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Structural Modeling & Seismic Protection of Masonry Structural Systems

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ABSTRACT

The Masonry structures are complicated structures and there is, currently, a lack of knowledge and information concerning the behaviour of their structural system under seismic loading. Successful modeling of a masonry structure is a prerequisite for a reliable earthquake resistant design. However, modeling a real structure to a robust quantitative (mathematical) representation is a very difficult and complicated task. This paper is presenting a contribution toward a solution of the problem. A new methodology for earthquake resistant design of masonry structural systems, either before or after their repair and/or strengthening is presented. The application of the proposed method is illustrated in the case study of a historical and monumental masonry structure in Crete, Greece, namely the residence of Eleftherios Venizelos in Chania, who had been prime minister of Greece and a leading national personality in the first half of the 20th century.

Keywords: *Failure modes, Historical Structures, Masonry, Restoration, Seismic Protection, Structural Modeling.*

1 INTRODUCTION

The main structural systems of historical structures or monuments comprise, most of the time, masonry elements, composed of stone, bricks and mortar. For all types of old historical masonry structures (including monuments) erected in seismic zones of high seismicity, earthquakes are their number one “enemy” due to their very bad response to earthquakes.

The responsibility of protecting a historical structure falls mainly on the shoulders of the engineer. A successful intervention on a monument requires a good comprehension of its structural behaviour under static and dynamic (earthquake) loading. For an engineer, taking part to the restoration process of a historical structure, through the analysis of its structural system, means mainly to face the demanding task of equipping the historical structure with the capability to withstand future actions with the minimum possible amount of damage, while bearing in mind the characteristics and values which make this structure unique and worthy of special attention. This has to be carried out within the conditions imposed by current regulations and scientific Charters (e.g. the Athens Charter 1931 [1], the Venice Charter 1964 [2], etc.), which make the process of analysis more complicated.

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The analysis of ancient monuments poses important challenges because of the complexity of their geometry, the variability of the properties of traditional materials, the different building techniques, the lack of knowledge on the existing damage from the actions, which affect the monuments throughout their lifetime and the lack of codes. Nevertheless, rational methods of structural analysis, based on modern engineering principles have been developed in the last two decades (Lourenco 1996; 2002; 2006; 2007) [3][4][5][6] (Syrmakezis 1990; 1994; 1995; 2006) [7][8][9][10] (Binda 1997; 2000; 2005) [11][12][13] (Asteris 2000; 2005; 2008; 2011) [14][15][16][17] (Devaux 2005) [18] (Belmouden 2006) [19] (Moon 2006) [20] (Chronopoulos et al. 2012) [21][22].

The estimation of the seismic vulnerability of a historical monument is a multi-phased (and multifaceted) process that ranges from the description of earthquake sources, to the characterization of structural response, and to the description of measures for seismic protection.

The basic tool for a reliable vulnerability analysis is the quantitative estimation of the damage level of the monument's structural system. To estimate and describe the damage of this system (usually masonry elements) an analytical cubic polynomial method (failure criterion) has been proposed by the author. In addition, for the implementation of the proposed failure criterion, a specific computer program has been developed. According to this program, which uses as Input Data the Finite Element Analysis results as well as the mechanical characteristics of masonry material, coloured graphic images of the failure for each individual element within the structure are produced.

Proper probabilistic analysis of the above results leads to the development of fragility curves. Based on these curves the probability of a building to be damaged beyond a specified damage stage for various level of ground shocking can be determined. This information is quite important during the analysis and redesign procedure since it gives the opportunity to investigate several different repair/strengthening scenarios.

2 STRUCTURAL MODELING AND SEISMIC PROTECTION METHODOLOGY

Modeling a historical masonry structure is a difficult task, since masonry does not easily conform to the hypotheses usually assumed for other materials (isotropy, elastic behaviour, homogeneity). Furthermore, appropriate constitutive laws for the materials are still not well developed. In addition, continual modifications, which have taken place during the building's history, introduce several uncertainties in the model definition (i.e. geometry, materials, connection).

