided, it has the immediate disadvantage of seeing the lower connections of the vertical to horizontal beam being covered by the concrete shoe. It is believed by the authors that this solution for the support conditions increases the stiffness and strength of the tested walls when compared to the reality of the site (at the site only the lower wall beam is connected to the wooden floor joists). On the second paper mentioned it was intended to assess the possibility and efficiency of using FRP rods and glass fibre fabric, together with epoxy adhesives, in the strengthening of damaged "frontal" walls. What is relevant to point out on this experimental work is that scaled models (1:3) of the wall panels were built and tested in diagonal compression (under monotonic loading). Although these tests are much less time consuming and easy to perform than panel shear tests on real size walls, the data obtained with monotonic testing is preliminary and limited to the most to envelope curves on force-displacement relationships of scaled models. For the detailed seismic analysis foreseen for the current study it is required to test the specimen under reversed cyclic loading to obtain the desired hysteresis behaviour of such walls.

The objective of the experimental work developed and presented herein is, therefore, to estimate the cyclic behaviour of the "frontal" walls. These properties shall be used in developing an analytical model of the structure to calculate its response to earthquake ground shaking, and in developing acceptance criteria for strength and deformations. The test specimen and experimental set-up tries to replicate the construction details, support and boundary conditions and loading conditions that can be found in reality in these buildings. A total of three walls were constructed and tested at the laboratory. Thus, the objectives of this study are to obtain, with static cyclic shear testing with controlled displacements, the hysteresis behaviour of the "frontal" walls.

2. DESCRIPTION OF THE SUBASSEMBLY

A schematic drawing of the wall construction details is shown in figure 3. The intention was to fabricate three specimen to test at the laboratory that very much resemble the real site specimens brought to test at the laboratory in Pompeu Santos [1997].

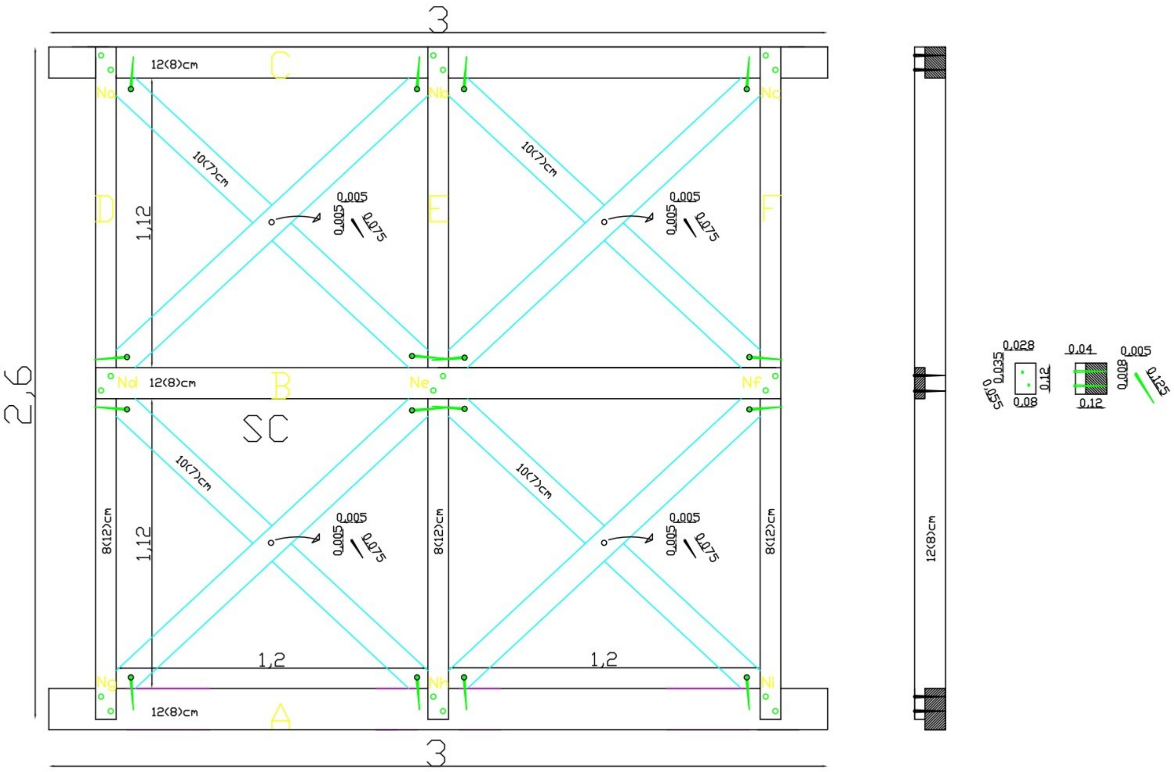

Figure 3: Schematic drawing of "frontal" wall specimen and of the details of its connections (units in meters otherwise specified).

