Seismic Tests to a Full Scale Monument Model of part of the São Vicente de Fora Monastery, in Lisbon.

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SEISMIC TESTS ON A FULL SCALE MONUMENT MODEL

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ABSTRACT

Pseudo-dynamic and cyclic tests on a full-scale model of part of the cloisters of the São Vicente de Fora Monastery, in Lisbon, are reported. After a first set of pseudo-dynamic and quasi-static cyclic tests performed on the original model, where local damages were observed, the model was retrofitted with four internal continuous bond anchors. A final campaign of quasi-static cyclic tests was carried out on the retrofitted model in order to investigate the effects of this retrofitting technique. The tests were carried out at the ELSA laboratory, JRC, and aimed at the characterization of the non-linear behavior of limestone-block structures under earthquake loading. Moreover, the assessment of the effectiveness of the retrofitting system was also a major objective of the test campaign. Local and overall stability of the stone-blocks, including columns and arches were assessed for large displacement amplitudes. Comparisons between the tests results before and after retrofitting allowed investigating the applicability of these retrofitting solutions and techniques for monumental structures.

1. INTRODUCTION

Public bodies in charge of the maintenance and preservation of cultural heritage are more and more expressing their willingness to assess the seismic vulnerability of important monumental structures and to develop suitable strengthening and retrofitting techniques. The joint research project, *the COSISMO project*, on seismic analysis/assessment of monuments is a first attempt to contribute to the required progress in the field. The Portuguese General-Directorate for National Buildings and Monuments (DGEMN) and the National Laboratory Civil Engineering (LNEC), in Lisbon, and the Joint Research Centre of the European Commission (JRC) setup a research programme including the following main tasks: 1) Dynamic characterization of a representative monumental structure, the São Vicente de Fora monastery in Lisbon, by *in situ* testing and numerical modelling; 2) Laboratory testing of a representative model of part of the structure, which will enable the calibration and/or development of non-linear numerical models to be used for predicting the earthquake response of such structures; 3) Development and calibration of non-linear and equivalent linear models appropriate for high intensity shaking; 4) Assessment of the seismic vulnerability of the Monastery, using the developed and calibrated models and appropriate seismic hazard characteristics; 5) Investigation of the applicability of some retrofitting solutions and techniques for monumental structures.

This paper focuses on the laboratory tests on a full-scale façade model carried out at the ELSA Laboratory under the framework of the above-mentioned project. In particular, to the recently performed tests on the retrofitted model is devoted special attention.

2. STRUCTURE AND TEST MODEL

The S. Vicente monastery (Fig. 1) represents, from the architectonic/engineering point of view, a typical monument of Lisbon, where limestone block masonry columns and arches, forming a resistant structure, are harmoniously combined with stone masonry bearing walls and ceramic dome floors/roofs.

The monument survived the catastrophic November 1st, 1755 earthquake; however, some of the effects of the strong ground shaking are still visible today. The 2 meters thick south external wall of the monastery became curved (mid-span dislocation of about 40 cm); the same happened to the west end-side external wall; and the East end-side edifice, where the Pantheons are presently located, collapsed. A quite detailed description of the damages inflicted to the monastery during the 1755 earthquake is available. Prediction of the damages to the monastery using the present modelling capabilities calibrated on the basis of the experimental results obtained by 'in situ' tests and by laboratory tests is therefore a great challenge.

Fig. 1. S. Vicente de Fora: General view (left), Cloisters internal view (right)

