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EVALUATION OF THE SEISMIC PERFORMANCE OF AN OLD MASONRY BUILDING IN LISBON

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ABSTRACT

The purpose of this study is the evaluation of the seismic performance of an old masonry building in Lisbon. The work comprised both experimental and analytical studies. The structure exhibited very poor strength, attributed to the low quality of the materials used. However, it revealed to possess an available ductility under monotonic loading higher than expected. It was concluded that if the building was acted by a strong earthquake, such as prescribed in the portuguese code of actions, it would probably collapse.

KEYWORDS

Seismic performance; old masonry building; limestone mortar; brick and rock filled; wood panels; experiments in situ; linear dynamic analysis

INTRODUCTION

The building which was analysed dates from the early 1920's and its structural system and materials are representative of a large number of buildings built in Lisbon from the middle 19th century to the middle 20th century, which account for a large percentage of the Lisbon building stock. The opportunity for the realisation of this study arouse from the fact that the building was being demolished and destructive tests to rupture were feasible.

DESCRIPTION OF THE BUILDING

The building is located in the slope of the right-hand side of the Alcântara valley, which was a limit of the town until the middle nineteen century. No engineer or architect participated in the design of the building, which was a current situation at the time.

The building comprised five floors, including the ground floor. There are two flats per floor with entrances to the stair landing. Figure 1 shows the ground floor plan of the building and a vertical cut. The materials used were:

- the exterior walls of the facades and sidewalls were made with a rock and brick filled mortar of limestone and sand, of extremely poor quality. In the walls of the facades there were also wood elements forming frames within its vertical and horizontal members.

- the walls of the staircase and the wall of separation of the flats, adjacent to the first one, are brick walls connected by the same kind of mortar used in the exterior walls. There are horizontal wood members at floor levels and vertical wood members at the corners. These members are connected to the rest of the walls by steel dowels spaced approximately 40 cm. The dimensions of the bricks are 23 x 11.5 x 6.5 cm^3 with a 1.5 cm cover on either side.