Based on the FEM, a basic methodology for the earthquake resistant design and rehabilitation of damaged masonry historical structures has been developed and presented here as a contribution to the solution of this complex problem. For the solution to a problem of this nature, one should go through the following nine distinct steps, briefly described as:

- Step 1: Preparation of detailed architectural and structural drawings, describing the current condition of the structure.
- Step 2: Determination of material characteristics. Mechanical characteristics of the materials composing the structure are the basic input data needed for the analysis. In particular, the compressive and tensile strength of the materials, their modulus of elasticity, and Poisson's ratio, are of primary importance.
- Step 3: Structural simulation. A three-dimensional (3D) FEM seems to be generally the most suitable one for the analysis. For higher model reliability, specific simulation parameters, like the rotation capacity of the wooden floor connection with the masonry wall, the rigidity degree of connections between intersected walls, the influence of spandrel beams, etc., have always to be taken into account.

- Step 4: Actions. Loadings foreseen by the codes for the relevant use of the structure have to be considered. An appropriate seismic loading has also to be taken into account, especially for structures built in seismic areas.
- Step 5: Analysis. Using the data of Steps 1–4, FEM elastic (or elastoplastic) analysis is performed and stresses (normal and shear), as well as displacements at the nodes of the mesh elements, are calculated.
- Step 6: Determination of Seismic Vulnerability. Taking into consideration conclusions made in Step 2, concerning material characteristics, a failure criterion is established and used for the definition of the failed regions of the structure. These failure results are used as input data for the development of fragility curves. Based on these curves the possibility of a structure to be damaged beyond a specified level (heavy, moderate, insignificant damage) for various levels of ground shocking is determined. This information is quite important during the analysis and redesign procedure for a historical structure since it gives the opportunity to investigate several different scenarios with different repair/strengthening decisions.
- Step 7: Making Repairing and/or Strengthening Decisions. Decisions have to be taken concerning repair and/or strengthening of the existing structure. The methods to be used, the extent of interventions, the type of the materials, etc., are mainly related to the results of Step 6. It has to be noted, however, that structural analysis is not always sufficient to give reliable judgements since, sometimes, there are too many uncertainties on material characteristics, inner cracks and discontinuities, permanent deformations and accumulation of stresses in plastic zones, which may impair the results of calculations (Croci 2001) [23]. For this reason, qualitative (and subjective) criteria based on the observation of the structure and the historical knowledge of the technologies, phenomena, events, etc. must also be considered before taking any repairing and/or strengthening decisions. This process is iterated (steps 3 to 7) for each repairing scenario considered.

All the above steps will be thoroughly presented in the next section along with a successful application of the proposed method in a historical and monumental masonry structure.

3 COMPUTATIONAL & MATHEMATICAL ASPECTS

In the present section, the quantitative and computational models underlying the proposed methodology are presented. In particular, a failure criterion for masonry materials, fragility curves, a damage index for masonry structures and the Structural Performance Levels for such structures are presented.

3.1 Failure Criterion

The basic step of the proposed methodology is the quantitative damage evaluation of masonry, which is the basic material of historical and monumental structures. The damage is estimated by a cubic polynomial function that is used for composite materials. In this method, the failure surface in the stress space can be described by the equation (Asteris 2000, 2010)[14][23], (Syrmakezis & Asteris 2001)[24]:

$$f(\sigma_\ell) = F_i\sigma_i + F_{ij}\sigma_i\sigma_j + F_{ijk}\sigma_i\sigma_j\sigma_k + \cdots - 1 = 0 \qquad (1)$$

