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Seismic Assessment by Numerical Analyses and Shaking Table Tests for Complex Masonry Structures: the Hagia Irene Case Study

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Abstract The document concerns with the design and study of a large masonry model of Hagia Irene in Istanbul. The goals of experimentation were to acquire a deeper structural behaviour knowledge than that possible to acquire with numerical models only and to assess the effectiveness of the consolidation works foreseen on Hagia Irene. The scale factor chosen is equal to 1:10. The tests were arranged into two different phases: for not reinforced and reinforced scale model .The displacements of the markers placed on the structure were registered by an innovative monitoring technique measuring 3D motion time histories.

Keywords: Masonry structure, large scale model, shaking table test, energy evaluation, power spectral density

Introduction

The shaking table experimentation was carried out, by SPC-SC&A-OSM Joint Venture, in the field of seismic strengthening program for cultural heritage buildings, under the responsibility of the Turkish Ministry of Culture and Tourism and IPKB (Istanbul Project Coordination Unit).

Because of Hagia Irene is structurally weaker for seismic actions in transversal direction, the shaking table test were designed for the transversal actions only. Thus the design of the scale model included some geometrical simplifications. The most important one was the removal of apse and narthex, both replaced by walls having the same stiffness. Moreover, the two domes were replaced by two cylinders with the same mass and centre of gravity. The model was constructed using three different materials. Two of these materials was used to simulate, once adequately scaled, the mechanical behaviour of the masonry. The third material was designed for the equivalent wall of the apse side (refer with Fig. 3).

Figure 1: External and internal view of Hagia Irene

The acceleration time-histories to be applied to the scale model were determined based on the studies carried out by Bogazici University about the seismicity of Istanbul.

In Phase 1, the scale model was subject to the same time history of acceleration but having each time increasing amplitude up to the reaching of relevant damages in the model. The strengthening works realized after Phase 1 consist of horizontal shear stiffening elements at the level of galleries floor and at the roof level, just over the longitudinal lateral main vaults. In Phase 2, the scale model was subject to time-histories, up to points of significant damage again. This phase allowed to assess the effectiveness of consolidation works.

Geometry and Simplification of Physical Model

The physical model was built with a scale factor 1:10. Main longitudinal and transversal dimensions of the monument are respectively 58m and 32m; the side of the squared shaking table of the Enea-Casaccia Research Centre is 4m; for this reason it was decided to implement a scale model of the real construction taking into account the removal of the apse and the removal of the narthex, both replaced by walls having the same stiffness. A further simplification with respect to the actual monument led to the replacement of the two domes with two cylinders characterized by mass and centre of gravity approximately the same than the ones of the real structures. This approximation, which is a child of considerations regarding construction feasibility, was deemed to be acceptable since the final goal of the testing is the determination of the global structural behaviour of the vertical bearing structures and of the main system of arches supporting the domes and not the deep analysis of stresses and strain characterizing the local behaviour of the two domes. In order to further simplify the construction process of the model, “matronei” galleries and the structures underneath were replaced by lighter steel structures where, subsequently, were housed adequate masses to reflect the whole original self weight of the missing structures (refer with fig. 2). Last it was also decided not to implement in the physical scale model the windowed walls of the lateral facades: this choice not only allowed an easier construction of the model but also provided for an easier central arch monitoring by cameras during the tests (refer with fig. 3).

Figure2: Additional masses in the physical model

Material properties			
	γ [kN/m^3]	E [daN/cm^2]	σ_C [daN/cm^2]
Material 1	22	6000	10
Material 2	18	3500	18
Material 3	22	7000	22

Figure 3:The scale model and material property

Scaling Factors and Material Design of Physical Model

To obtain the scaling factors of physical model the Buckingham theorem (or theorem π) was used; according to this theorem, if a dimensional system with "n" physical quantities includes "k" fundamental quantities, each physical phenomenon can be represented by m=n-k non-dimensional relationships. Considering the scale factor of the model to be tested on the shaking table (1:10) and taking into account the fact that, with respect to the actual situation, were set up the acceleration of gravity "g", the seismic acceleration "a" and the mass density "ρ", it was finally possible to determine the scaling factors shown on Table 1.

