THE RETROFIT OF HISTORIC BUILDINGS THROUGH SEISMIC ISOLATION: RESULTS OF PSEUDO-DYNAMIC TESTS ON A FULL SCALE SPECIMEN

A DE LUCA[1], E MELE[2], J MOLINA[3], G VERZELETTI[4] And A V PINTO[5]

SUMMARY

An experimental test program on a full scale model representing a subassemblage of the cloister facade of the Sao Vincente de Fora monastery, retrofitted through BIS, has been recently carried out at the European Laboratory for Structural Assessment (ELSA) of the JRC. In this paper an overview on the laboratory model and on the experimental results is provided. In particular, firstly the test model is described, including the geometry and mechanical properties of the masonry specimen, the design of the isolation devices, the substructuring of the isolation system and the seismic inputs adopted for the PsD tests. Then the experimental results are reported and discussed assuming as term of comparison the analogous results obtained on the "as is", fixed-base (F.B.) subassemblage model. Finally the implications of the test outcomes are emphasised and future developments of this research line are presented.

INTRODUCTION

The very recent earthquakes occurred in Italy have highlighted the particular vulnerability of historic masonry buildings and the need of addressing the problem both of the assessment of their seismic capacity and of the definition of properly designed strengthening measures. The assessment of the seismic capacity of historic masonry buildings presents objective difficulties deriving from the analytical treatment of the masonry material non-linearity, which displays nearly no-tension characteristics and requires proper experimental data for the calibration of the numerical models. In addition, the complexity of geometrical configuration of this building typology often requires the implementation of models characterized by a large number of degrees of freedom. Therefore a full nonlinear dynamic analysis of masonry buildings is not an immediate task.

Due to the inherent value of historic buildings, also the definition of retrofit interventions for improving the seismic capacity of the structure is a point of major importance and involves the research for an optimal balance between two opposite requirements, i.e. architectural preservation and the structural safety. With this regard, the base isolation system (BIS) has been recently suggested as an innovative retrofit strategy, and has been adopted for the seismic upgrading of some major monumental buildings in the U.S.A., due to the possibility of efficiently improving the seismic capacity of the building with minimal disruption to the architectural features.

FRAMEWORK OF THE EXPERIMENTAL RESEARCH AND OBJECT OF THE PAPER

With the idea of checking the application of BIS to the retrofit of Italian and, more in general, European monumental heritage, a research program, developed in Italy by the first author in the context of the "*Progetto Finalizzato Beni Culturali*" sponsored by the Italian National Research Council (CNR), is currently in progress with the following major tasks:

[1] *Università degli Studi di Napoli 'Federico II', P.le Tecchio n°80, 80125, Napoli, Italy. E.mail: adeluca@unina.it*
[2] *Università degli Studi di Napoli 'Federico II', P.le Tecchio n°80, 80125, Napoli, Italy. E.mail: elenmele@unina.it*
[3] *ELSA Laboratory, ISIS, Joint Research Centre, 21020 Ispra (VA), Italy E.mail: javier.molina@jrc.it*
[4] *ELSA Laboratory, ISIS, Joint Research Centre, 21020 Ispra (VA), Italy*
[5] *ELSA Laboratory, ISIS, Joint Research Centre, 21020 Ispra (VA), Italy E.mail: artur.pinto@jrc.it*

- study of the U.S. monument retrofit projects through BIS (De Luca & Mele, 1995);
- definition of the specific design issues and construction aspects involved in the application of BIS for monument retrofitting (De Luca & Mele, 1996; Mele & De Luca, 1997);
- selection of a representative historic building typology, the basilica plan churches, and of some specific case studies (Mele et Al., 1999 a, b);
- evaluating the possibilities of applying BIS to the seismic retrofit of the selected case studies (Mele et Al., 1998);
- numerical analyses of the case studies and evaluation of performance improvements through the comparison of response parameter values for the "as is" and retrofitted buildings. (De Luca et Al., 1999).