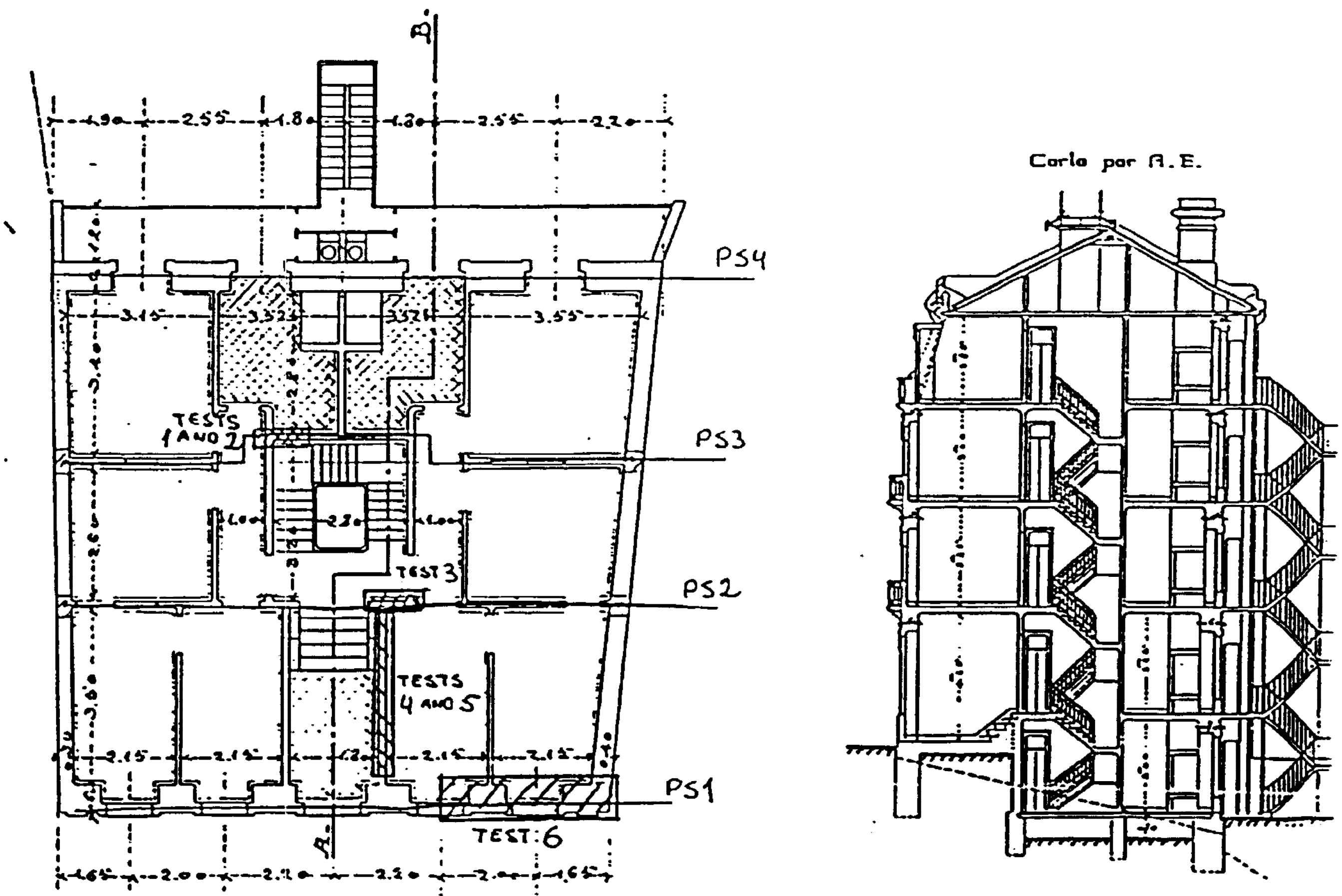


Fig. 1 Ground floor plan and vertical cut AB

- all the other interior walls are made of wood and comprise of two kinds of walls. The ones that are oriented perpendicular to the facades consist of vertical strips of wood approximately 15 cm large and horizontal wood square strips (2 to 3 cm wide) filled with the some kind of mortar used in the facades. These walls are parallel to the main wood beams that support the pavements, and therefore, these walls do not support vertical loads and their function is essentially of partition walls. The walls parallel to the facades comprise the same kind of vertical wood strip as the previously described and similar strips inclined 15° to the vertical. The rest of the space of the panel, approximately 15 cm thick, is filled with the same mortar used in the rest of the structure.

- the pavements are made of the usual 15 cm large wood strips running parallel to the facades supported by 0.10 m x 0.20 m rectangular wood beams running in the perpendicular direction, which are supported in the facades and in the interior wood panels parallel to the facades.

METHODOLOGY OF ANALYSIS

The analysis of the overall structure was performed by analytical methods using the program SAP 90 (Habibullah and Wilson, 1989). However, the greatest shortcoming of this kind of study when dealing with old buildings, is the characterisation of the properties of the elements and materials that constitute the structure, as the properties of the materials used in old buildings are usually not known. These buildings are usually built with wood, a material whose properties are strongly dependent of the degree of humidity and that degrade with time, and brick and rock filled masonry of extremely variable characteristics, generally very difficult to predict. Therefore the most reliable method to find out the properties of the materials used in old buildings is to test the prototypes of that kind of building.

Experimental Studies

The building object of this report was being demolished, what allowed several tests to rupture to be performed. However, the contractor that was executing the demolition was being paid for that purpose only and therefore the timing of the tests to be performed had to follow the timing of the demolition work. This limitation conditioned the kind of tests that could be performed.

The application of vertical loads and cyclic horizontal loads would demand experimental set-ups that would not be possible to build and place in position in the available time. Therefore it was decided to perform only monotonic tests to rupture, this being the greatest shortcoming of the present study. The implications of this shortcomings will be discussed at a later stage.

