

Seismic Strengthening of Masonry Buildings

Application to “Gaioleiros” buildings

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SUMMARY

The current study follows a preliminary analysis in which different techniques of seismic retrofit were compared. Now, based on a masonry building from the beginning of the XXth century, the study considered first different methodologies to strengthen the building floors, to enable their behaviour as stiff diaphragms. In a second phase it was studied the seismic protection of the building, with the implementation of viscous dampers.

1. Introduction

All major cities are facing problems related to the lack of space for new constructions and to the high number of buildings in poor conditions, some of them are even under risk of collapse [1].

Among the degraded buildings, the masonry construction is the one that presents more problems and the most in need of rehabilitation. This kind of construction presents several structural deficiencies, such as the fragilities of the main walls and foundations, and the almost complete absence of seismic resistance behaviour. Despite its poor conception it is also important to point out the contribution of a deficient maintenance and frequently, of recent interventions that contributed to the reduction of the building strength.

This work follows a previous one, in which it was studied four seismic retrofit techniques applied to a masonry building from the early XXth century, situated in Lisbon. Then, it was analyzed its structural strengthening with reinforced concrete walls, with a base-isolated solution and with the application of viscous dampers [2].

In the present work, in order to perform a deep analysis of the building behaviour, it was developed a computer simulation, with the aid of the finite element method. The analysis studied two methodologies of structural strengthening. The first analysis considered different techniques to strengthen the floors, to make them behave as stiff diaphragms. In a second stage it was considered different viscous dampers solutions to enhance the structure strength to seismic actions.

All the techniques evaluated are designated as passive retrofit, because during an earthquake they do not need the supply of energy to work.

2. Modeling

The building selected to this study is located in Lisbon and was built in 1911, during an important urban expansion occurred during the beginning of the XXth century. Its original characteristics were kept almost untouched since its construction [3].

The numerical modelling tried to achieve the best simulation possible of the seismic behaviour of the selected building. Therefore it was done thorough a choice of modelling elements and material properties. In addition it was made the comparison between the model modal frequencies and the frequencies measured through a dynamic characterization test to check the accuracy of the simulation.

2.1. Building characteristics

The building architectural characteristics gave it the designation of “Gaioleiro”. It has one basement, five floors and a mansard (Figure 1). It has three interior patios which enabled the entrance of light and air to the interior rooms (Figure 2). These elements will have an important role in the structural rehabilitation process.



Figure 1: Building's facade.



Figure 2: Example of the scheme design of a floor.

The foundations were executed in trenches opened in the ground and filled with stones connected by a low resistant mortar. The walls present three different typologies. The exterior walls and the interior patio walls were built in stone masonry connected by a low resistance mortar. The basement's walls were built in brick masonry. The partitioning walls of the rooms consisted in wooden strips nailed together, presenting poor structural properties. The floors consist in a grid of wooden beams, over which lays board flooring. The roof was made with ceramic tiles which were supported by a wooden structure. It was built with a mansard [3][4][5].

2.2. Model

The model was intended to be as accurate as possible to give a good simulation of the dynamic characteristics of the studied building. The model was developed using the software SAP2000, which uses the finite element method [6].

The restraints considered the walls of the basement to be fully supported in their basis. The exterior walls, in stone masonry, were modelled with three-dimensional solid elements. This element allows a better simulation of the distortion phenomena, while deforming outside the plane of the wall. The brick

masonry walls were modelled with shell elements. These plane elements were used to produce a lighter model, since this wall typology is not very important to the global dynamic response of the structure. The partitioning walls were modelled with frame elements arranged in an equivalent braced portal frame. The stiffness of the braced frame was determined according to distortion experiments, conducted in a building with similar characteristics [7].

The floor was modelled with frame elements to take into account the absence of stiff diaphragm properties. It was considered a distinction between the primary and the secondary direction, in which the floor works. It was assumed that the primary direction is perpendicular to the exterior walls and supported in the air shaft's walls. In the secondary direction there were only small wooden beams to brace the main beams which support the floor.

2.3. Calibration

The calibration of the model was based upon an in situ dynamic characterization test. The experiment enabled the analysis of the structure response to ambient vibrations. Its results were recorded as accelerations, which allowed the determination of the modal frequencies through the Fourier transform of the signal [2] [8].