As for the construction of the PM, the choice of using mortars rather than masonry is justified by the observation that most part of the existing construction is made up of mortar, because of the presence of very thick joints and because the masonry is often made up of an internal core rich in mortar with stones of different sizes. A model made of a continuous material also allows a better identification of the crack pattern.

Table 1: Scaling factors

Physical Quantities	*scale*
Length, displacements	0,10000
Stresses	0,10000
Elastic Modula	0,10000
Frequencies	3,16228
Time, Period	0,31623
Masses, weights	0,00100

Monitoring System and Record of Frequencies

The acceleration transmitted by the base of the model during the different steps of the tests were registered using accelerometers located on the shaking table. Other accelerometers were applied on the model, together with an innovative high resolution 3D optical movement detection and analysis that track the dynamic displacement of the selected points of the scaled mock-up during the earthquake shake table tests. A cluster of high resolution (up to 4Mpixel) Infrared Cameras (Fig. 4) has been used to measure accurate 3-D positions of the markers placed on the structure during the seismic tests (refer with Fig. 4). The signals recorded at the different accelerometers permitted to evaluate the modal shapes of the physical model. Fig. 5 shows the frequency content recorded during the test n°1 (Phase1, PGA=0.02g, structure not damaged). Fig. 6 shows the power spectral density and frequency contribution to the RMS values of the acceleration on the base table and at the central arch springing (Phase 1,test with NPA 0.29g).

FEM Modal Analysis: FEM of Real Structure (FEM-RS) and of Physical Model (FEM-PM)

It was very significant to compare the results of the modal analysis of the FEM model of the real structure and the one of the scale model. The finite element models were developed in ALGOR, using bricks (finite three-dimensional elements). Bricks are elements counting six or eight nodes located in the three-dimensional space and characterized by three translational degrees of freedom only per each node (i.e. rotational degrees of freedom, are not considered). Considering the first modes that in both cases excite the structure transversally and operating the proper scaling of the periods, for the model of the real structure a period of 0.12s is obtained, for the model of the scale construction a period of 0.10s is obtained. The scale model turns to be slightly stiffer. Participating masses of the models are similar: 69.4% for the real construction, 63.7% for the scale model. As for the modal shapes it is to be noted that similar results are obtained for both models (fig. 7).