Test model

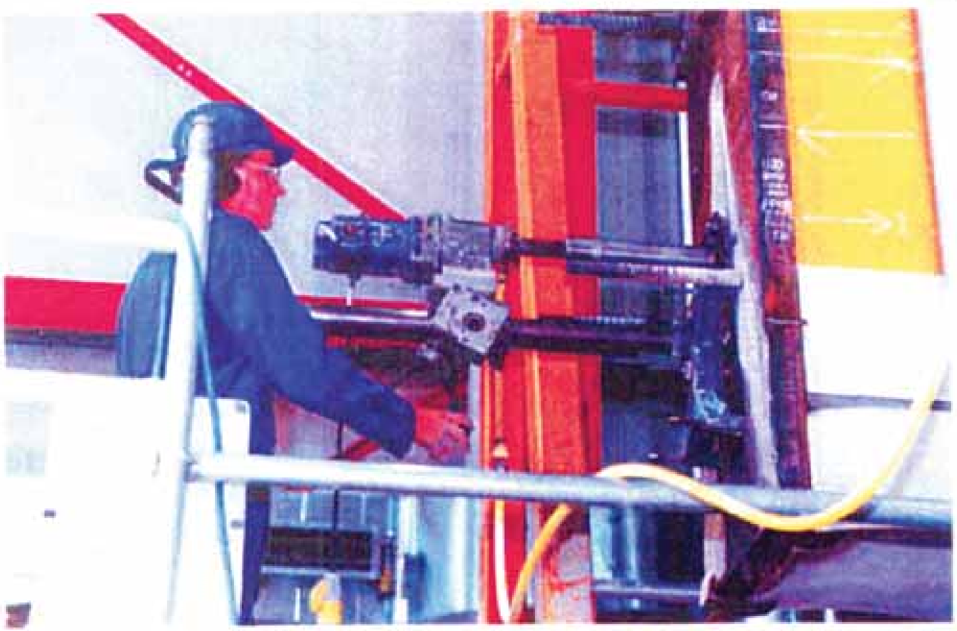
The test model (Fig. 2) was built using materials and construction techniques (stone blocks arrangement) similar to the prototype cloister facade. It is a plane structure with three stone block columns, two complete arches and two external half arches. The upper part of the model is made of stone masonry. Three millimeters thick mortar joints were assured during the construction. The model was defined taking into account the following two main aspects: it should represent typical monumental structures and it should reproduce the construction techniques (realistic, in terms of materials, scale and stone arrangements).

The 'retrofitting' technique

The model was retrofitted with four internal continuous bond anchors, two at each level with 2 meters overlapping (Fig. 2). The anchors were placed in horizontal holes drilled from each end side of the model and the grouting was carrying out in two phases. First, anchorage of the anchor was guaranteed. Then, a pre-compression of 20 kN was imposed in each steel bar and the finally grouting of the holes was done.

3. TESTING OF THE FULL SCALE MODEL

The question of using a scaled model has been raised many times by the team involved in this project and it was agreed to work with a full-scale partial test-model. It was additionally required a model to which simple and realistic boundary conditions could be applied. After several numerical simulations with existing models, see Pegon & Pinto (1998), it was decided to adopt the model given in Fig. 2. Even the boundary conditions for a long façade (periodic structure), which include equal displacement of the two lateral end-sides, zero rotation and equal vertical displacements, would had rendered the test set-up very complex. On the basis of the numerical simulations, it was concluded that post-tensioning of the two opposite end-sides of the model would lead to suitable test conditions. Moreover, such testing conditions may represent two distinct



The drilling and installation of Cintec Reinforcement Anchors retrofitted to full-scale model.

parts of a long façade: one internal (internal column and the two internal half-arches) with period boundary conditions; and the two lateral parts of the model (external columns) representing the external parts of the edifice.

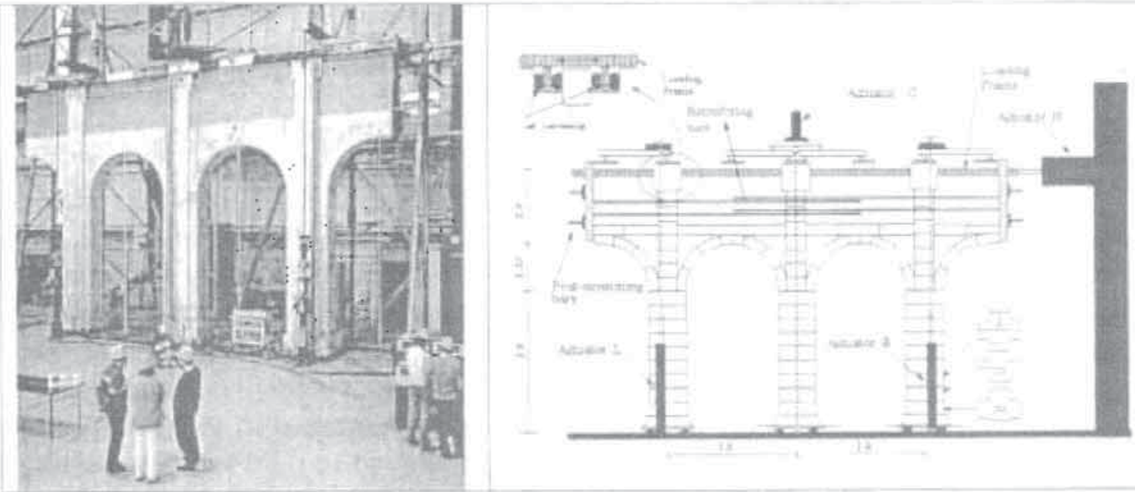


Fig. 2. Façade full-scale model in the ELSA reaction wall laboratory (left) and test set-up - schematic (right)