However, the timber species used are probably not the same and the section dimensions in the real site specimens are slightly higher than the ones used herein; on the other hand, it is important to notice that, in reality, the section dimensions can vary a lot. The timber used in the walls consists of pine ordered from the timber mill. It was ordered a typical softwood Portuguese species (Pinus Pinaster Ait., *Pinho Bravo*). The reasons for selecting this species was that this is one of the most common species found commercially and therefore easy to find in quantity. Nevertheless, the typical timber species found on the *Pombalino* buildings vary from hardwood species such as Oaks and Chestnuts to softwood species such as Pines; unfortunately, hardwood species are harder to find in abundance in the timber mills which inhibits its use. In these old buildings, the found softwoods such as Pines seem to be more seldom. The choice of this *Pinho Bravo* is also due to the fact that this wood specie is the most commonly used timber for structural purposes in wood buildings in Portugal nowadays. The woods were ordered dried from green wood to 12% moisture content (the moisture content found in woods at the ambient temperature of 20º and ambient relative moisture of 65%), given that in the old Pombalino buildings the woods have had sufficient time to dry to the ambient moisture content. The wood sections were limited to the common sections found in the timber sawmill: 16x8 cm^2; 12x8 cm^2 and 10x7cm^2 in section area. Further ahead the wood beams were cut in the carpentry and transported to the university laboratory. In relation to the connections (number of nails, positioning of nails) these were imitated in more detail according to what was possible by the nails existing commercially nowadays. The details of the connections and nails used can be seen in figure 3. The nails used were all pyramidal of 12.5 cm in length by a section of 10x6 mm^2 in the base section. Exceptionally, the nails used to connect the crossed braces among each other are smaller, of 7.5 cm in length by a base section of 5x5 mm^2. In the main elements a pre-drilling was made of 7.5 mm in diameter only on the upper wood element. To be noted that the nails were bought from a store and were the only ones found, which are still fabricated by handicraft, and of forged steel, thus giving a good imitation of what existed in the past. For this reason also the nails can vary slightly in their dimensions (for instance up to 0.5 cm in length and/or up to 0.5 mm in side base section).

Afterwards the walls were mounted as described as follows. First, the elements were set horizontally on the floor and the main elements (all except the crossed braces) were attached together. The nails were nailed manually with a hammer. Later on, the diagonal crossed braces were set in the available space and nailed to the main elements with the longer nails. It was made sure that no empty spaces were found on the connection of the diagonals with the main elements. In relation to the support conditions, to tighten the walls to the horizontal reaction beam, it was built six steel shaped omega flanges to connect with screws the walls to the horizontal reaction beam, as is further ahead explained. These were set in the proper positions according to the positioning of the holes in the reaction beam. The walls were set vertically to proceed with the filling of the masonry.

The issue of the masonry filling is a problem not easy to solve. It is important to try and reproduce the usual filling that was used for these buildings. However, a great variety of fillings can be found for these walls. From the observations made by companies in rehabilitation or demolition works its has been found, different types of masonry such as fillings of typical mortar with bricks or mortar with tiles or even mortar mixed with small stones (which is thought to be the debris or leftovers from the earthquake). As for the mortar, only one study was found on the type of mortar used [Oz, 1994]. This study, based on a collection of real site fragments of mortar from a building under reconstruction, indicates that the used of lime was probably hydraulic; the predominant sand size was between 0.5 to 2 mm and that no particles of cement were found, as it is expected. Furthermore, the relation water\lime was probably superior to 1.0. Based on these observations it was decided that the masonry filling would consist of hydraulic lime with intersections of tiles and broken bricks. A 1.75/2 relation was set for the water/lime relation; a 1/3 relation was set for the lime/sand relation (this is the typical relation used in construction works); as for the sand, a 1/1 relation was set for the sand of river/sand of megrim. Previous studies [Carvalho, 2007] indicate that the mechanical resistance in compression, in accordance with EN1015-11 [1999], of such a composition of mortar does not exceed 1.5 MPa being at 28 days or at 195 days indicating it is a low strength mortar. Three months were the drying time for the masonry.