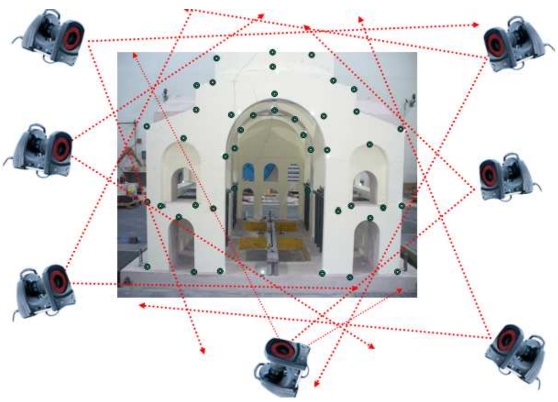

Figure 4: High resolution 3D-Vision cameras at the ENEA Casaccia Laboratory

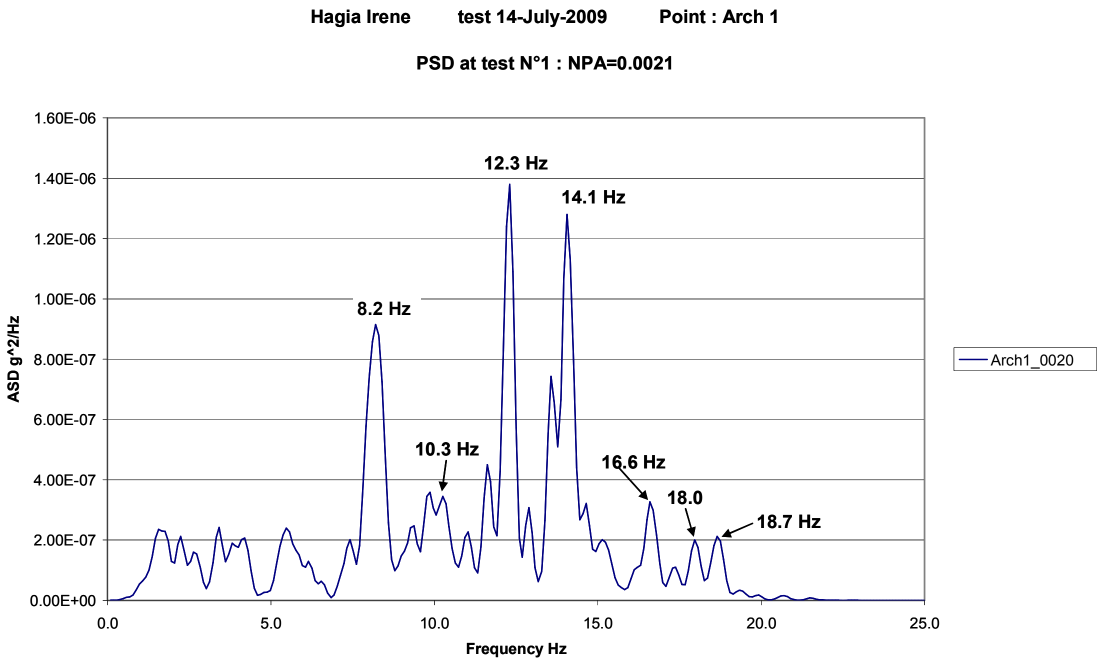

Figure 5: Frequency content recorded during the test n°1 (Phase 1)

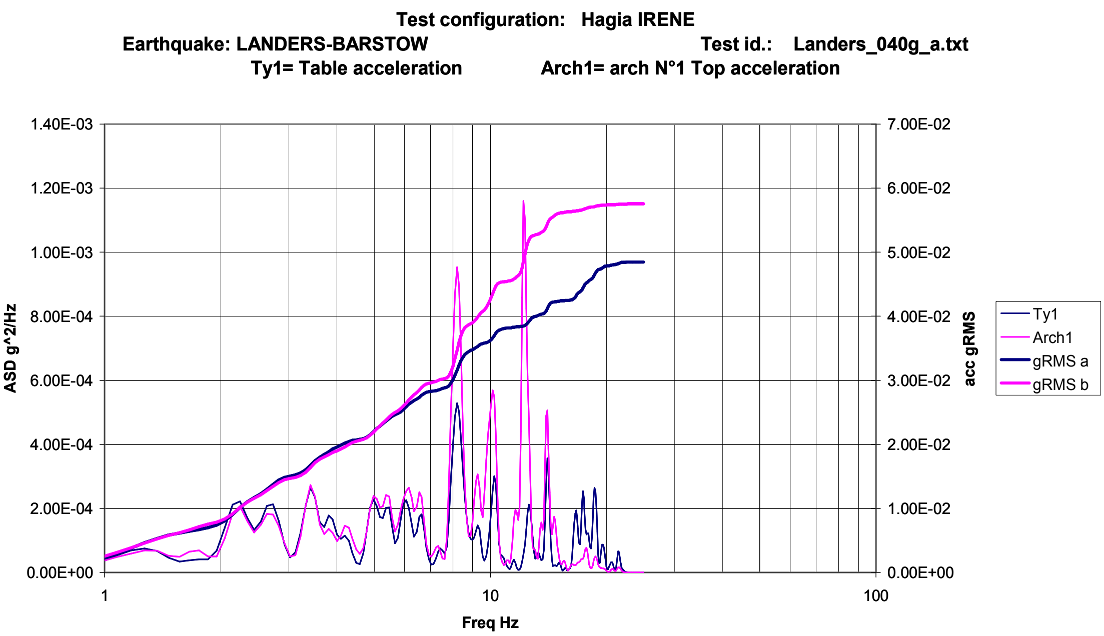

Figure 6: Power spectral density and frequency contribution to the RMS values of the acceleration, test with NPA 0.29 g (Phase 1), at the base of the table and at the central arch springing