Furthermore an European research program, the COSISMO project, has been recently carried out by the Joint Research Centre of the European Commission (JRC) in collaboration with the Portuguese General-Directorate for National Buildings and Monuments (DGEMN) and the National Laboratory of Civil Engineering (LNEC), in Lisbon, through the following major phases:

- dynamic characterisation through numerical analysis and in situ tests of a representative monumental building, the Sao Vincente de Fora monastery in Lisbon (Dyngeland et Al.,1998);
- laboratory seismic testing of a full scale model representing a subassemblage of the cloister facade of the Sao Vincente de Fora monastery (Pinto et Al., 1998);
- linear and nonlinear analyses of the building though numerical models calibrated on the basis of experimental data (Pegon & Pinto, 1998);
- assessment of the seismic vulnerability of the building;
- evaluation of retrofitting solution and techniques.

Starting from the above research experiences, an experimental test program on the Sao Vincente de Fora subassemblage model, retrofitted through BIS, has been recently planned and carried out at the European Laboratory for Structural Assessment (ELSA) of the JRC. The tests on the base isolated model have been planned with the following aims:

- assessing the seismic behaviour of the isolated masonry subassemblage;
- evaluating the performance improvements due to the introduction of BIS, through the comparison of the response obtained for the "as is" subassemblage and for the retrofitted specimen;
- providing a benchmark for numerical modelling of isolated masonry subassemblage.

For this purpose, some pseudo-dynamic (PsD) tests have been carried out on the full scale model of the base isolated (B.I.) masonry subassemblage. In this paper an overview on the laboratory model and on the experimental results is provided. In particular, firstly the test model is described, including the geometry and mechanical properties of the masonry specimen, the design of the isolation devices, the substructuring of the isolation system and the seismic inputs adopted for the PsD tests. Then the experimental results are reported and discussed assuming as term of comparison the analogous results obtained on the "as is", fixed-base (F.B.) subassemblage model. Finally the implications of the test outcomes are emphasised and future developments of this research line are presented.

THE TEST MODEL

The Sao Vincente cloister facade subassemblage

The test model represents a subassemblage of the cloister facade of the Sao Vincente de Fora monastery, a typical monument of Lisbon, erected by the end of the 16th century and the beginning of the 18th century (figure 1). The structural system of the monastery is composed by limestone block masonry columns and arches, together with stone masonry bearing walls and ceramic domes. More details on the historic, architectural and structural characteristics of the monument can be found in (Pinto et Al., 1998). The test model is a plane structure consisting of three stone block columns, two complete arches, two external half arches, and an upper part of the arcade made of stone masonry. In figure 2 a view of the full-scale specimen in the ELSA laboratory is shown, while in figure 3 the schematic layout of the specimen, with the main geometrical dimensions, is provided. As reported in (Pinto et Al., 1998), the specimen has been built in the ELSA Laboratory by using materials and construction techniques as close as possible to the original ones of the actual cloister. The structure displays a typical architectural/structural solution, given by the assemblage of columns supporting arches and/or vaults, which frequently occurs in the masonry monumental constructions, particularly in basilica type churches, where the longitudinal sections of the church along the nave arcade present the same structural configuration and similar dimensional ratios as the test model.

Figure 1. The Sao Vincente de Fora Monastery

Figure 2. View of laboratory test model

The test model has been originally set-up for representing in a realistic manner a portion of the facade of the cloister. Thus, in order to simulate appropriate boundary conditions of a periodic structure, with relative vertical and horizontal displacements of the two ends equal to zero, horizontal post-tensioning was applied to the two ends of the model (Pinto et Al., 1998). The presence of the vertical loads resulting from the upper part of the monument, not included in the test model, was simulated by vertical actuators located close to the columns. At the beginning of the test, at the base of each column, a value of vertical load equal to 824 kN results from this load condition and from the own weight of the specimen. During the test, the vertical actuators were controlled to avoid the rotation of the upper masonry beam and, at the same time, maintain a constant total vertical load. An "original" loading apparatus was specifically designed for applying the horizontal loads on the specimen. A system of rubber cushions filled with water and communicated by tubes allowed to apply evenly distributed forces at the upper part of the masonry beam (Pinto et Al., 1998), while the horizontal displacement at the center of it was controlled for the pseudo-dynamic integration.