3. DESCRIPTION OF THE EXPERIMENTAL SET-UP

The test specimen described in this section is illustrated in figure 4. Figure 5 presents a photograph of the test frame with a specimen installed and figure 6 shows a schematic drawing of the anchorage system onto the horizontal reaction beam. In figure 6 one can see that the wall SC is anchored to the horizontal reaction beam at the bottom by steel braces in shape of omegas that embrace the lower beam (section 16*8 cm^2), which are anchored to the horizontal reaction beam by M24 (353 mm^2 in section area) screws, as shown in the figure. The anchorage system is very stiff and can firmly take the shear and moment reactions and avoid the rigid body motion of the wall while loaded (uplift forces). A total of 16 M24 bolts were used. This is believed to be a better system for the support conditions of the wall when compared to the possibility of constructing a concrete shoe, as in Pompeu Santos [1997].

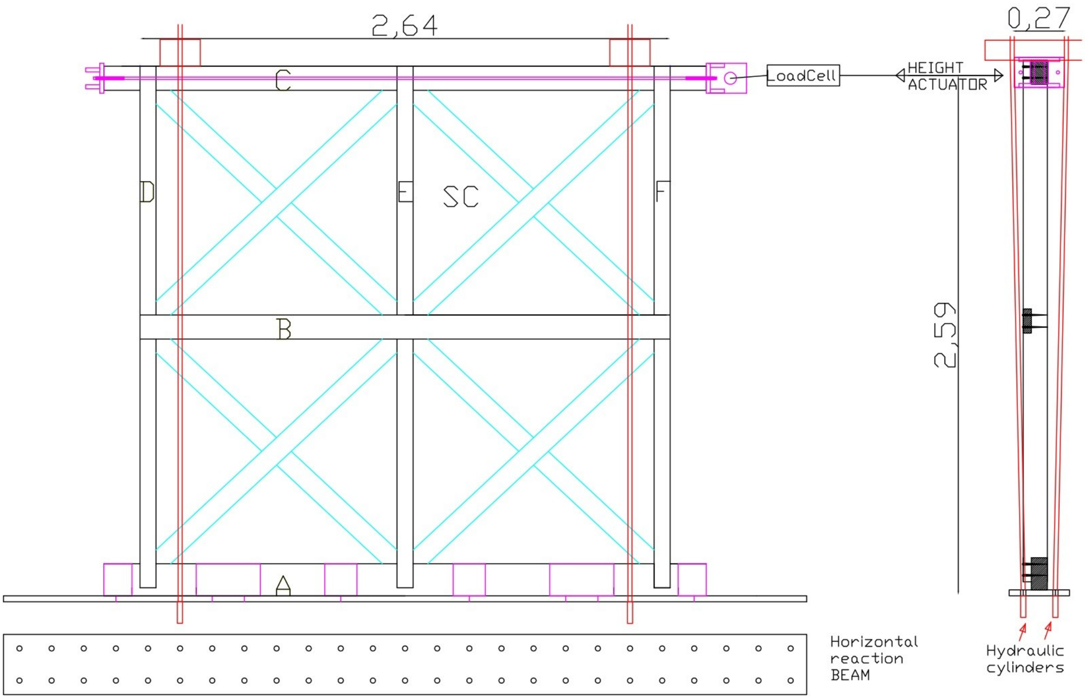

Figure 4: Schematic drawing of experimental set-up of specimen SC (units in meters).