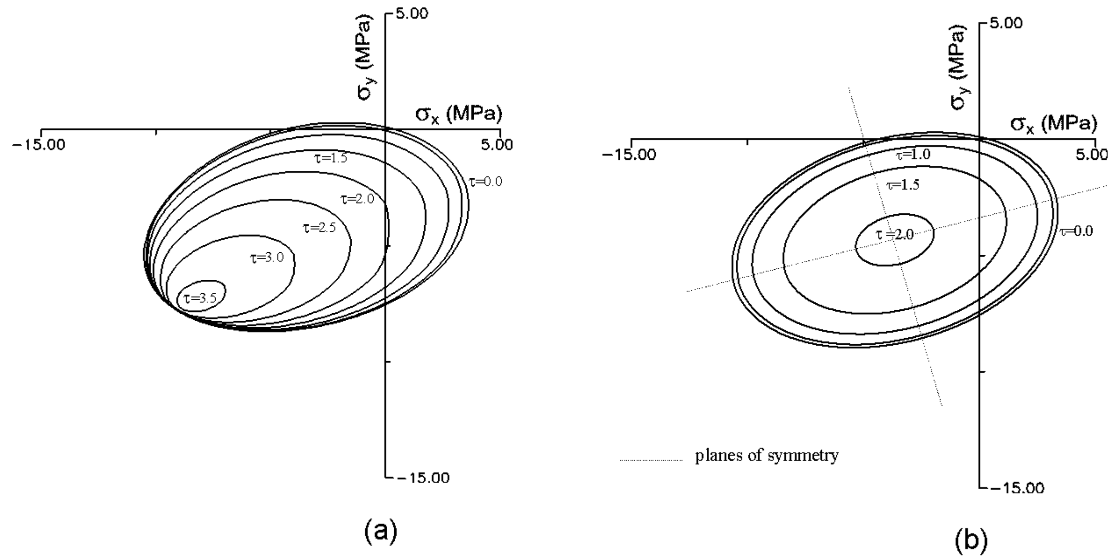


Figure 1 - Failure surface of masonry in normal stress terms: (a) General failure criterion; (b) Simplified failure criterion

In this equation σ_ℓ (ℓ = 1, 2,..., 6) are the components of stresses and Fi, Fij, Fijk (i, j, k = 1, 2,..., 6) are coefficients to be properly determined. If one restricts the analysis to a plane stress state, keeping terms up to third order, then Equation (1) reduces to:

$$\left.\begin{aligned} f(\sigma_x,\sigma_y,\tau) = F_1\sigma_x + F_2\sigma_y + F_{11}\sigma_x^2 + F_{22}\sigma_y^2 + F_{66}\tau^2 + 2F_{12}\sigma_x\sigma_y + \\ 3F_{112}\sigma_x^2\sigma_y + 3F_{122}\sigma_x\sigma_y^2 + 3F_{166}\sigma_x\tau^2 + 3F_{266}\sigma_y\tau^2 - 1 = 0 \end{aligned}\right| \tag{2}$$

Eliminating all third order terms in Eq. 2, a simplified yield criterion can be derived:

$$f(\sigma_x,\sigma_y,\tau) = F_1\sigma_x + F_2\sigma_y + F_{11}\sigma_x^2 + F_{22}\sigma_y^2 + F_{66}\tau^2 + 2F_{12}\sigma_x\sigma_y - 1 = 0 \tag{3}$$

This simple form of the yield criterion has already been used by other investigators (Andreaus 1996) [26] (Dhanasekar 1985) [27] to define the failure of brick masonry under biaxial stress. According to Syrmakezis and Asteris (2001)[25], the general yield criterion (Figure 3a) fit the non-symmetrically dispersed experimental data better than the simplified model (Figure 3b).

For the implementation of the proposed failure criterion, a special-purpose computer program, named FAILURE, has been developed (Syrmakezis et al. 1995) [9]. The program uses as Input Data the Finite Element Analysis results (stresses), and the mechanical characteristics of masonry material (strengths), and produces coloured graphic images of the failure for each individual element within the structure.

3.2 Fragility Curves

The prediction of structural damage is critical for the assessment of economic losses in seismically active regions, and should be estimated with an acceptable degree of credibility in order to mitigate the potential losses that are dependent on the seismic performance of structures in that region. One of the most important tools to describe the structural damage distribution in a given region consist of the *fragility curves.*

Evaluating seismic fragility information curves for structural systems involves a) information on structural capacity, and b) information on the seismic hazard. Due to the fact that both the aforementioned contributing factors are uncertain to a large extent, the fragility evaluation cannot be carried in a deterministic manner. A probabilistic approach, instead, needs to be utilized in the cases in which the structural response is evaluated and compared against "limit states," that is, limiting values of response quantities correlated to structural damage.