In addition to the lateral boundary conditions, the upper part of the façade (the height of the stone-masonry wall) was also investigated; obviously, it depends on the kind of existing opening (door, window or no opening). In the S. Vicente Monastery, at the second floor level of the cloisters exist windows, which justifies a model with a medium height wall.

Test set-up, loading devices and measuring system

Having defined the physical model and the required boundary conditions a second phase should be undertaken, which is the definition of realistic loading conditions. Two loading types are considered: the vertical loading due to the remaining upper part of the monument; and the horizontal loading resulting from earthquake excitations.

Concerning earthquake loading two important conditions should be guaranteed namely a rather uniform distribution of forces and a deformation pattern similar to the one resulting from the long façade. Therefore, overturning moment due to horizontal forces applied at the top of the model (see Fig. 2) must be compensated by means of the vertical servo-actuators located close to the columns of the model. In order to simplify the testing apparatus a constant vertical force at the central actuator was further assumed. Therefore, in addition to the vertical forces due the missing upper part of the monument, the two vertical external actuators must impose a deformation pattern dictated by the *shear-like* deformation. The control of the two external actuators is performed by imposing the following two (displacement and force) conditions

$$d_L = d_R \quad and \quad F_L + F_R = F_V \tag{1}$$

where d denotes the vertical displacement at the top of the column, the subscripts L and R stand for Left actuator and Right actuator, respectively and F_V is the total constant vertical force representing the weight of the 'missing upper part'. This additional load at the top of each column was estimated in 440 kN.

As anticipated above, the additional vertical loads represent the weight of the upper part of the monument, not reproduced in the physical model. Hence, distribution of these loads should reflect the real conditions. For a homogeneous structure a uniform force distribution would be appropriate, but, in our case, a very stiff column-arch structure (limestone blocks) is combined with masonry walls (stone masonry with poor mortars) leading to non-uniform distribution of loads. Numerical simulations with linear and non-linear models indicated that a distribution of forces, in the column and masonry parts represents quite accurately the real situation. Values of 400 and 100 kN were estimated for column and masonry parts, respectively.

The application of horizontal forces, resulting from earthquake excitations was made through an 'original' loading system, which provides uniform distribution of forces. As shown schematically in Fig. 2 (detail at top-left-side), the horizontal forces are equally distributed because they are transmitted from the loading frame to the model through water-pad bearings, which are interconnected. In fact, at each transversal beam of the loading frame, the Left-side pad bearings (L bearing in Fig. 2 – detail) communicate. Therefore, the same pressure develops and the force is proportional to the area of the pad bearing. When the horizontal loading is applied in the left to right sense, the same happens to the right side bearings, R, in Fig. 2). Such a system guarantees a pre-defined distribution of horizontal forces and avoids unrealistic local deformations at the zones of application of loads. In spite of the long and 'flexible' steel-loading frame, a specified distribution of forces can be imposed.

Concerning the measuring system designed for this test, the following types of measurement were adopted: 1) at the base of the columns, underneath the steel base plate, load cells were placed in order to obtain the evolution of vertical forces and the column-base flexural moments; 2) rotations of the column stone-blocks were measured along the height; the horizontal deformation pattern of the columns (internal and one external) can be also obtained from the set of horizontal displacement traducers placed at several levels; 3) deformation of the arches can be derived from the 3 transducers located at the contact zones and from global displacements of the arch stones; 4) vertical and horizontal relative displacements at the key-stone of the arches were also recorded; 5) deformation of the masonry part of the model (upper part) can be derived from the displacement transducers placed in this zone;6) the forces in each pre-stressing horizontal bar were monitored; in addition, application of forces (horizontal and vertical actuators) were continuously recorded as well as the controlling displacements.