The tests had to be planned in such a way that the necessary set-up could be placed in position very quickly. Therefore it was not possible to use reaction walls or frames. Instead, parts of the building had to be used for that purpose. Thus, all the tests followed the organisation shown in fig. 2, with part of the existing structure used as reaction wall. The weight of the jacks was supported in general by the part of the structure used as reaction wall. The instrumentation comprised one load cell to measure the applied force and one or two sets of three transducers to measure horizontal displacements of the elements to be tested at three vertical levels. The transducers were fixed to a scaffold independent of the elements being tested, thus measuring absolute displacements.

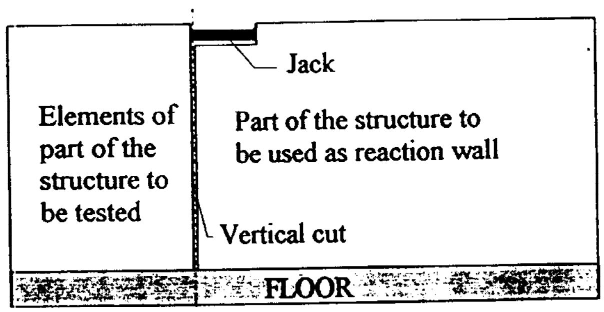


Fig. 2 Test arrangement

A total of six tests were performed. Three in interior wood panels, two in the staircase and one in the main facade. The elements tested are identified in fig. 1. The other elements in the same plane work as reaction walls during the tests with the jack in between. The vertical cuts, as shown in fig. 2, were purposely done for the tests. Figure 3 shows a schematic representation of test 1 and test 2, performed on the staircase. The dimensions of the tested elements, and the location of the transducers and the jack are shown. Figure 3 also shows a similar representation of test 3 on a vertical resisting interior wood panel. Figure 4 shows the same kind of representation for tests 4 and 5 in the partition only wood panel. Figure 4 also shows the schematic representation of the test of the facade.

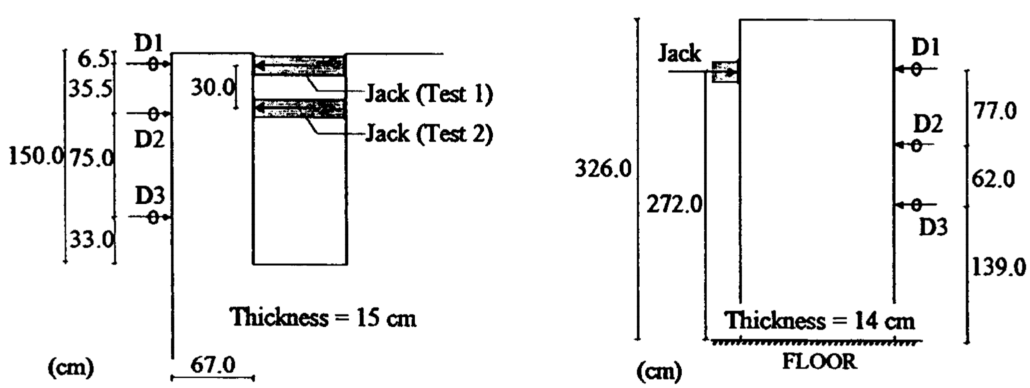


Fig. 3 Tests 1 and 2 in the staircase and tests 3 on a vertical resisting wood panel