According to the modal frequencies identified, it was possible to proceed the corrections in the numerical model to get the model frequencies closer to the ones obtained experimentally. The final numerical model is presented in Figure 3

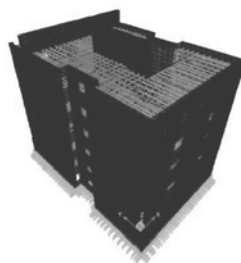


Figure 3: Model's perspective.

3. Seismic Action

The seismic action considered in this work is the one defined in the Portuguese Code (RSA). The software used in the modelling process allows the consideration of the seismic actions with a response spectrum analysis or by time-history sequences.

The time history analysis needs to calculate the dynamic equilibrium equation for each instant of the input signal. The input signal is usually a sequence of ground accelerations through time. The program allows the use of the fast nonlinear analysis, specially indicated for mainly linear models and with a few nonlinear elements. In the model used, the only elements which have nonlinear properties are the viscous dampers. Like this it was only necessary to consider the nonlinear behaviour in a few degrees of freedom of the structure.

Due to the randomness of the action, it was necessary to use 10 different ground acceleration signals, for each of the seismic action types defined in the code [9].

4. Rehabilitation Principles

From the seismic resistance point of view, the “Gaioleiros” buildings have several deficiencies. To begin each floor does not have the stiff diaphragm properties. A floor which behaves as a stiff diaphragm means that it has the ability to preserve its shape horizontally, without suffering dimensional changes or distortion for horizontal loads. Its main advantage is being able to have the same horizontal displacements in every points of each floor and therefore it can distribute the forces in accordance to the stiffness of the resistant vertical elements. This way it is possible to control the forces distribution, reducing the probability of the collapse of the exterior walls. This property also allows reducing the model degrees of freedom, therefore reducing the amount of calculation to be executed.

One of the main collapsing mechanisms in masonry buildings is the fall of the exterior walls by excessive deformation in the floor plane (Figure 4). With the increase of the floor stiffness it is possible to make the horizontal displacements compatible and minimize the risk of this type of collapse.

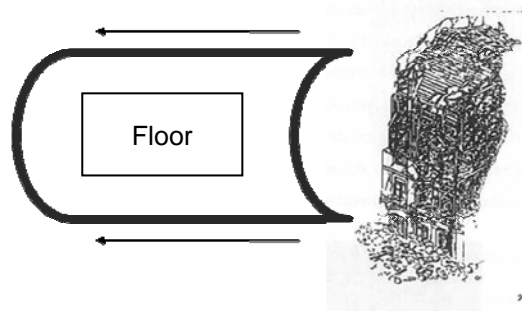


Figure 4: Mechanism of collapse in a masonry building (by floor deformation).

On the other hand, in these buildings, the resistance to horizontal actions is only accomplished by the stone masonry walls. This material has good behaviour for compression stresses, generated by vertical loads, such as the gravity loads. Nevertheless, under horizontal actions, bending efforts appear which produce tensile stresses in the masonry. The tensile strength is only equilibrated by the compression generated by gravity loads and by the mortar strength, which in most cases has already lost its bonding properties along the years. To improve the global behaviour under the effects of the seismic loads, the effect of the installation of viscous dampers in the building will be analyzed in the second part of this study.

The problems of structural safety, that rehabilitation projects induce, are in one hand associated to the definition of the seismic efforts level to be considered, and, in other hand, to the interence methods in the existent structure. It should be minimized the interference both esthetical and structural and it should be guaranteed the reversibility of the interventions to realize. These aspects are also analysed in the rehabilitation study developed.

5. Floor Strengthening

As previously referred one of the disadvantages of masonry buildings is its floor not behaving as stiff diaphragms. Therefore the seismic rehabilitation has to start with the strengthening of the floor. With this intent it was studied four different techniques, with different advantages.

After the development of the model of the original building, it was tested four different solutions for the floor: by reinforced concrete slabs; by composite steel-concrete slabs; by a metallic grids supported by metal beams; and by strengthening of the original floor with steel ties keeping the original pavement.

5.1. Solutions description

5.1.1. Concrete slab

The first strengthening technique tested was the substitution of the original floor by a concrete slab with 0,20m of thickness. This solution has the advantage to guarantee the stiff diaphragm property and therefore distribute the forces in an effective way to the resistant walls. Nevertheless it is an intervention which produces a few problems that should be accounted for.