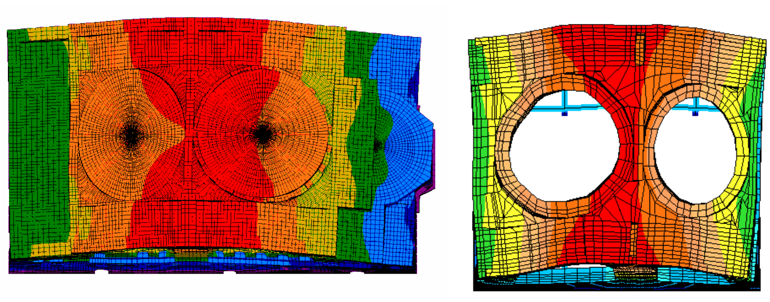

Figure 7: Comparison between the most significant mode of real structure and scale FEM models

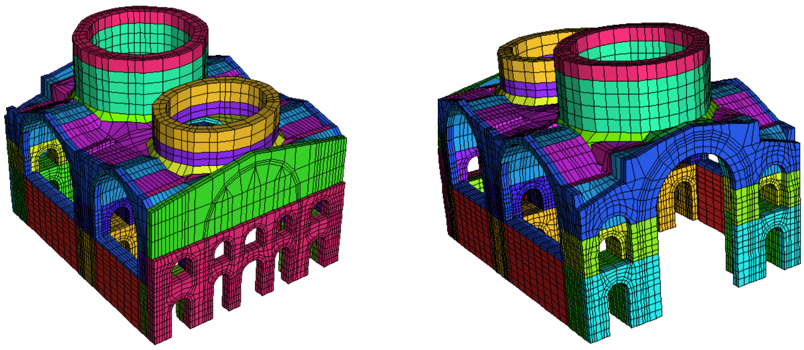

Figure 8: Finite element model of the structure

The modal analysis of FEM-PM (fig. 8) allowed to evaluate the effectiveness of the finite element model to evaluate the dynamic behaviour of the scale structure.

The following table shows the excellent agreement between the frequencies experimentally determined (structure not damaged) and the modal analysis results.

Table 2: Experimentally results (PM: physical model) and modal analysis (FEM-PM).

FEM-PM		PM	
MODE	FREQUENCY OF FEM-PM (Hz)	MODE	FREQUENCY OF PM (Hz)
1	8.81	1	8.20
2	10.37	2	10.30
		3	12.30
3	14.12	4	14.10
4	16.89	5	16.60
5	17.22	6	18.00
6	20.26	7	18.70

Frequencies Shifts of the Scale Model

The following fig. 9 shows the PSD spectrogram during the sequence of test of the Phase 1 (on the left) and Phase2 (on the right).With reference to Phase 1, it is evident a first frequency shift at the test of PGA 0.29g when the first damaging occurs and most relevant frequency shift at the test of PGA 0.32g. The observation of spectrogram of Phase 2 shows a first frequency shift for a PGA 0.42g and the most important shift with 0.49g. In the fig. 10, the cracks of central arch after the Phase1 and the tensile stresses in the FEM are shown.