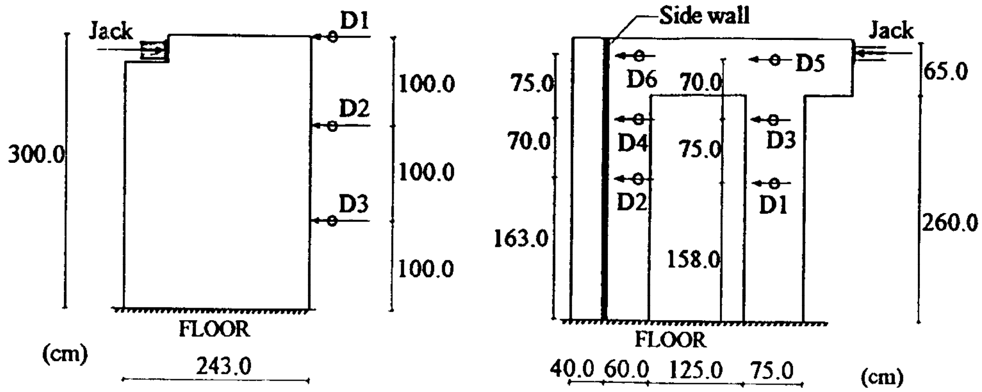


Fig. 4 Tests 4 and 5 on the partition wood panels and test 6 on the facade

Analytical Studies

Since the sidewalls were not tested, the two analysis performed were plane linear dynamic analysis on the direction parallel to the facades. As the pavement is made of wood strips and beams, it is highly questionable to assume it will impose a rigid body movement to the floor plan when the building is horizontally loaded. Therefore, two analysis were performed:

- one comprised only a plane linear analysis of the main facade only. All the connections between the different members were modelled as being perfectly rigid, what may not correspond exactly to the reality. The masses considered in the dynamic analysis correspond in each floor, to the weight of the part of the structure transferred to the foundation by the facade. It was considered one degree of freedom per floor, which was the respective horizontal displacement. All horizontal members of the model were assigned large areas in order that all nodes in each floor had the same horizontal displacements.

- The second analysis consisted on the plane dynamic analysis of all the plane structures of the building parallel to the main facade associated in series in a single plane frame. The different plane structures are identified in Fig. 1. All the connections were considered rigid, with two exceptions in PS2 and PS3 (plane structures 2 and 3):
- the parts of the sidewalls working together with those frames, were hinged to the horizontal members as these were wood bars 0.20 x 0.10 m^2 with a weak connection to the sidewalls.

- in the case of PS3, as the staircase was not aligned with the wood panels, transfer of moments was not considered and the respective connections are hinged. The same horizontal displacement was imposed to all nodes at each floor level as in the previous model. The mass assigned to each floor was the total mass of that floor.

In the above-mentioned calculations the following weights were considered for the different components of the building:

- wood pavements	f = 1.5 kN/m^2
- interior wood walls	f = 1.35 kN/m^2
- walls of the facades and sides	f = 22 kN/m^3
- brick wall of the staircase	f = 16 kN/m^3
- ceiling	f = 1 kN/m^2

It was considered soil type II and a relative damping coefficient of 0.10. The analysis was performed for both types of seismic ground motion defined in the Portuguese Code of Actions (RSA, 1983). Ground motion type I represents a near field event of medium intensity and ground motion type II a far field event of large intensity. Figure 5 shows the response spectra for both kinds of seismic ground motion.

The mechanical and physical properties of the different members of the models were calibrated from the experimental results. The effect of the global torsion of the building was considered only for the second analysis, since in the first one it is assumed that the pavements are not rigid in their own plane. The effect of torsion was studied by a simplified method based on the assignment of a single value of stiffness, to each plane structure as if it was a single degree of freedom system. Details of these studies are given in the references (Azevedo and Lopes, 1995)

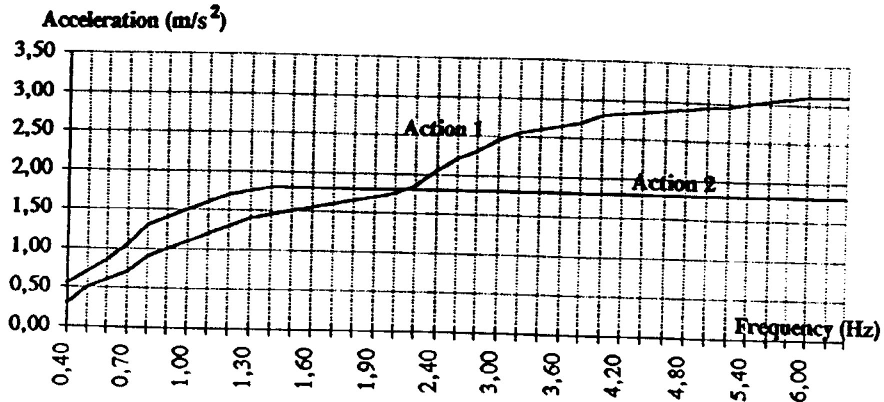


Fig. 5 Acceleration response spectra for type II soil and 10% critical damping

RESULTS

Results of the Experiments

The force displacement relationship corresponding to the displacements measured during the test were plotted. Only selected results are inserted in the text to illustrate the main subjects discussed. Full results are given elsewhere (Azevedo and Lopes,1995). These reference also includes a brief description of the tests.