The original situation has a weight of about $1,1\text{kN/m}^2$, while this solution has a weight of 5kN/m^2 , excluding fillings. This excessive increase in mass has two direct consequences, which make the concrete slab a less attractive solution. In one hand the larger mass produces the increase of inertial forces during an earthquake. On other hand this technique overloads the masonry walls which might already be in poor conditions. This solution, to be executable, it would be necessary to strengthen the foundations and build concrete columns in the exterior walls, to bear the increase of loads. This accessory works make this solution less viable from the simplicity of execution and reversibility point of view.

The modelling of this strengthening technique consisted in the use of shell elements with 0.20m of thickness replacing the frame elements, which simulated the wooden pavement (Figure 5 and Figure 6).

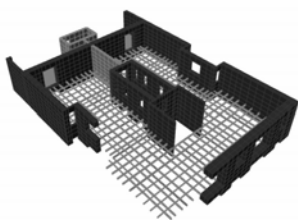


Figure 5: Model with wooden pavement.

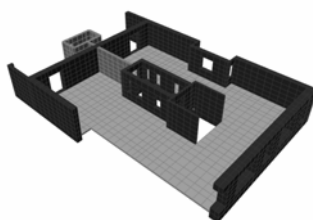


Figure 6: Model with concrete slab pavement.

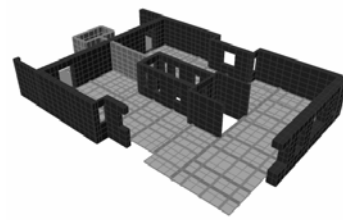


Figure 7: Model with composite slab pavement.

5.1.2. Composite steel-concrete slab

Another solution tested was the replacement of the wooden pavement by a composite steel-concrete slab placed over a grid of steel beams which are pinned in the masonry walls. It was considered a slab with a total thickness of 0,10m of concrete over steel sheeting with 0,75mm thick. The composite slab lies over

HEA200 steel beams, placed 2m apart, which are supported by primary beams, with the section of HEA300.

This solution has the advantage of being lighter than the previous one with the weight of 2kN/m^2 . The functional demands related to fire resistance, acoustic and thermal insulation are accomplished with the use of special paints and foams, which may increase its cost. Another issue is the need to remove the original pavement, what makes this technique impossible to execute in inhabited buildings.

The numerical modelling consisted in the simulation of the steel beams with frame elements with the same geometrical properties as the real ones. The composite slab was modelled with shell elements with an equivalent thickness to preserve the real weight and inertia (Figure 7).

5.1.3. Metal pavement

The solution of replacing the pavement with a steel grid placed over steel beams is not adequate to use in habitation. Although it can be interesting to consider it if the building is to be used for office or industrial purposes.

In comparison with the previous techniques this has the advantage to be lighter, with the weight of 1kN/m^2 . Nevertheless it has several functional problems, since it is not able to guarantee an efficient separation between floors, neither thermal nor acoustic insulation.

The model only considered the use of the metal beams loaded with the weight of the metal pavement. It was assumed that the metal floor will not contribute for the resistance to horizontal loads (Figure 8).

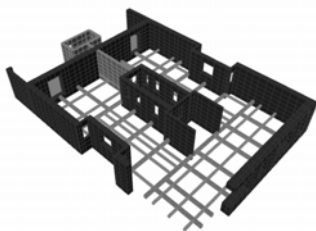


Figure 8: Model with steel pavement.

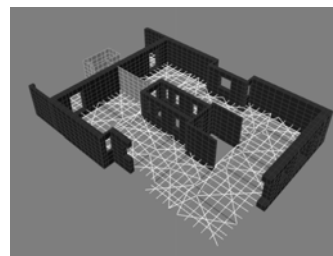


Figure 9: Model with steel ties.

5.1.4. Steel ties

The last strengthening technique consisted in the application of steel ties through the wooden beams of the existing pavement. It was intended to strengthen each room separately and connecting each other along the interior walls. The steel bars are anchored in each corner of the room, forming a cross, and are tensioned with the support of devices placed between the wooden beams. The connection between rooms is assured through metal pieces and beams connected by bolts. In the direction parallel to the primary wooden beams it was used UNP beams, while in the other direction it was used LNP beams which were interrupted by the wooden logs to preserve its integrity (Figure 10).

This solution has several advantages such as being reversible, it minimizes the interruption of the normal function of the building and it does not contribute to an excessive increase of the structure's weight. In

opposition to the other techniques presented, which need to relocate its inhabitants during the rehabilitation, this solution can be executed with the displacement of people for a short period of time.