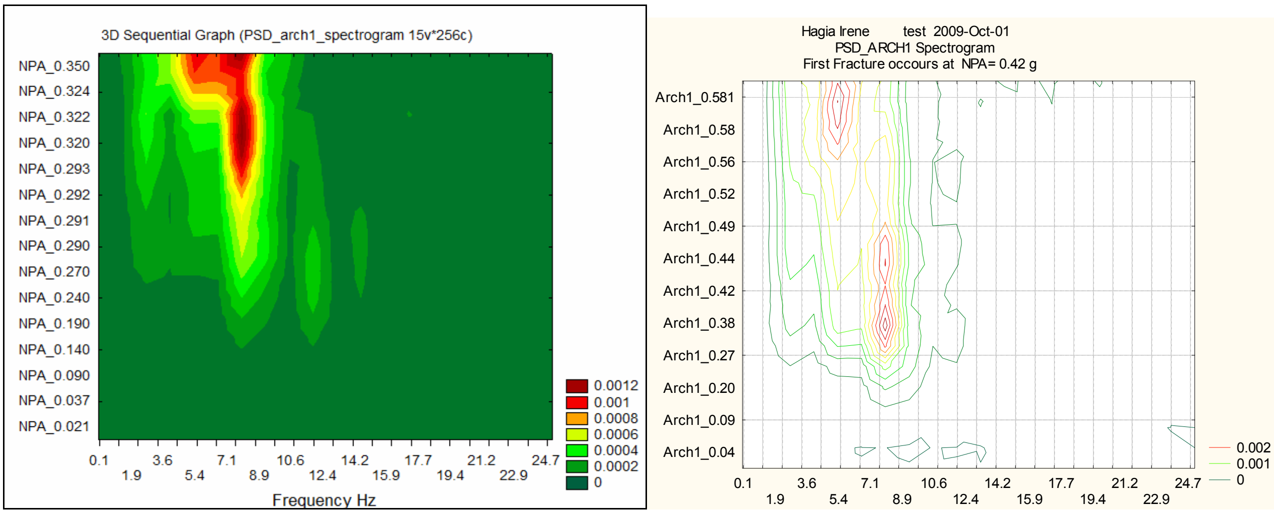

Figure 9: PSD spectrogram during the test sequence of Phase 1

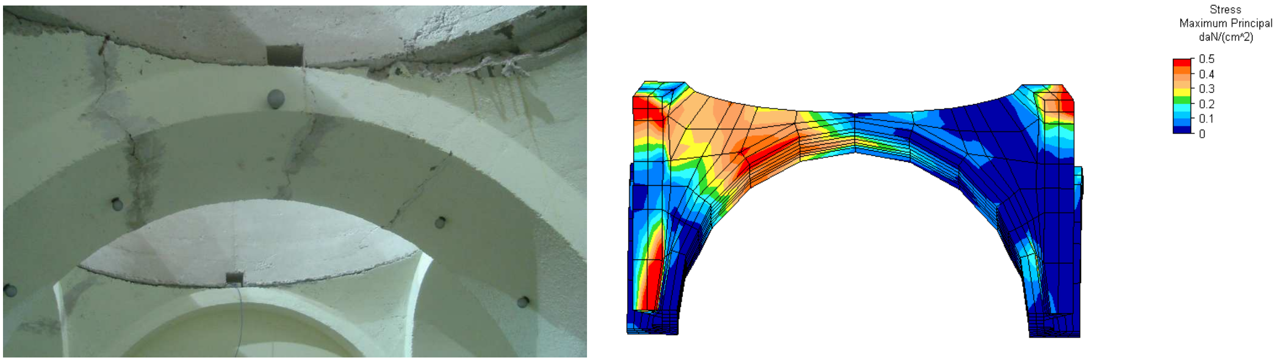

Figure 10: Crack in the central arch (Phase1, scale model not reinforced) and maximum tensile stress in the FEM model (PGA 0.36g)

Conclusions

The present paper concerned the study of the seismic behaviour of a large masonry model of Hagia Irene. The experimentation carried out at the ENEA-Casaccia Research Centre allowed to asses the seismic capacity of the reinforced model in comparison to the not reinforced one. The large scale model confirmed the weakness of central arch and lateral vaults for seismic actions in transversal direction as showed by analytical model of the actual structure. The results showed for reinforced model increasing of seismic capacity of 50-60% about (Table 3). The effectiveness of the studied consolidation works is therefore verified.

Table 3: Seismic capacity of not reinforced and reinforced model

model	*not reinforced*	*reinforced*
First damages	0.29 g	0.42g
Relevant damages	0.32g	0.49g
Maximum acceleration reached	0.36g	0.58g
Collapse acceleration estimated	0.40g4	0.65g