The total inertial mass moving with the masonry beam was estimated of 400 tons which included masses of other parts of the monument which were not resting on the columns. The first period of the masonry subassemblage, identified through numerical analyses and in situ dynamic tests, was originally set at $T_{F.B.ini}$=0.25 s. This corresponded to the initial stiffness of the specimen and the mentioned mass of 400 tons. From a cyclic test at 8 mm amplitude carried out after the PsD tests on the F.B. specimen, a value of the secant stiffness KF.B.=20.94 kN/mm has been derived. Thus the period of the F.B. equivalent SDOF system, at the time of the tests described in this paper, has been obtained from the value of $K_{F.B.}$, providing: $T_{F.B.}$=0.87 s. The significant elongation of the period with respect to $T_{F.B.ini}$ is related to the extensive cracking and local damage occurred in the previous F.B. tests. The instrumentation of the specimen provides continuous recording of the forces applied in the vertical and horizontal actuators and of the controlling displacements.

The isolation system

The devices selected for the isolation system are High Damping Rubber Bearings (HDRBs). In order to introduce the isolation system, a rigid connection among the column bases has been provided through a reinforced concrete 0.40x0.80 m beam, as reported in the schematic layout in figure 4.

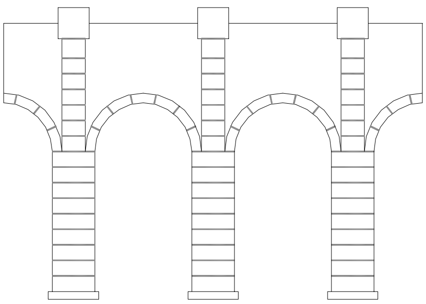


Figure 3. Layout of the masonry subassembalge

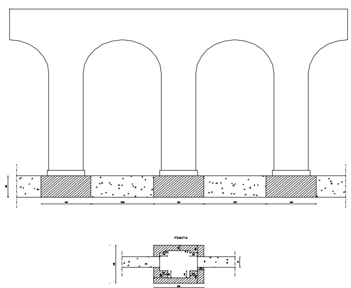


Figure 4. Schematic layout of the r.c. beam connecting the column bases

The mass of the beam is equal to 30 tons. Different working hypotheses on the isolation period, average compression stress and shear modulus of the rubber have been considered in the design phase. The design procedure consisted in the main following steps:

- selection of the isolation period T_I (2.5 ÷ 3.5 s);
- computation of the isolation stiffness K_{BIS} required for shifting the total mass M_{tot} (430 tons) of the test model (masonry subassemblage, r.c. connecting beam and isolation devices) to the selected value of T_I : $K_{BIS} = 4\pi^2 M_{tot} / T_I^2$
- computation of the single device cross section area A and then of the device diameter Φ on the basis of an allowable level of average compression stress σ_v (3 ÷ 10 MPa) under the vertical load acting at the base of each column W_{col} (824 kN): $\Phi = (4(W_{col}/\sigma_v)/\pi)^{1/2}$
- definition of the rubber height h_{rub} of the single device, which provides, for different values of the shear modulus G (0.4 ÷ 0.8 MPa), the required lateral stiffness $K_{dev} = K_{BIS}/3$: $h_{rub} = G\,A / K_{dev}$
- check of the value of the aspect ratio Λ of the device (3 ÷ 5), in order to guarantee global stability: $\Lambda=\Phi/h_{rub}$
- definition of the single rubber layer thickness t_{rub}, on the basis of a selected value for the shape factor S of the device (≥ 30): $t_{rub} = \Phi / 4\,S$
- check of the outcomes of the design process and choice of the isolation devices.

The isolation system chosen at the end of the design process consists of three HDRBs, one under each column, with a diameter of 400 mm and 108 mm of total rubber height. The rubber is characterized by a shear modulus G equal to 0.8 MPa, and the working stress level under vertical load is 6.7 MPa. This isolation system, at a shear deformation γ equal to 100% of rubber height, shifts the first period of the subassemblage to 2.5 s.