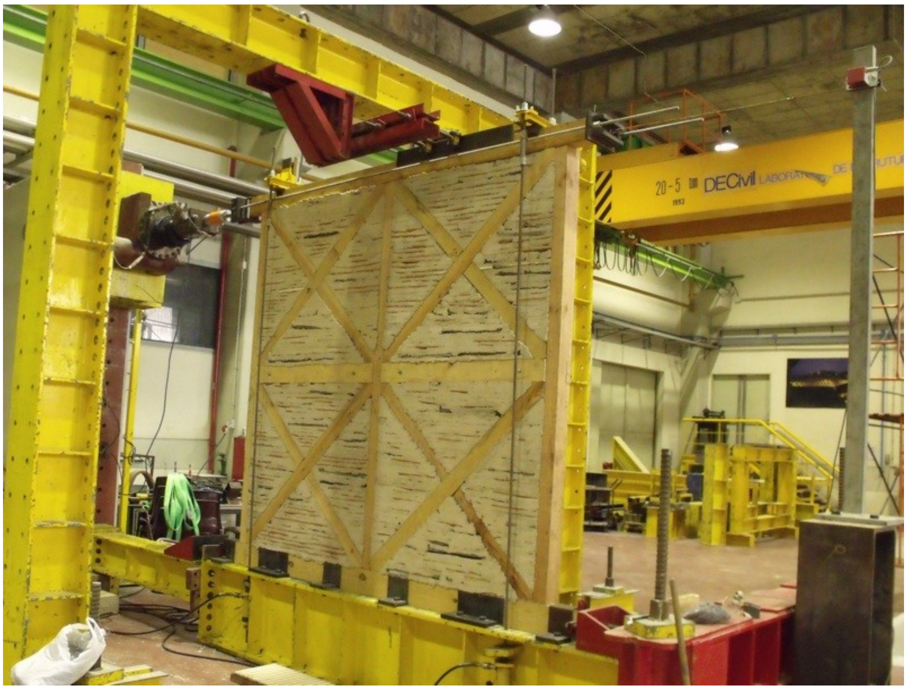

Figure 5: Photograph of specimen SC mounted with final layout.

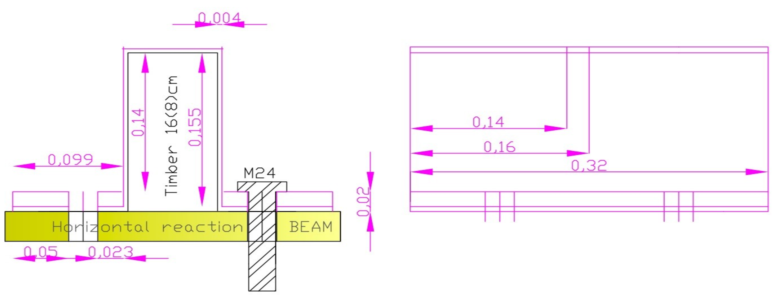

Figure 6: Schematic drawing of the anchorage system (units in meters otherwise specified).

The walls were tested with the loading applied at the top of the wall, using a 1000 kN capacity actuator with a 400 mm stroke. Data from two transducers placed on the specimen was recorded. It were recorded the load from the actuator with a load cell and the lateral displacement at the top of the wall via a linear variable differential transducer (LVDT). All data were acquired using a personal computer running Visual Basic. Transverse movement of the specimen during testing (out-of-plane) was resisted by a system of lateral roller bearings (in red in figure 5) supported in a metallic frame (in yellow in figure 5). The weights of the walls are, respectively: wall 1: 0.766 ton; wall 2: 0.756 ton; wall 3: 0.766 ton.

4. LOADING PROTOCOL DEFINITION

The CUREE protocol [Krawinkler *et al*., 2000] for ordinary ground motions was used to study the cyclic behaviour of the “frontal“ walls. This protocol consists of cyclic displacement sequences increasing in amplitude throughout the test; each segment consists of a primary cycle with amplitude defined as a multiple of the reference displacement. The primary cycle is followed by a series of cycles with amplitude equal to 75% of the primary cycle. The sequences of the cycle vary in length from 3 to 7 cycles. The input displacements for each of the tests performed can be seen in figure 7. All tests were conducted such that the initial position of the actuator was at half stroke allowing the maximum deflection in each direction (200 mm). The tests were conducted at a rate of 0.25 mm/s. The data were read at a rate of 1mm/sample as well as, in terms of force, at a rate not higher than 0.25 kN/sample.

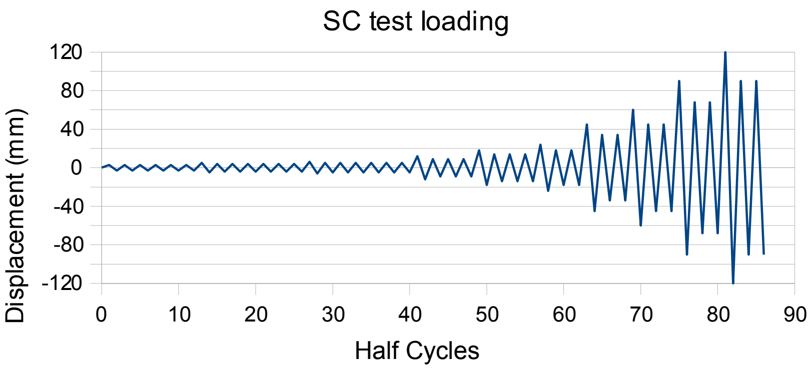

Figure 7: The loading protocol for the tests performed.