Fragility curves can be obtained from a set of data representing the probability that a specific response variable R (e.g. displacement, drift, acceleration, damage) exceeds predefined limit states $r_{\lim}$ for various earthquake hazards on a specific structure or on a family of structures.

Numerical calculation of fragility requires information on the expected response and its variability. This involves the creation of a detailed model of the structure and the application of numerical techniques for probabilistic evaluation of the structural response.

Simplified methodologies for fragility evaluation have been proposed by Kircher et al. (1997) [28] and incorporated in HAZUS99 (FEMA, 1999) [29]. These methodologies assume that the spectral ordinates are log-normally distributed, assuming the variability is represented by the logarithmic standard deviation.

Fragility is evaluated as the total probability of a response R exceeding the allowable response value $r_{\lim}$ (limit-state), for various earthquake intensities I. In mathematical form, this simply is a conditional probability (Barron-Corvera, 2000, Reinhorn et al, 2001) [30][31] given by:

$$Fragility = P\left[R \geq r_{\lim} | I\right] = \sum_{j}^{3} P\left[R \geq r_{\lim} | I, C\right] P\left(C = c_j\right) \tag{4}$$

where $P\left(C = c_j\right)$ is the probability that capacity c_j occurs. In the following example basic steps for the development of the fragility curves, are thoroughly presented.

3.3 Damage Index

Damage control in a building is a complex task. There are several response parameters that can be instrumental in determining the level of damage that a particular structure suffers during a ground motion; the most important ones are: deformation, relative velocity, absolute acceleration, plastic energy dissipation and viscous (or hysteretic) damping energy dissipation. Controlling the level of damage in a structure consists primarily in controlling its maximum response.

Damage indices establish analytical relationships between the maximum and/or cumulative response of structural components and the level of damage they exhibit (Park 1987) [32]. A performance-based numerical methodology is possible if, through the use of damage indices, limits can be established to the maximum and cumulative response of the structure, as a function of the desired behavior(s) of the building for the different levels of design ground motion. Once the response limits have been established, it is then possible to estimate the mechanical characteristics that need to be supplied to the building so that its response is likely to remain within these limits.

For the case of masonry structures a new damage index [DI} which employs as response parameter the percentage of the failed area of the structure to the total area of the structure has been proposed (Syrmakezis et al. 2006)[33]:

$$DI = \frac{A_{fail}}{A_{tot}} \times 100 \tag{5}$$

where A_{fail} is the failed surface area of the structure and A_{tot} the total surface area of the structure.

3.4 Structural Performance Levels

As practiced today, performance-based seismic design is initiated with a discussion between the client and engineer about appropriate performance objectives. The engineer then prepares a design capable of meeting these objectives. Performance objectives are expressed as an acceptable level of damage, typically categorized as one of several performance levels, such as immediate occupancy, life safe or collapse prevention, given that ground shaking of specified severity is experienced.

In the past, the practice of meeting performance-based objectives was rather informal, non-standard, and quite qualitative. Some engineers would characterize performance as life-safe or not; others would assign ratings ranging from poor to good. This qualitative approach to performance prediction was appropriate given the limited capability of seismic-resistant design technology to deliver building designs capable of quantifiable performance.

We consider three structural performance levels: a) heavy damage, b) moderate damage and c) insignificant damage, in a similar way to the Federal Emergency Management Agency (FEMA 273) [34]. The performance levels are defined by the values of DI (as shown in Table 1). Especially a value of [DI] less than 10% can be interpreted as insignificant damage; from 10% to less than 20%, as moderate damage; and larger or equal than 20% as heavy damage.

Table 1 - Proposed Structural Performance Levels for un-reinforced masonry.