4. TESTING PROGRAMME AND TEST RESULTS

Several tests were envisaged for this model apart from the initial dynamic characterization tests in order to obtain frequencies and mode shapes and evaluate damping for very low deformation levels (microns) and the initial stiffness tests. First, two pseudo-dynamic tests were performed for earthquake intensities corresponding to a low and a medium value of return period and then the model was subjected to cyclic tests with pre-defined displacement histories. After these tests the model was retrofitted and more cyclic tests were performed. The cyclic tests with pre-defined displacement histories performed on the original and retrofitted models permitted to investigate the influence of boundary conditions on the results, namely the post-tensioning forces in the horizontal ties, and the effectiveness of the retrofitting technique.

Pseudo-dynamic tests

From the dynamic characterization tests and the stiffness tests, initial stiffness was obtained, which, in conjunction with the required initial frequency of the model, dictated the mass to be used in the pseudo-dynamic tests. It is noted that for the pseudo-dynamic tests a one-degree of freedom system (1DOF) was considered. It is well known that the pseudo-dynamic test method allows such an uncoupled definition of the stiffness and mass characteristics, which is not possible in a truly dynamic test. Taking advantage of this feature, the frequency of the system was set to 4Hz, because, the experimental and analytical studies identified such a frequency value (Dyngeland *et al*, 1998). In fact, the results of the *in situ* dynamic tests of the Monastery (Campos-Costa *et al*, 1996), and of the numerical dynamic linear analyses found that the frequency corresponding to the first mode of vibration involving global deformation of the cloisters (façade) is approximately 4Hz. Consequently, a mass value of m = 400 tons was adopted for the equivalent 1 DOF system.

Two pseudo-dynamic tests were initially envisaged, corresponding to moderate and high earthquake intensities. Moreover, two types of earthquake loading were used, because two earthquake scenarios should be considered for Lisbon (far-field and near-field) with rather different spectral energy content. As shown in Fig. 3, which plots the displacement response spectra (5% damping) for the two earthquake types and for two return periods (174 and 975 years), the near-field accelerogram response spectrum contains energy in the higher frequency ranges (frequencies higher than 2 Hz), while the contrary happens for low frequencies. Based on these spectral differences and on the pre-test numerical simulations, the following strategy for the pseudo-dynamic tests was adopted. The low-level seismic test was carried out with a near-field type accelerogram (174 years return period) and the high level test was performed with the far-field 975 years return period accelerogram. The accelerograms used in the tests are shown in Fig. 3. The near-field signal, for the low-level test, is a 10 seconds duration while the high-level tests were performed for 30 seconds duration accelerograms. After the high level test, the input signal was multiplied by a factor of 1.5 and a new pseudo-dynamic test was carried out.

The results from the pseudo-dynamic (PSD) tests, in terms of global force displacement diagrams, are given in Fig. 4 for the low-level (LL) and high-level (HL) tests. It must be underlined that the LL test was carried out with a pre-compression force of about 50kN, while the HL test was carried out with a much higher compression force (350kN). During the first PSD test, the LL test, which reached a top displacement less than 8 mm, the moderate value of the compression force lead to the cracking of the model between the column and the masonry. Due to this, the model lost part of its structural 'framing' capacity and the horizontal resisting force dropped down. This case may be considered as representative of a structure without ties located at the extremity of an edifice. In order to avoid premature collapse without exploring other deformation mechanisms, the high level tests were performed with much higher compression forces. Hence, the pre-compression action through the steel bars must impose a higher strength of the upper part of the model in order to explore the 'deformation capacity' of the columns.

It is apparent from the diagrams presented in Fig. 4 that for the high-level earthquake tests the structure reaches its strength capacity and maintains it even for high deformation levels. The butterfly-type diagram is typical of these structures (a very high stiffness for low deformation levels and a 'sudden' decrease for important deformations); however, as it will be discussed latter on, the equivalent damping is quite important for the high level tests. It should be noted that the second high-level test (1.5 times the initial accelerogram (1.5HL)) led to diagrams and mechanisms similar to the HL test and to top displacements proportional to the input intensity. Peak values of top displacement of 43 mm and 64 mm were respectively recorded for the HL and the 1.5HL earthquake tests. There is another important aspect reflected in the energy diagrams given in Fig. 4. In fact it is apparent that the dissipated energy is, for all seismic tests, directly proportional to the cumulative displacement, which confirms that energy dissipation results from friction.