A substructuring method (briefly described in the following paragraph) has been used in the PsD testing procedure, thus the isolator devices have not been directly settled under the masonry subassemblage: the two parts of the B.I. specimen (masonry and isolators) have been separately tested and the interaction between them has been analytically accounted for. Thanks to this substructuring arrangement, different scales have been used for the two parts (masonry and isolators) of the test model: the monument subassemblage is full-scale, while for the isolation system four HDRBs (rubber EN60) with a diameter of 250 mm and 66 mm of total rubber height have been used instead of the designed three devices Φ400 with 108 mm of rubber height. Measured forces and displacements are numerically corrected during the testing procedure in order to account for the different scales of the two parts constituting the specimen. The substructuring strategy adopted for the PsD tests also allows to account for different values of the shear modulus G, thus in the tests two different rubber stiffnesses have been simulated: a "stiff" rubber, characterized by G=0.8 MPa, which shifts the period to 2.5 s, and a "soft" rubber, with G=0.4 MPa, which shifts the period to 3.5 s.

A typical cyclic response of the HDRB Φ250 is provided in figure 5, while in figure 6 the shear modulus and the damping ratio values, obtained in the device characterization tests, are reported as a function of the strain amplitude.

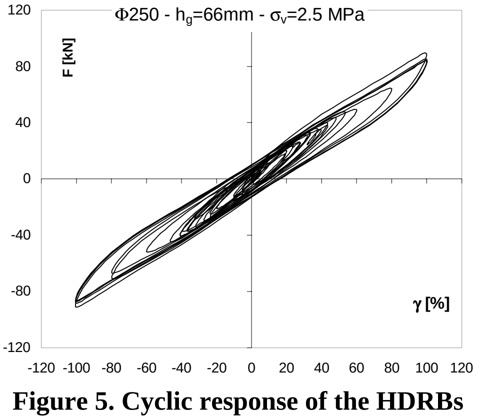


Figure 5. Cyclic response of the HDRBs

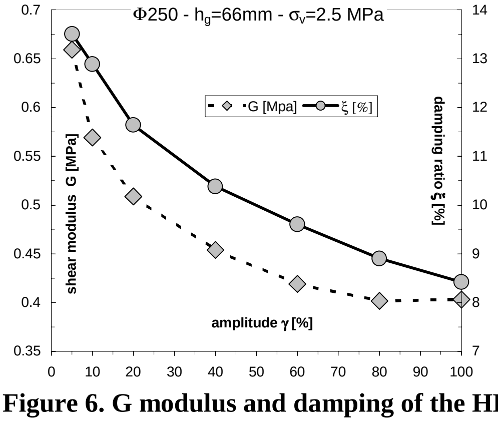


Figure 6. G modulus and damping of the HDRBs

The substructuring method of isolators

The ELSA laboratory team has set up a substructuring method for performing PsD tests on base isolated structures. In fact, as reported in (Molina et Al., 1996) the flexibility of the PsD technique allows to divide the restoring force system in several substructures, which can be separately tested or numerically simulated. Thus the experimental arrangement has not required the direct settling of the isolators under the masonry subassemblage, since the isolation system has been simultaneously, but separately, tested, and the interaction between the masonry specimen and the isolators has been analytically considered (Figure 7). Then, only base-relative displacements are imposed to the masonry specimen and the restoring force for the base is obtained as the static reaction to the measured forces on the masonry specimen plus the force at the isolator specimens when submitted to the specified displacement. The described substructuring testing technique was particularly advisable in this specific case since the Sao Vincente de Fora subassemblage specimen, in the "as is" F.B. condition, had been extensively tested in previous experimental programs (Pinto et Al., 1998) and therefore it was already clamped to the strong floor of the laboratory.