In this case study the calculation of the reference displacement was not done, as it would imply one specimen to be tested under monotonic loading like suggested in the CUREE protocol. Instead, is was set a maximum displacement in the loading protocol equal to the maximum displacement obtained at the LNEC experimental testing [Pompeu Santos, 1997], that is 120 mm. Nevertheless, these specimens are one module higher than the ones tested herein; it is for this reason expected that specimens SC1 to SC3 can attain a lower maximum displacement. Accordingly, at experiments of specimen SC1, SC2 and SC3 the structures were seen to stand heavy damage at a displacement of around 90 mm. For security reasons in experiments SC1 and SC2 the experiment was stopped here; nevertheless, in experiment SC3 it was carried out until a displacement of 120 mm since it was the last structures being tested.

5. VERTICAL LOADING

The vertical loading to impose on the test structure was determined based on Eurocode 1 [CEN, 2002] and is given by S_d=self weight+0.3xlive load. Is was considered that the wall was placed at the first floor of a three storey building plus ground floor and attic, so for the calculations of the vertical loads, the dead and live loads were multiplied by three. The area of influence of the walls was considered to be of four meters. For the vertical loads to be imposed on the structure it was considered a live load of 2 kN/m^2 and for dead load it was considered the weight of the compartment walls (0.1 kN/m^2), the weight of the each wall (3.0 kN/m), the weight of each of the wooden floors (0.7 kN/m^2) and the weight of each of the ceilings (0.6 kN/m^2). The output is a total vertical load of 30 kN/m along the wall (per meter of wall). The vertical loading was distributed along 4 hydraulic jacks having each one a total force of 19.2 kN.

6. RESULTS AND CONCLUSIONS

The obtained hysteresis curves for specimens SC1, SC2 and SC3 can be seen in figures 8, 9a) and 9b), respectively. The hysteretic behavior of the "frontal" walls subjected to cyclic loading are characterized by nonlinear behaviour describing the monotonic envelope. It is also observed pinching behaviour associated with strength degradation and generally fat loops can be identified dissipating reasonable amounts of energy. It is also observed a high ductility of the response. The maximum strength of the walls is around 50 kN. The ultimate displacement obtained without collapse of the structure is around 90 mm, resulting in an ultimate drift of around 3.5%.

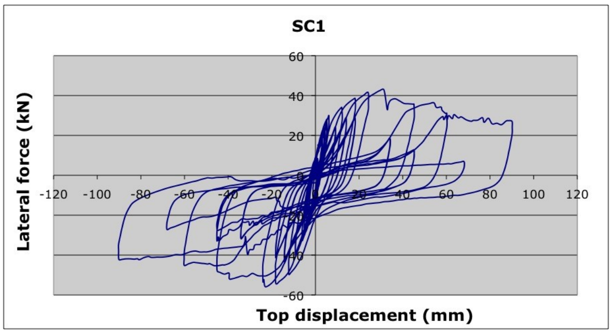

Figure 8: Hysteresis curve of wall SC1.

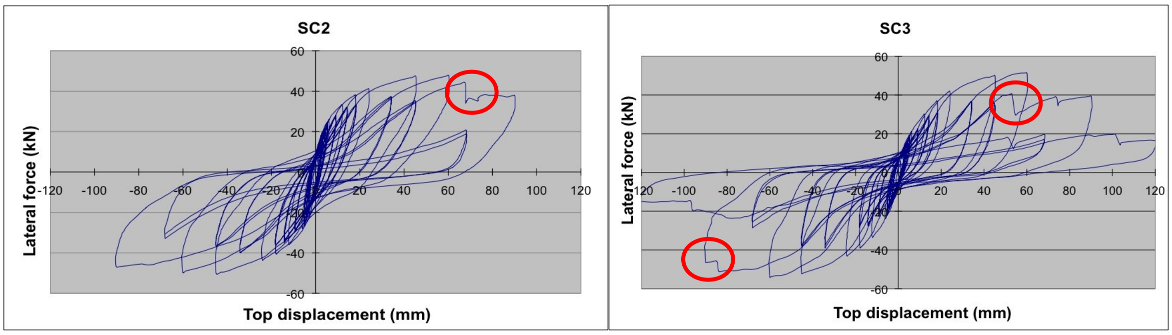

Figure 9: Hysteresis curve of: a) wall SC2, b) wall SC3.