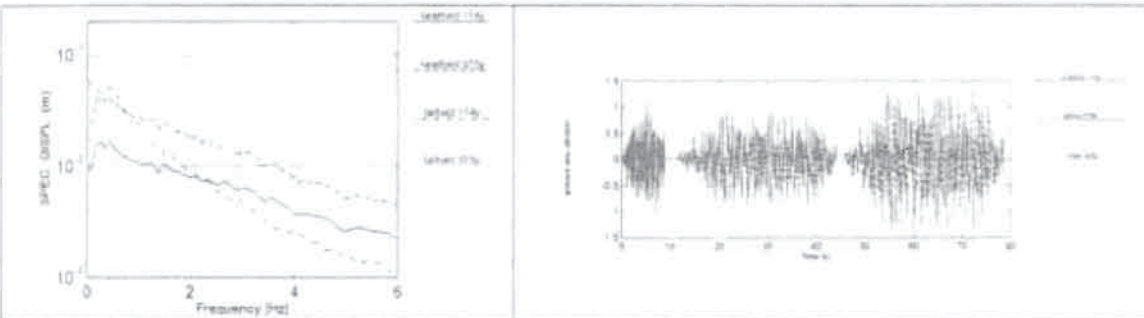


Fig. 3. Accelerograms used and Spectral displacements for far-field and near-field earthquake samples.

As already mentioned, one of the aims of the research project was to identify suitable and realistic parameters to be used in equivalent linear analyses. To this end, the earthquake test results were analyzed using time-domain identification methods (Molina and Pegon, 1998; Molina et al., 1999) and the outcome of this study is shown in Fig. 5. The left column shows the displacement response for the three earthquake tests and the corresponding evolution of eigen-frequencies and equivalent damping. The right column shows the relationship between the displacement amplitude and the eigen frequency and between the displacement amplitude and the equivalent damping. It is apparent that a linear relationship holds between displacement amplitude and equivalent frequency. Moreover, values no lower than 5% were computed for the equivalent damping for very small displacement amplitudes and the equivalent damping reaches quite high values (7–12%) for medium amplitude levels.

Quasi-static cyclic tests (original and retrofitted models)

Cyclic quasi-static tests were performed on the original and on the retrofitted model in order to obtain data to calibrate analytical models, to study the effect of the pre-compression forces on the behavior of the façade and to study the effectiveness of the retrofitting bars. Two cyclic tests were performed on the original model and three were performed on the retrofitted one. The imposed increasing-displacement histories had two constant amplitude cycles for each level and rang from 8 to 100 mm.

The first cyclic test on the retrofitted model was carried out with the pre-compression force of the last cyclic test carried out on the original model (175 kN). A displacement controlled time history was imposed with amplitudes ranging from 8 to 30 mm. Then the pre-compression force was reduced to 45 kN and a displacement controlled time history was imposed with amplitudes from 30 to 100 mm. Finally, no pre-compression was applied and a new displacement history was imposed, these time with amplitudes ranging from 8 mm to 60 mm.

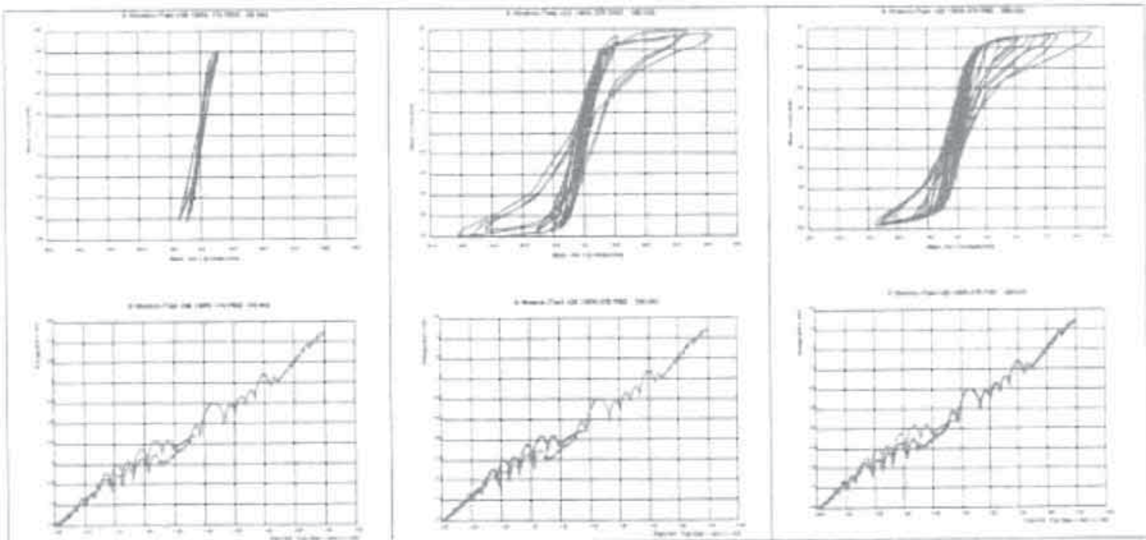


Fig. 4. Force-Displacement and Energy-Displacement (cumulative displacement) diagrams for the low-level (*Column 1*), high-level (*Column 2*) and 1.5x high level (*Column 3*) earthquake tests