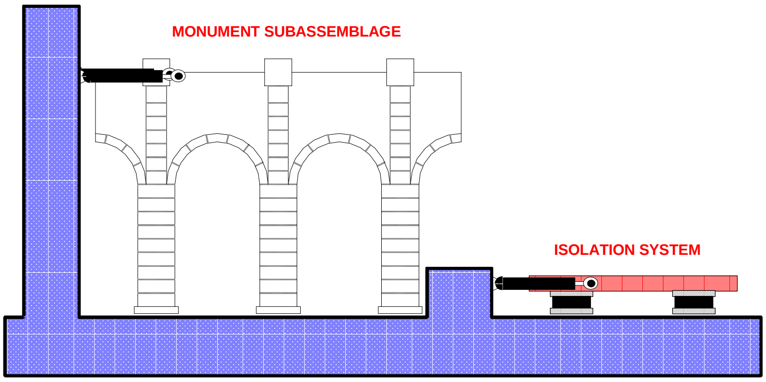
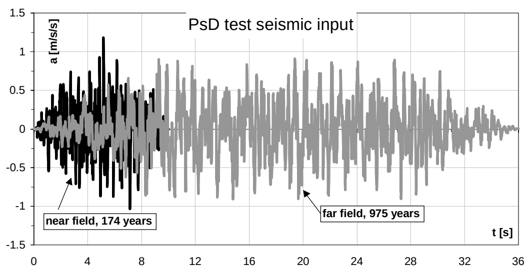


Figure 7. Substructuring method adopted in the PsD tests.

A method for compensating strain rate effects on the elastomeric bearings due to the PsD testing speed, proposed by (Gutierrez & Verzeletti 1993) and already utilised in previous tests performed at the ELSA laboratory (Molina et Al., 1996), has been adopted during the tests herein described.

The seismic input

The PsD tests have been carried out with two acceleration histories, reported in figure 8, already utilised for the previous tests on the "as is" (fixed base) masonry specimen, respectively corresponding to a moderate (174 years return period) and high (975 years return period) earthquake intensity. In the tests, the high intensity acceleration history has been applied to the specimen scaled at 1.5 times. The seismic inputs correspond to two earthquake scenarios envisaged for Lisbon, namely a near-field and a far-field type signal, characterised by major energy content in different frequency ranges and by different duration, respectively 10 and 30 s. In the following the moderate-intensity (174 years return period), near-field type seismic input is appointed as lip174, while the high-intensity (975 years return period), far-field type seismic input is appointed as lia975.



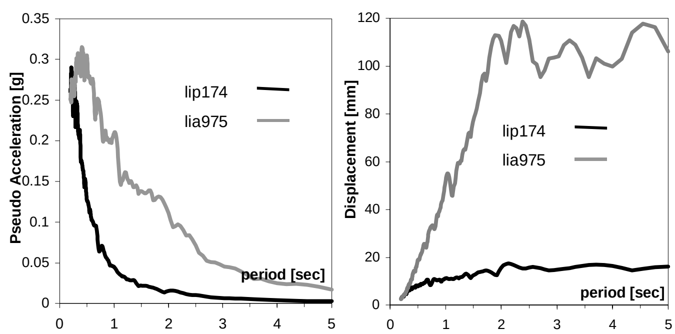


Figure 8. Acceleration histories **Figure 9. Acceleration and displacement spectra.**

In figure 9 (a) and (b) are provided the response spectra, respectively in terms of acceleration and displacement. It is evident that the near-field seismic input has the highest spectral ordinates in the low-period range, while for periods greater than 1 s a considerable reduction of the accelerations, with not significant increase of the displacements, can be derived. The far-field signal presents a less sharp reduction of acceleration in the long-period range than the near-field signal and very large displacements (larger than 100 mm) for periods greater than 2 s. Since the BIS has been designed for shifting the first period of the subassemblage to 2.5 s (with HDRBs characterised by G=0.8MPa) and to 3.5 s (with HDRBs characterised by G=0.4MPa), the maximum reduction of forces and deformation in the masonry part is expected under the near-field type input. The reduction of the forces in the elevation part is due to the effects of the period shifting and of the isolators damping. The order of magnitude of the structural response reduction can be estimated on the basis of the ratio between the spectral acceleration at $T_{F.B.}$ and T_I. In table 1 the values of the ratio $R_{SA}=SA(T_{F.B.}, \xi_{F.B.})/SA(T_I, \xi_I)$ are reported for the two seismic inputs, assuming T_I equal to 2.5 and 3.5 s, ξ_I equal to 5% and 10%, $T_{F.B.}$ equal to 0.87 and $\xi_{F.B.}$ equal to 5%.