The modelling consisted in the use of two frame elements crossed per room, with a diameter of 32mm (Figure 9) [8].

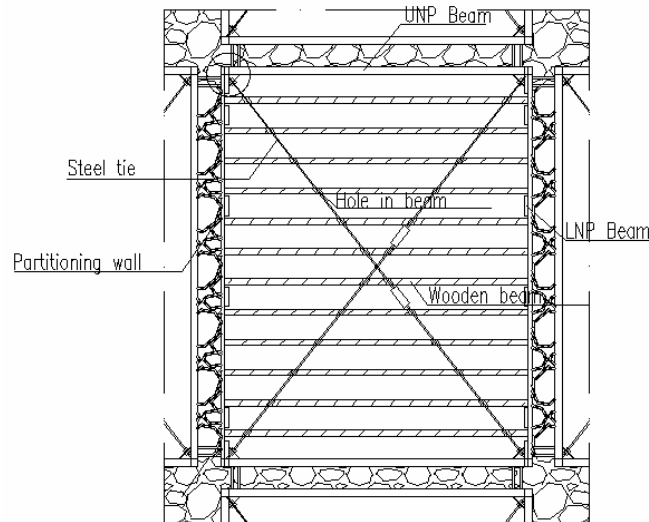


Figure 10: Strengthening with steel ties.

5.2. Results

The evaluation of the seismic behaviour of the different techniques presented was based upon the comparison of horizontal displacements due to the actuation of a seismic load as defined in chapter 3. The evaluation of displacements has two different objectives. First it was compared the displacements in each floor and in a second phase it was compared the displacements in different points on the top floor.

The comparison of the displacements in each floor allows determining which one of the techniques is able to minimize the seismic stresses. The results for the horizontal displacements along the smaller dimension of the building are presented in Figure 11.

The best results were achieved by the composite slab. The strengthening technique with steel ties presents results similar to the concrete slab. The technique with the worst results is the steel pavement with worse results than the original configuration. This behaviour is due to the larger inertial loads generated by the larger amount of mass vibrating.

The second level of analysis intended to sort out the horizontal displacement variation in the last floor. The closer the behaviour of a technique is of a stiff diaphragm, the smaller will be the difference between displacements along the perimeter of the building. In Figure 12 there are identified the horizontal displacements at different points of the last floor. The number 1 and 3 represent the corners of the walls, while number 2 represents a point in the middle of the wall, along the walls perimeter. It was also compared the displacement of points along the exterior walls and along the patio walls.

In the original model it is easily noticed that the exterior walls and the patio walls have different displacements due to a flexible pavement. Also point 2 presents higher displacements than the observed in the corners (points 1 and 3). The steel pavement has even worse results. The composite and reinforced slabs do not present such scattered results, although the reinforced concrete slab presents higher displacements due to the increase of mass. The solution with steel ties presents good results, although a little more scattered than the two previous ones [12].

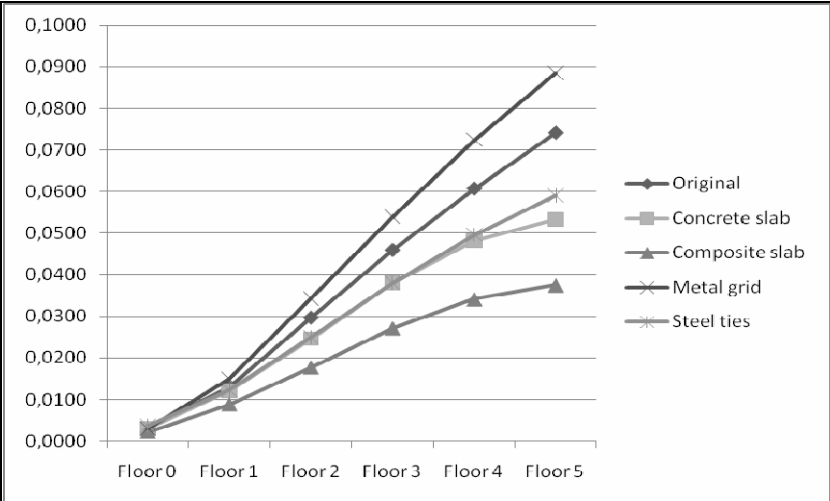


Figure 11: Maximum horizontal displacement in each floor along the building's smaller dimension.