In specimen SC1 the mode of failure was essentially the failure of the connections by cracking of the wood elements at the nodes, see figure 10 a). The failure of these connections in such terms is mainly due to the fact that these were designed as corner connections with no continuity towards the next storey of the vertical wood elements. Nevertheless, in reality it can happen that this continuity of the vertical elements towards the next storey exists, also because it is assumed that the tested walls are placed at the first floor. For this reasons, it was decided to avoid this type of failure in the next tested walls and the upper and lower connections in SC2 and SC3 specimen were reinforced with some steel plates as is depicted in figure 10 b). The mode of failure observed in specimens SC2 and SC3 can be seen in figure 10 c) and is characterized by the buckling and further rupture of diagonals, out-of-plane of the structure. The sudden loss of lateral resistance of the wall when a diagonal ruptures can be observed clearly in the hysteresis curves of figure 9, marked with red circles.

(a) (b) (c)

Figure 10: a) failure by cracking of wood at upper connection for wall SC1, b) node reinforcement for walls SC2 and SC3, c) failure by buckling of diagonal for walls SC2 and SC3.

It is important to refer the resemblance between the results herein obtained for the hysteretic behaviour of the three walls built in the laboratory and the results obtained for the hysteretic behaviour of the three site specimen walls tested in LNEC [Pompeu Santos, 1997]. In terms of general shape of the curves they are the same and one observes strength degradation and pinching behaviour in both. In terms of maximum strength attained, the results are also similar with slight higher strength for the higher real site walls. Finally, in terms of ultimate drift obtained the results are close for the two walls being around 3.5%. The results obtained herein are important for further work in modeling the behavior of such walls and also because testing on these walls is very limited. The obtained hysteresis curves can be incorporated into frame analysis software to study the behaviour of “frontal” walls under reversed cyclic loading or in simpler models, taking advantage of the envelope curves, to analyse the behaviour of “frontal” walls under monotonic loading. However, further validation with more extensive test data is clearly needed before any model can be used with confidence for this purpose.

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REFERENCES

Eurocode 1: Actions on structures - Part 1-1: General actions -Densities, self-weight, imposed loads for buildings, EN 1991-1-1, Comission of the European Communities (CEN), Brussels, April 2002.

EN 408-Timber structures- structural timber and glued laminated timber- Determination of some physical and mechanical properties, 2003.

EN12512-Timber structures- Test methods- cyclic testing of joints made with mechanical fastners, 2005.

EN1015-11- Methods of test for mortar for masonry. Part 1.1 Determination of flexural and compressive strength of hardened mortar , august 1999.

Helena Cruz, José Pedro Moura and José Sporiti Machado, The use of FRP in the strengthening of timber reinforced masonry load-bearing walls, Historical Constructions, Guimarães, page 847, 2001.

Jorge Mascarenhas, Sistemas de Construcção; V-O Edifício de rendimento da Baixa Pombalina de Lisboa, Materiais Básicos 3º Parte: O Vidro. Livros Horizonte (in Portuguese), 2005.

Joao Carvalho, Caracterização Mecânica de Paredes Resistentes em Alvenaria de Pedra Através de ensaios não destrutivos; Master thesis, Instituto Superior Técnico (IST) (in Portuguese), 2007.

Krawinkler, H., Parisi F., Ibarra L., Ayoub A. and Medina R , Development of a testing protocol for wood frame structures. CUREE-Caltech Woodframe Project Rep., Stanford University, Stanford, California, 2000.

Langlois J.D., Gupta R., Miller T.H., Effects of Reference Displacement and Damage Accumulation in Wood Shear Walls, Journal of Structural Engineering ASCE, 2004.

OZ Empresa Lda, work developed for Intergaup, Levantamento e Apoio à Modelação estrutural dos Edíficios do BCP da Rua Augusta e Rua dos Correeiros em Lisboa (Caracterização dos materiais) (in Portuguese) , 1994.

Pompeu Santos, Ensaios de Paredes Pombalinas. Nota Tecnica Nº15/97, NCE/DE, LNEC Lisboa (in Portuguese) , Julho 1997.

Pompeu Santos, Laboratory Tests on Masonry walls taken from an Ancient Building in Lisbon. Procc. of the 34th Meeting of CIB W023 Comission in October 1997 in Lisbon, CIB Publication 231, pg. 89-94, 1999.