Evaluation of Equivalent Stiffness Properties

As already referred to, the mechanical properties of the different bars used in the analytical models were calibrated from the experimental results. For the elements of the facades, only the moments of inertia were evaluated considering the real dimensions of the elements. The shear deformation was not considered in the analytical models. The results of test n°. 6 were used to calibrate an equivalent Young modulus to be used in the analysis. Figure 6 shows the relationship force versus average of displacements 5 and 6 (defined in fig. 4). Since the behaviour is not linear and a linearization of this relationship was necessary for the purpose of deriving the equivalent Young modulus, a fictitious yield point was defined approximately (point A in fig. 6). The stiffness of the part of the structure tested with relation to the degree of freedom horizontal displacement at the level of displacements 5 and 6 was then evaluated as being the secant stiffness at point A.

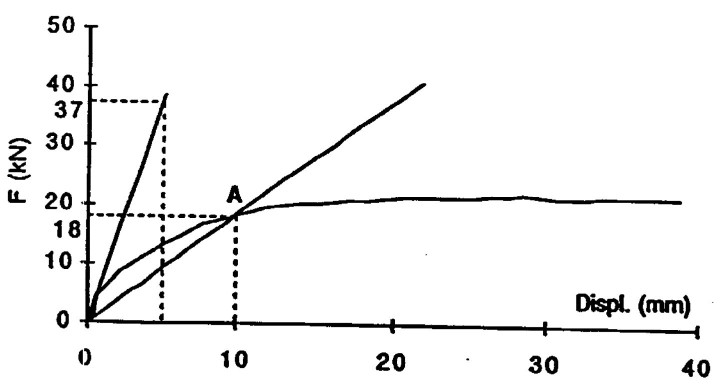


Fig. 6 Test of the facade. Relationship force versus average of top displacements

$$K = \frac{F_A}{Displ_A} = \frac{18kN}{10mm} = 1.8kN/mm$$

This value is more than 4 times less than the initial stiffness which is approximately:

$$K_{initial} = \frac{37kN}{5mm} = 7.4kN/mm$$

To calculate the equivalent Young modulus, an analytical model of the part of the structure tested, shown in Fig. 7, was analysed.

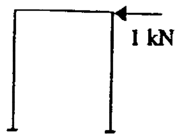


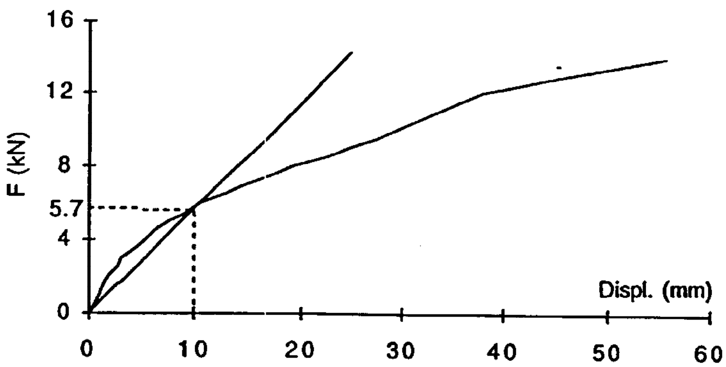
Fig. 7 Analytical model of the part of the facade that was tested

It was found that an Young modulus of E = 0.16 x 10^6 kN/m^2 should be assigned to the members of the analytical model for this to have the stiffness measured in the tests. The equivalent Young modulus for the initial stages of loading would be:

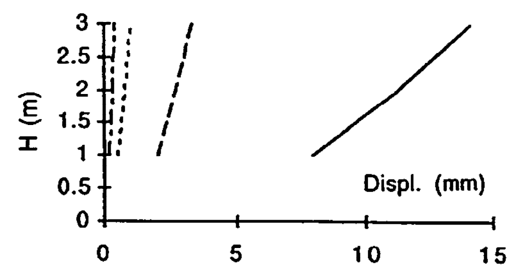
$$E_{initial} = \frac{7.4}{1.8} \times 0.16 \times 10^6 = 0.66 \times 10^6 kN / m^2 = 6600 kgf / cm^2$$

This weak value is attributed to the low quality of the construction.