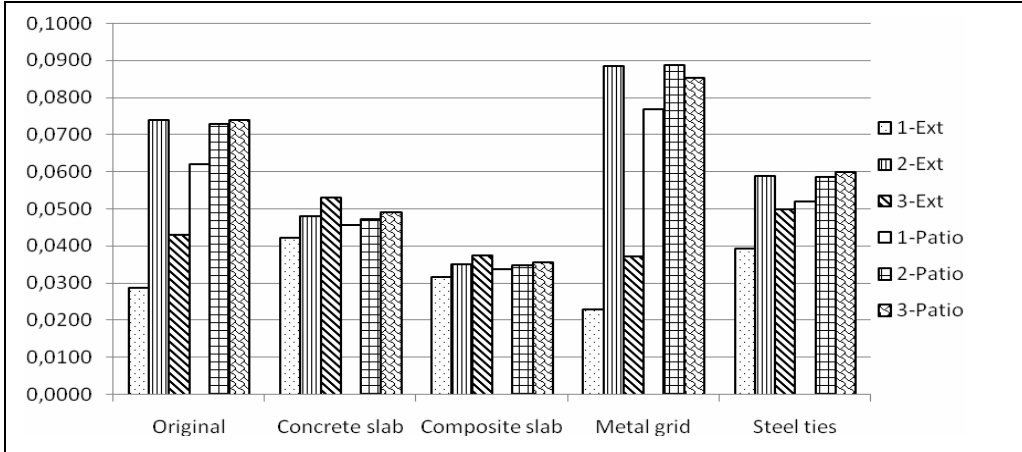


Figure 12: Horizontal displacement in the last floor along the building's smaller dimension.

6. Strengthening with Viscous Dampers

The use of energy dissipation by damping phenomena to increase the seismic response of a structure is already common in bridges, as it was used in the Vasco da Gama Bridge in Lisbon. Although its use in buildings is not common, since it implies a more sophisticated study. In this kind of analysis it is not possible to use a behaviour coefficient to account for the nonlinear stresses during a dynamic load. To obtain reliable results it is necessary to use a nonlinear analysis based in time-history sequences of loads or ground acceleration.

There are two main types of dampers: the hysteretic and the viscous. The hysteretic dampers dissipate energy through the plastic deformation of metallic elements, such as steel. The viscous dampers depend on the relation between force (F), velocity (v) and the viscosity of the fluid inside the damper's piston,

defined by the C coefficient in the following equation. In this study it was used viscous dampers to strengthen the structure of the masonry building [8] [10] [11].

$$F=C.v(t) \tag{eq. 1}$$

To achieve the best performance of this technique, it was necessary to strengthen the pavement, so it behaves as a stiff diaphragm. Therefore and according to the results from the previous chapter it was used the reinforcement technique with steel ties. This choice is due to its good performance and because it is the solution which causes less damage to the original building.

6.1. Analysis performed

The existence of the interior patios allows the construction of an auxiliary structure to support the viscous dampers, with little interference with the building structural elements.

The study began with the evaluation of the performance of a solution with equal viscous dampers (with the same C value) placed in every floor. Latter and according to the results of the first test, it was possible to distribute the dampers characteristics along the building's height in order to take into account the distribution of axial forces in the devices or according to the relative displacements between levels. Finally it was used the most efficient distribution of the C coefficient from the previous tests, and tested a new way to place the dampers, to attempt to reduce its number.

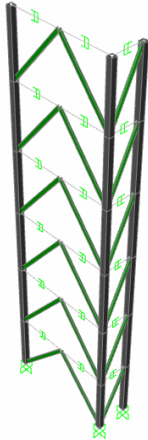


Figure 13: Auxiliary structure to support the dampers used in tests 1 to 3.

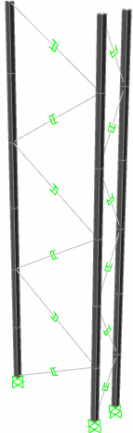


Figure 14: Auxiliary structure to support the dampers used in test 4.

6.1.1. Test 1

The first test consisted in the placement of six dampers per level, with the same C coefficient of 12 000kN.s/m, according to previous studies. The devices are placed horizontally in the plane of each floor and are connected to a metallic structure placed in the central air shaft as shown in Figure 13 [2].