Table 1

Seismic input	$R_{SA}=SA(T_{F.B.}, \xi_{F.B.})/SA(T_I, \xi_I)$			
	ξ_I = 5%		**ξ_I = 10%**	
	T_I=2.5 s	**T_I=3.5 s**	**T_I=2.5 s**	**T_I=3.5 s**
Lip174	5.5	10.2	7.5	11.7
Lia975	2.9	6.3	4.2	7.6

EXPERIMENTAL RESULTS

The PsD test results concern displacement and restoring force histories of the specimen both at the isolation level and at the top level of the masonry subassemblage. In the following, the four PsD tests are appointed as:

(a) seismic input: 100% lip174, rubber: G = 0.8 MPa
(b) seismic input: 150% lia975, rubber: G = 0.8 MPa
(c) seismic input: 100% lip174, rubber: G = 0.4 MPa
(d) seismic input: 150% lia975, rubber: G = 0.4 MPa

A large difference between the two displacement histories, at he isolation level (0) and at the masonry level (1), is evident from the test results. Larger absolute displacements at the level 0 as well as lower relative base-to-top displacements in the masonry subassemblage have been observed in the tests with rubber devices characterized by G=0.4 MPa (test c and d) than in the tests with G=0.8 MPa (test a and b). The maximum displacements experienced by the specimen in the four PsD tests are close to the spectral displacements, respectively at T=2.5 s for the tests with the stiff rubber devices (G=0.8 MPa) and T=3.5 s for the tests with the soft rubber devices (G=0.4 MPa). The experimental displacements and the spectral values (ξ_I =10%) are compared in table 2.

Table 2

Seismic input	Displacements [mm]			
	Spectrum		**Test**	
	T_I=2.5 s	**T_I=3.5 s**	**G=0.8 MPa**	**G=0.4 MPa**
Lip174	11.5	14.2	10.8 (lev.0) 11.6 (lev.1)	14.4 (lev.0) 14.7 (lev.1)
Lia975	113	136	108 (lev.0) 130 (lev.1)	138 (lev.0) 149 (lev.1)

From the restoring forces registered at the isolators level (level 0) and at the masonry subassemblage level (level 1), it is possible to derive the cases in which the BIS has the maximum efficacy in reducing the forces in the elevation parts. As expected the near-field, lip174, seismic input (test a and c) leads to the lower values of forces in the upper part (F_{max} = 58 kN (a), 28 kN (c)), particularly when "soft" (G=0.4 MPa) rubber devices are adopted. Significantly higher forces have been registered in the specimen under the far-field, lia975, seismic input: the maximum value reaches 322 kN in the case (b) (rubber with G=0.8 MPa) which is reduced to 226 kN in the case (d) (rubber with G=0.4 MPa).

COMPARISON TO THE FIXED BASE EXPERIMENTAL RESPONSE

In the figures 10 (a-d) the experimental cyclic response is plotted in terms of global restoring force vs relative displacement between the isolators and the masonry subassemblage. In the same charts the analogous experimental responses obtained in the tests on the F.B. specimen, carried out by (Pinto et Al.), are reported. The four charts refer to the different PsD tests, as already specified in the previous paragraph. The comparison between the response of the B.I. model and the response of the F.B. model allows evaluate in quantitative terms the improvement deriving from the introduction of BIS. The observations concerning the effect of the seismic input and of the rubber shear modulus on the response of the specimen are confirmed with reference to the global force vs displacement cyclic curves.