The mechanical properties of the interior wood bearing walls and the staircase were calibrated by the same method as for the facade. From test 2, performed on a interior bearing wall the experimental stiffness of K = 570 kN/m can be obtained (fig. 8). Figure 8 also shows the deformation of this wall along the height, for several load levels.



Relationship force versus displacement 1.



Deformed shape along the height for several load levels

Fig. 8 Test 3 on the interior wood bearing wall.

The fig. indicates that the predominant deformation mode is a shear or a rigid body rotation around the base. In either case the overall behaviour can be simulated by a cantilever with shear deformation only. For a shear cantilever with the dimensions of the element tested (shown in fig. 3) to have the experimentally measured stiffness its shear modulus should be G=8860kN/m^2. However, the tested panel was free at the top what would not be the real situation. This means that if the predominant deformation mode was the rotation at the top and bottom connections, the stiffness could be higher. Therefore, a value of G for the wood panels 4 times higher was also considered and both cases were analysed. The staircase was also considered to have only shear deformation. The experimentally measured stiffness was K = 300 kN/m, as shown in fig. 9. The corresponding analytical value for G is 4300 kN/m2.

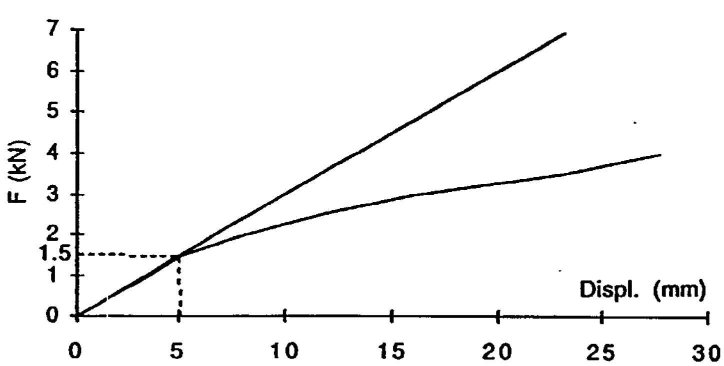


Fig. 9 Test 2 on the staircase. Relationship force versus displacement 1

Results of Analytical Studies

The dynamic analysis of the facade has yielded the following results:

Table 1 - Frequencies for the plane dynamic analysis of the facade

Mode	Frequency (Hz)
1	0.677
2	1.877
3	2.682
4	3.764
5	5.349

The analysis yielded higher values of displacements and forces for ground motion type II, what is due to the low fundamental frequency. Therefore, only the results for this analysis are presented. The shear forces on the two bars that represent the elements of the facade tested were:

$V_{21} = 22.9$ kN $\quad\quad$ $V_{26} = 28.0$ kN

The displacement at top of the building was $\delta_{top} = 0.074$ m and the seismic coefficient c = 0.08. The plane dynamic analysis of the overall building led to the following results considering G = 8860 kN/m^2. The values in brackets were obtained using the same model but considering $G_{wood\ panels} = 4 \times 8860$ kN/m^2.