Force-displacement diagrams for the cyclic tests are given in Fig. 6. The type of the diagrams is apparently the same for both cyclic tests and also for the high-level earthquake test (see Fig. 4). However, a slight difference exists between the test with high compression forces and with the medium compression forces. Both situations develop equivalent strengths for the maximum amplitude (100mm). The main difference between the two comparable cyclic tests (only the pre-compression forces are different) is the smoother transition between the two stiffness (closing and opening of the column-block joints). Therefore, one may conclude that 'a minimum' pre-compression level should be applied in order to maintain stability of the upper part of the façade and to improve deformation capacity of these structures. Furthermore, it was verified that compression forces higher than a minimum limit do not improve significantly the cyclic performance of the structure, for the experienced deformation amplitudes.

The results from the tests on the retrofitted model (with continuous-bond anchors), in terms of force-displacement, are given and compared, with the case with low compression forces, in Fig. 6 (right). Similar diagrams were obtained for both cases. However, the retrofitted case showed very important differences in terms of elongation of the upper part of the model. This is apparent in Fig. 7a), which presents the evolution of the elongation of the upper part of the model for the retrofitted and the other comparable cases. Furthermore, the initial length is almost completely recovered in the original models whilst a final permanent deformation is observed in the continuous-bond anchors case. It is shown in Fig. 7c) and 7d) that the opening takes place mainly at the masonry-column interface for all cases; i.e., deformation patterns and mechanism are similar for all cases. Therefore, the stiffness and strength of the anchors passing through the interface zones control this opening. The continuous-bond anchor (one 20mm diameter bar) and the passing bars (two 36 mm diameter bars) can develop quite different stiffness and strengths, which depend on the steel section, the bond length and the steel yielding strength. It is suppose that strong nonlinear deformations were experienced in the continuous bond anchor. However, the ultimate deformation capacity was not reached during the tests.