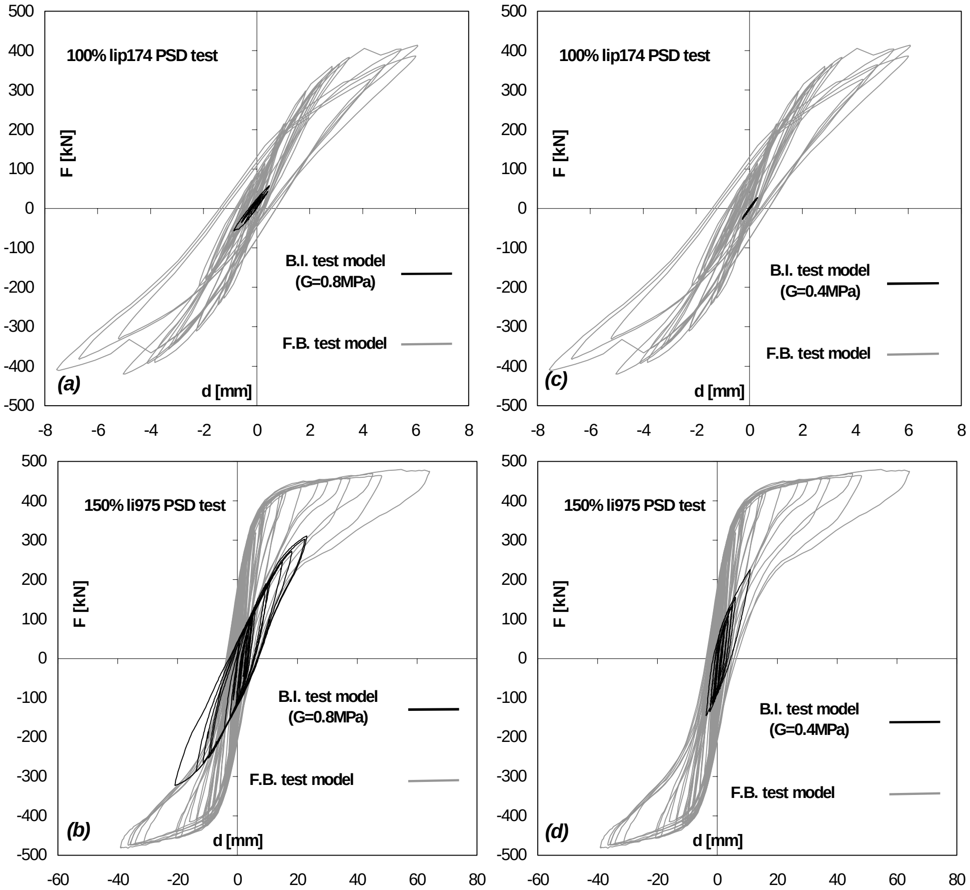


Figure 10. Force displacement cyclic response: comparison of F.B. and B.I. specimens.

The comparison between the test cyclic response of the F.B. and B.I. specimen reported in figure 10 (a) provides reduction factors for the forces equal to 7.25 and for displacements equal to 8.9; these reduction factors are remarkably increased considering the results obtained in the test with soft rubber devices (G=0.4 MPa), as can be observed from figure 10 (c): the ratio of F.B. to B.I. maximum force in the elevation is equal to 15, while the same ratio in terms of maximum displacement reaches 24.5. Lower reduction factors are obtained in the cases of figure 10 (b) and (d), which report the results of the tests with the far-field lia975 seismic input amplified 1.5 times. The ratios of the maximum displacements (F.B. / B.I. values) provide 2.8 and 5.8 respectively for stiff (b) and soft (d) isolation devices, while the same ratio in terms of forces in the masonry part respectively provides: 1.5 (b) and 2.1 (d). In table 3 and 4 the above reduction factors are summarized together with the maximum absolute values of forces and displacements experienced in the tests by the F.B. and the B.I. specimens.