The auxiliary structure is formed by four SHS300x300x30 columns placed in the four corners of the patio and connected to the building through metal pieces sealed in the masonry wall at the level of the pavement. Between floors it was placed UNP300 tilted beams, connecting the lower level and the damper

from the next level. This way the damper works horizontally as the diagonal beams transmit the displacement from the lower level to the devices plane. In the larger dimension of the interior patio it was placed two devices per wall, while in the smaller one it was only placed one per wall, due to its dimension.

6.1.2. Test 2

The second configuration tested intended to optimize the first model through the consideration of the efforts that each dampers has to bear. In Table 1 there are presented the maximum axial forces recorded in the dampers at each floor, according to the building's major direction, after test 1. The evolution along the building height allows the definition of new damping coefficients used in the test 2. Through this analysis it was intended to guaranty that the total force absorbed by the dampers was kept the same, but each one of the devices was functioning according to the level of forces it faced, enhancing the technique's performance.

<i>Floor</i>	<i>Force F (kN)</i>	<i>Relation F/ΣF (-)</i>	<i>C Coefficient (kN.s/m)</i>
5	75	0,013	9 000
4	54	0,009	6 000
3	1266	0,223	160 000
2	1534	0,270	190 000
1	1691	0,298	220 000
0	1062	0,187	150 000

Table 1: Determination of the C coefficient to use in test 2.

6.1.3. Test 3

In the third test it was intended to do a parallel analysis to test 2, but with the distribution of the C coefficient according to the relative displacements between floors. This test takes into account that the structure's response is concentrated in the resonance frequency (p), so the acceleration ($a(t)$) comes as:

$$a(t)=A.\text{sen}(pt) \quad (\text{eq. 2})$$

Integrating into velocity ($v(t)$) and displacement ($d(t)$), it is given by:

$$v(t) = -(A/p). \cos(pt) \quad (\text{eq. 3})$$

$$d(t) = (A/p^2).\text{sen}(pt) \quad (\text{eq. 4})$$

As the force in the damper is defined by (eq. 1), the forces are higher were the speed is higher and the speed is higher were the displacements are greater. Therefore it is possible to arrange the dampers characteristics according to the displacements distribution.

In Table 2 there are presented the maximum relative horizontal displacements recorded at each floor, according to the major direction, after test 1 and the equivalent C coefficients used in test 3. Comparing the two distributions of the damping coefficient in test 2 and 3 it is possible to conclude that the

distribution according to the displacements leads to a less scattered variation of coefficients than through the analysis of the forces.

Floor	Rel. Displ. (m)	Relation $d/\Sigma d$ (-)	C Coefficient (kN.s/m)
5	0,000551	0,115	80 000
4	0,000397	0,083	60 000
3	0,000876	0,183	130 000
2	0,00109	0,228	160 000
1	0,001208	0,252	180 000
0	0,000665	0,139	100 000

Table 2: Determination of the C coefficient to use in test 3.

6.1.4. Test 4

After the comparison of the three previous tests it was tested a new support structure to the dampers. The new structure was only formed by four metal columns placed in each corner of the central air shaft. The dampers were only placed one per wall and in the diagonal between floors, to absorb the difference between displacements of each level. This installation is presented in Figure 14. It was assigned a distribution of C coefficient equal to test 2, since this was the solution with the best results, as shown next.

6.2. Results

The evaluation of the performance of the four distributions of the dampers was based upon the comparison of the maximum horizontal displacements and the maximum axial forces recorded in the dampers at each level. In Figure 15 (enlarged at right) it is presented the horizontal displacement along the major dimension of the building and in Figure 16 it is presented the maximum axial forces at the dampers on each floor.