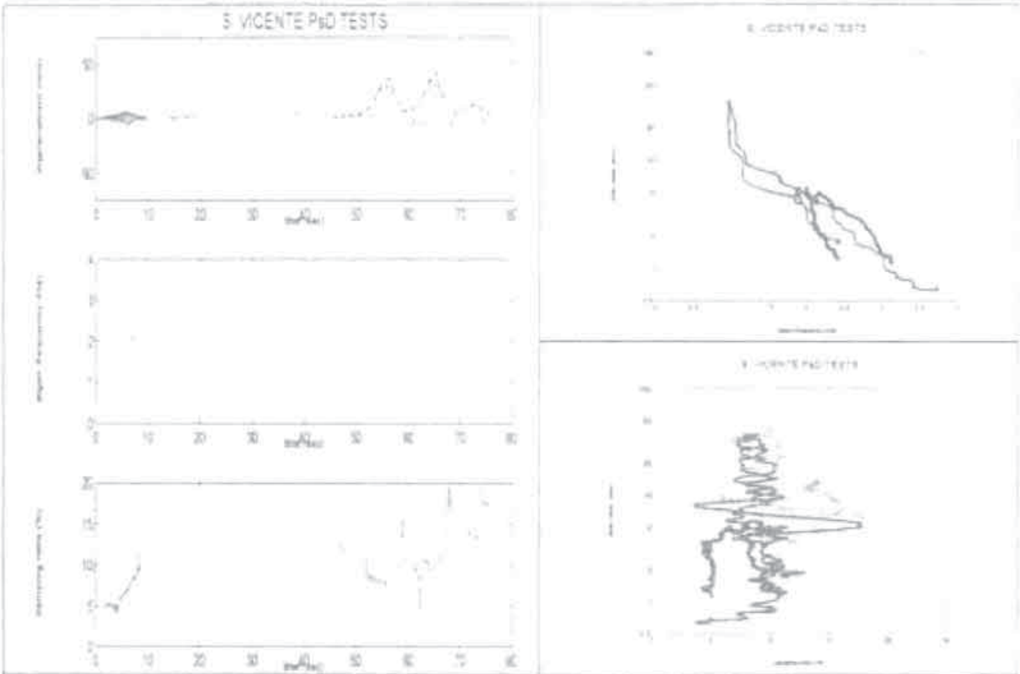


Fig. 5 History of the response to the three imposed earthquakes (left), correlation between displacement amplitude and natural frequency as obtained from the experimental response to the three earthquakes (top right) and correlation between displacement amplitude and equivalent viscous damping ratio as obtained from the experimental response to the three earthquakes (bottom right).

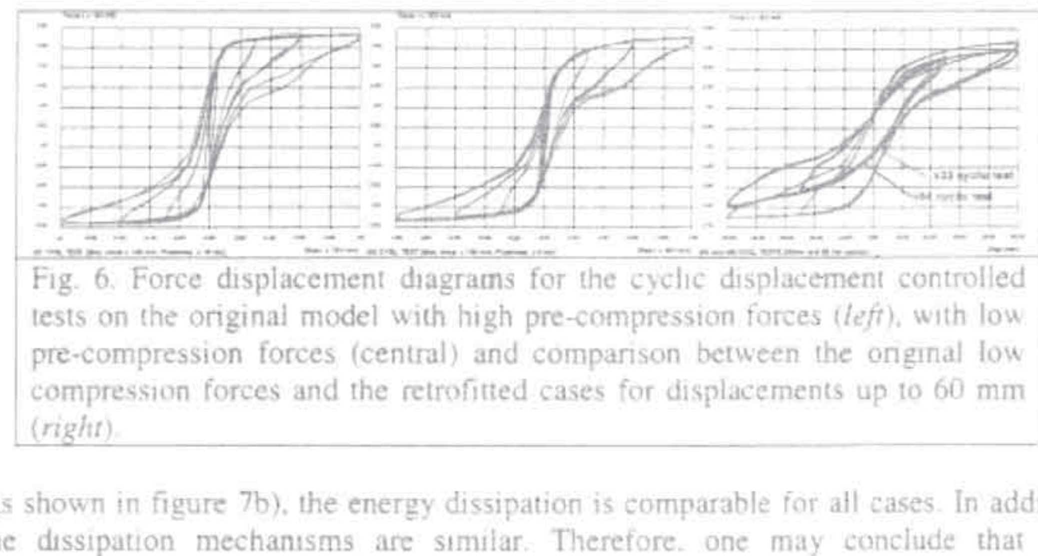


Fig. 6. Force displacement diagrams for the cyclic displacement controlled tests on the original model with high pre-compression forces (*left*), with low pre-compression forces (central) and comparison between the original low compression forces and the retrofitted cases for displacements up to 60 mm (*right*).

As shown in figure 7b), the energy dissipation is comparable for all cases. In addition, the dissipation mechanisms are similar. Therefore, one may conclude that both solutions are effective. The open issue is to develop suitable models for the design (dimensioning) of the anchors.

5. FINAL REMARKS

The tests on the model of the S. Vicente façade of the cloisters have shown the deformation capacity of the column-arch system commonly used in many monumental structures. Drifts of about 2% were imposed without loss of the load carrying capacity

Furthermore, the model has shown important dissipation characteristics due to cracking and friction. Cracking appeared at the interfaces block–masonry in the arch zones and at the interface between the upper part of the columns and the masonry (see Fig. 8). Such dissipation characteristics for important deformation levels exist thanks to the tie bars (pre-compression forces), which allow developing the required confining of the upper masonry part of the façade model.