Table 3

Seismic input	Force [kN]				
	F.B.	**B.I.**		**F.B. / B.I.**	
		G=0.8 MPa	**G=0.4 MPa**	**G=0.8 MPa**	**G=0.4 MPa**
Lip174	420	58	28	7.25	15
Lia975	481	322	226	1.5	2.1

Table 4

Seismic input	Displacements [mm]				
	F.B.	**B.I.**		**F.B. / B.I.**	
		G=0.8 MPa	**G=0.4 MPa**	**G=0.8 MPa**	**G=0.4 MPa**
Lip174	7.55	0.85	0.3	8.9	24.5
Lia975	64.1	23.1	11.0	2.8	5.8

CONCLUSIVE REMARKS AND FURTHER DEVELOPMENTS

In this paper the pseudo-dynamic tests carried out on the full scale model of a base isolated (B.I.) masonry subassemblage have been presented. An overview on the laboratory model and on the experimental results has been provided, assuming as term of comparison the analogous results obtained on the "as is", fixed-base (F.B.) subassemblage model.

The experimental results suggest a particularly improved behaviour of the B.I. specimen. The masonry part above the isolators experiences displacements up to 24 times smaller than the ones registered in the F.B. test model. The forces are reduced up to 15 times. No further cracking and damage in the masonry elements of the specimen, which extensively occurred during the previous F.B. tests, have been observed following the B.I. tests. These test results, even though confirming the well known 2DOF-type behaviour of base isolated structures, are particularly significant since have been obtained on a full scale model, and demonstrate that the BIS can efficiently reduce force and deformation in monumental buildings. In addition such results are currently being used as reference for calibrating numerical models.

The research line on seismic protection of historic buildings through BIS, of the CNR “Progetto Finalizzato Beni Culturali”, has proved to be very promising, and following these experimental tests, European research projects are presently being planned on this subject.

Acknowledgments : For the first two authors, this research is supported MURST PRIN. The tests have been possible thanks to the high and enthusiastic skill of the other three authors. The photos have been kindly provided by Dr. A. Gago.

REFERENCES

De Luca,A., Mele,E. (1995). L'isolamento sismico nel restauro di edifici storici. Atti del 7° Convegno Nazionale "L'Ingegneria Sismica in Italia", Siena, 25-28 settembre, 1995, Vol.3°.

De Luca A., Mele E. (1996). The seismic isolation in the retrofit of historic buildings. Proceedings of USA/ITALY Seminar Seismic Restoration of Historic Buildings, Los Angeles, USA, July 1996.

Gutierrez, E., and Verzeletti, G. (1993). Possibilities of vibration isolation testing at the ELSA laboratory of the Joint Research Centre. Proceedings of the XII Post-SMiRT Conference on Isolation, Energy Dissipation and Control of Vibrations of Structures. Capri.

Mele E., De Luca A. (1997). Seismic strengthening of cultural heritage through base isolation: some case studies. Proccedings of Internationa Conference New Technologies in Structural Engineering , Lisbon, Portugal, July 1997.

Mele E., Modano M., De Luca A. (1998). The seismic retrofit of historic masonry buildings through BIS: preliminary analysis for application to church typology. Proc. of MONUMENT '98 Workshop on Seismic Performance of Monuments, Lisbon, Portugal, November 1998.

Mele E., De Luca A. (1999). Behaviour and modelling of masonry church buildings in seismic regions, Proc. 2nd Int. Symp.on Earthquake Resist. Engrg. Struct. ERES ’99, Catania, Italy, June 1999.

Mele, E., Giordano, A., De Luca, A. (1999). Nonlinear analysis of some typical elements of a basilica plan church. Proc. 2nd Int. Symp.on Earthquake Resist. Engrg. Struct. ERES ’99, Catania, Italy, June 1999.

Molina, J., Gutierrez, E., Magonette, G., Negro, P., Tirelli, D., Verzeletti, G. (1996) Pseudo-dynamic simulation of base isolation on a reinforced concrete building by means of substructuring. 1st European Conference on Structural Control, Barcelona, May 1996.

Pegon, P. & Pinto A. Numerical modelling in support of experimental model definition-The S.Vincente de Fora model. *Proc. MONUMENT '98 Workshop on Seismic Performance of Monuments*, Lisbon, Portugal, pp. 3-12, 1998.

Pinto, A.V., Verzelleti, G., Molina, F.J., Plumier, C. Seismic tests on the S.Vincente de Fora model. *Proc. MONUMENT '98 Workshop on Seismic Performance of Monuments*, Lisbon, Portugal, pp. 33-46, 1998.