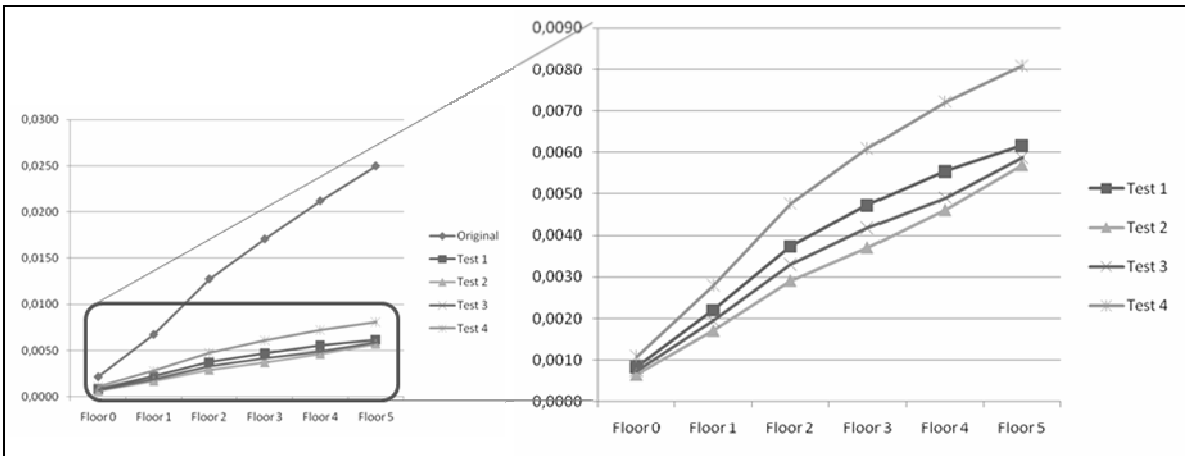


Figure 15: Horizontal displacements along the major dimension for the viscous dampers retrofit [m].

The benefits of the strengthening with viscous dampers are well acknowledgeable through the reduction in the displacements at each floor. The changes in the C coefficient achieved a better performance. This benefit is more significant when the C coefficient is distributed according to the axial forces in dampers

(test 2). The second auxiliary structure (test 4) presents better results than the original solution, although it presents worse results than the other installations. This fact can not be judged a part from being used here less twelve devices than in the other tests.

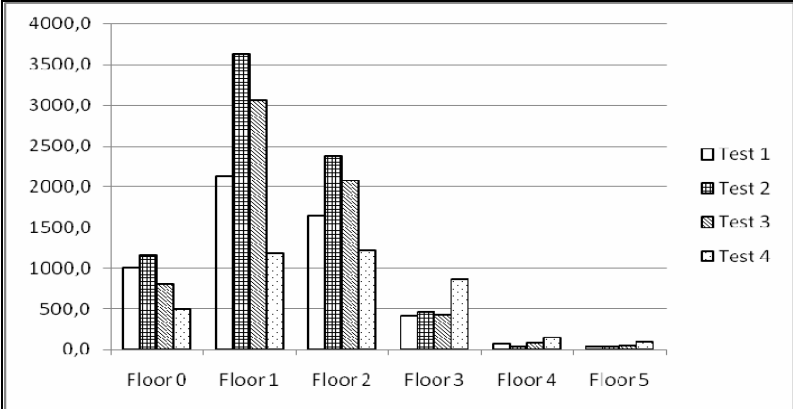


Figure 16: Axial force at the dampers according to the smallest dimension [kN].

The forces generated in the dampers (Figure 16) allows the identification of which are the more stressed floors and adequate the dampers properties to those forces. This way it is possible to enhance this technique’s performance [12].

7. Conclusions

The strengthening of the tested building, as in similar masonry buildings, should imply different levels of intervention, not only aesthetical, but also intense structural retrofit, improving its behaviour in the event of an earthquake. To execute an efficient seismic strengthening, it is necessary to guaranty the resistance of the existing structural elements, including walls and foundations. It is also necessary to improve the floors characteristics so they behave as stiff diaphragms, and therefore enabling a better distribution of forces. Other techniques may also consider to place in the structure elements with the capacity to resist or dissipate the energy generated during an earthquake. In this work it was studied the last two issues.

The pavement strengthening, to give stiff diaphragm’s properties, allows reducing the floors displacements and distributing the forces to the resistant elements in a more effective way. Among the techniques tested it is important to refer the solution with a composite slab and with steel ties. The first one allows equalizing the horizontal displacements along the pavement’s perimeter in the same way as a reinforced concrete slab would, but with the advantage of being lighter. The second technique, despite being less effective, has the advantage of being less intrusive, since it does not require removing the original pavement and takes advantage from it.

The structure’s strengthening with viscous dampers presented very positive results, as shown by a significant reduction in the horizontal displacements of each floor. By changing the dampers characteristics according to the efforts they were subjected, it is possible to achieve a better performance. In this way the more stressed levels are able to dissipate a greater amount of energy. It was also tested a second installation for the devices in which it was possible to reduce the number of

dampers. Although the results were worse, as it used fewer devices. This configuration could be interesting for a less intensive rehabilitation or a more economical one.

8. References

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