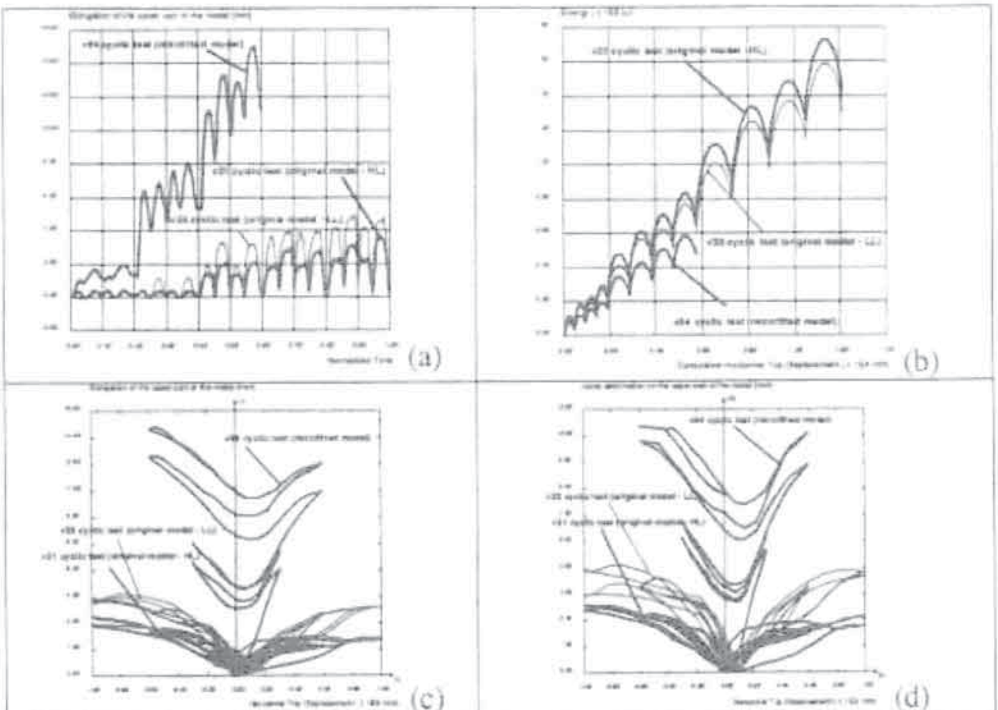


Fig. 5. a) evolution of the elongation of the upper part of the model; b) energy dissipation; c) evolution of the elongation of the upper part of the model; d) evolution of the total joint openings in the upper part of the model.

'Only' local damages were observed during the tests, namely slight dislocation of column and arch stone-blocks (15 mm maximum value), crushing and delamination of stone blocks at the most stressed contact zones, large cracks in the masonry between contiguous arch-bases and passing through the upper columns and failure (spalling) of a few limestone cover plates. Good deformation and dissipation characteristics of this type of structures are expected if a rational distribution of ties at the floor levels exists. Design and practical application of these tie systems using new analysis tools and construction techniques are under investigation. The test campaign carried out on the retrofitted model showed that the continuous-bond anchors have a rather good performance comparable with the one of the pre-compression ties. Moreover, it was apparent a 'better distribution' of the cracking in the upper part of the model. In fact, distributed cracks appeared in the stone-masonry wall and the cracks in the masonry-column interface. These tests have shown the applicability and effectiveness of such a kind of retrofitting in terms of deformation capacity and strength of the model. Another important issue was the performance of the system in the anchors overlapping zone (2m overlapping). It was observed that no damages or loss of bond appeared in this overlapping zone. Therefore, one may assume that such a system can be applied drilling holes with rather small diameters from both sides of the construction, which is an important advantage. There remains however the question of reversibility of the retrofitting solution, which should be taken into account.

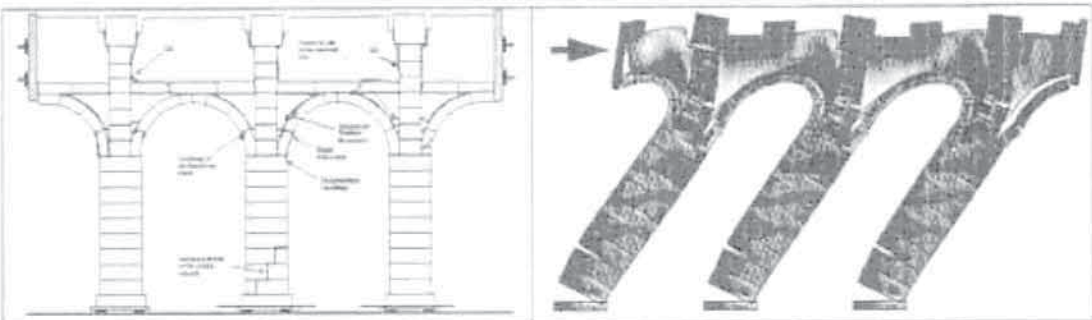


Fig. 8. Deformation pattern and damages (*top*)- analytical result (*bottom*)

6. ACKNOWLEDGEMENTS

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