Table 2 - Frequencies for the plane dynamic analysis

Mode	Frequency (Hz)
1	0.661 (0.785)
2	1.181 (1.192)
3	2.130 (2.475)
4	3.732 (2.743)
5	3.852 (4.037)

$V_{21} = 16.6\ (15.4)$ kN $\quad\quad$ $V_{26} = 23.4\ (22.0)$ kN

$\delta_{top} = 0.075\ (0.066)$ m $\quad\quad$ c = 0.077 (0.1)

The results of the overall plane analysis must be corrected by multiplying the shear forces by a factor ξ to take into account the torsional effects. The distance of the center of stiffness to the main facade (y_{CR}) and the value of ξ for the main facade are respectively:

$y_{CR} = 5.476\ (5.396)$ (m)

$\xi = 1.043\ (1.035)$

The point of load application to calculate the values of ξ is at a distance of 0.05 x 10.4 = 0.52m from the center of mass, measured in the direction of the facade. If the values of V_{21} and V_{26} are multiplied by the corresponding values of ξ, the following values are obtained:

$V_{21} = 17.3\ (15.9)$ kN $\quad\quad$ $V_{26} = 24.4\ (22.8)$ kN

DISCUSSION

The value of the seismic coefficients are lower than it could be expected, what can be attributed to the low fundamental frequency, which is also lower than it could be expected. This is a consequence of the stiffness degradation the structure would undergo during the earthquake. If the structure is subjected to vibrations that would not induce a nonlinear response, such as ambient vibrations, its dynamic structural characteristics would correspond to higher frequencies and lower damping ratios.

To assess the seismic performance of the building under earthquake loading the value of the acting forces in the elements of the facade tested must be compared with the respective strength. The measured strength of the part of the facade tested can be taken from fig. 6 and is $V_{rd} = 22$ kN. The acting shear force on this elements is

V_{sd} = 50.9 kN for the plane analysis of the facade and V_{sd} = 41.7 kN or V_{sd} = 38.7 kN (depending on the value of G considered for the wood panels) for the two global analysis of the entire structure including the effect of torsion. The first value of 50.5 kN is considered the most realistic since the other values were obtained on analysis based on the hypothesis of rigid floor behaviour, which is not realistic. However, this kind of analysis was performed to show that, whatever the hypothesis adopted, the conclusion is always that the acting forces largely exceeded the available strength.

If the plane model of the facade was correct it could be concluded the building only possessed 43% of the required strength to resist the prescribed seismic ground motion, this is, it would collapse. However, the model is not correct, due to several factors discussed as follows:

- the non-linear behaviour of the structure. It was considered only by means of an equivalent linearization through the appropriate choice of the yield points in the different tests. This obviously led to a more flexible structure, lower frequencies and lower seismic coefficients, than it would be obtained if the initial stiffness had been used in the analysis. However, this did not account for the possible hysteretic dissipation of energy. The test of the facade showed this wall is surprisingly ductile. However, the test was monotonic and under cyclic load the ductility could be considerably less. And there is no data about the shape of hysteresis loops. It is thought that the non-linear behaviour would lead to a moderate reduction of the strength demand on the structure. This reduction could only be evaluated by non-linear analysis based on models calibrated from experimental data.

- The non consideration of the effect of the axial force. This would probably lead to a stiffer structure, and therefore higher seismic forces and an increase on the strength demand. On the other hand it would probably enhance the strength capacity. For the evaluation of the influence of this factor it would have been necessary to have performed the tests with the application of axial forces. This factor would probably lead to a slight improvement of the expected performance of the structure.

- The strength of the foundations (or lack of it), the irregularities at ground floor level, and the fact that the building is located on a inclined slope could have a negative influence on the evaluated seismic performance of the building, not accounted for in the analysis.

It can be concluded from the above discussion that the factors not considered in the analysis could change quantitatively the conclusions derived in this report, but it would be unlikely that this change could have the magnitude necessary to produce a quantitative change in those conclusions. Therefore, it can be concluded that if the building analysed in this report was hit by an earthquake of the intensity similar to the ones defined in the Portuguese Code of Actions, would probably collapse.

There are thousand of buildings similar to the one analysed, in the Lisbon building stock. Many of them probably offer similar standards of safety to the analysed building. This means that if a very strong earthquake hits Lisbon, many of those buildings, hundreds, perhaps thousands, may collapse.

CONCLUSIONS

The main conclusion of this report is that the building analysed would probably collapse, if it was hit by an earthquake of intensity similar to the ones defined in the Portuguese Code of Actions. The extrapolation of this conclusion to the Lisbon building stock indicates that probably thousands of buildings would collapse in similar conditions.

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