Overall Damage	**Heavy Damage**	**Moderate Damage**	**Insignificant Damage**
	Extensive cracking: face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Minor cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.
[DI]	≥ 20%	10% ≤ ~ < 20%	< 10%
	Collapse prevention	Life safety	Immediate occupancy

4 APPLICATION

The proposed methodology has been applied in a historical and monumental masonry structure in Crete, Greece. In this section, the entire procedure is presented through its seven distinct following steps:

Step 1- Identity of the Structure: This structure is typically a neoclassical building from the late 19th century. This is a representative sample of the architectural heritage of the city of Chania, as it was developed outside the walls of the Old City at the end of Turkish occupation. Its general form is characterized by symmetry and regularity, and has a uniform and compact size. It includes semi-basement, ground floor and first floor. As it concerns the masonry walls, there are made of local soft limestone and low quality mortar. Noteworthy is the presence of iron tiers which are at the levels of the top floor and roof deck at the four corners of the building. The tiers, until today, and despite the erosion and the subsequent decay, restrained the masonry well. The floors and roof are wooden. The roof is hipped, based on the perimeter bearing walls of the building. Moreover, two additional buildings are attached to the main one laterally as shown in figure 2.

Figure 2 – South facade of the building

The main building has dimensions of 15.2 x 15.85m^2, the west building 2.65 x 5.05m^2 and the east 5.65x 7.00m^2. The height of the main building is 10.87m and the maximum height including the roof is equal to 13.30m. The ground floor shows almost complete internal with a wide main aisle that divides the building into two wings of equal width. At the centre of the left is a double staircase. As it regards the morphological description it is observed symmetry in the facades, which are characterized by elaborate ornamentation. Most architectural and decorative elements are concentrated on the front facade whereas the rest are distinguished by simplicity.

Generally the condition of the building is good although the structural materials of the bearing walls are not satisfactory. Several problems are listed locally, but not very seriously damages by seismic or geological reasons. A summary of the injuries and damage that have been found in the structure are: cracks, wall disconnections, deterioration of mortar or stone, masonry disruption, traces of moisture, wear and damage of linear elements on doors and windows, Deterioration of wooden roof elements and wooden flooring, corrosion of embedded iron tiers at the corners of the walls.

Step 2-Material Characteristics: The materials composing the structure are: natural stone, elements of reinforced concrete, wooden roof, timber planking and metal elements. In the present construction, masonry is the dominant material and its mechanical characteristics are essentially shaping the response of the building. The properties of the masonry are determined by the materials that compose it (natural stone and mortar). Especially, for masonry material the mechanical characteristics are: f_{wk}= 3050 MPa, γ= 22kN/m³, E= 3050000 kN/m², ν= 0.3.

Step 3-5 - Structural Modeling: The program used to simulate the structure is the software SAP 2000 14 Nonlinear. With this software was formed an appropriate FEM model to calculate the response of the structure. The development of the finite elements networks was such that the ideal concentration of masses at the nodes to better simulates the real mass distribution. This ensures a faithful simulation of the inertial loads of construction for dynamic analysis. To fully determine the deformation of the system, six degrees of freedom for each node were considered Oxyz. The six degrees of freedom correspond to three transfers, along the axes x, y, z and three rotations of vectors, parallel to the same axes. The model of the building is shown schematically in figure 3.

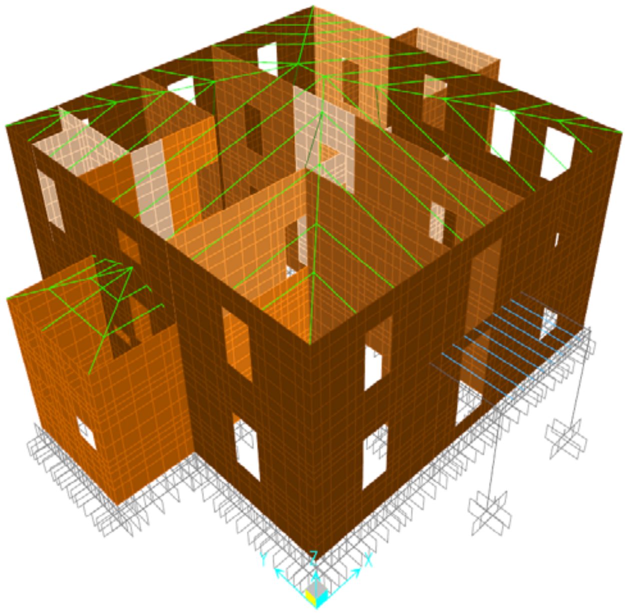


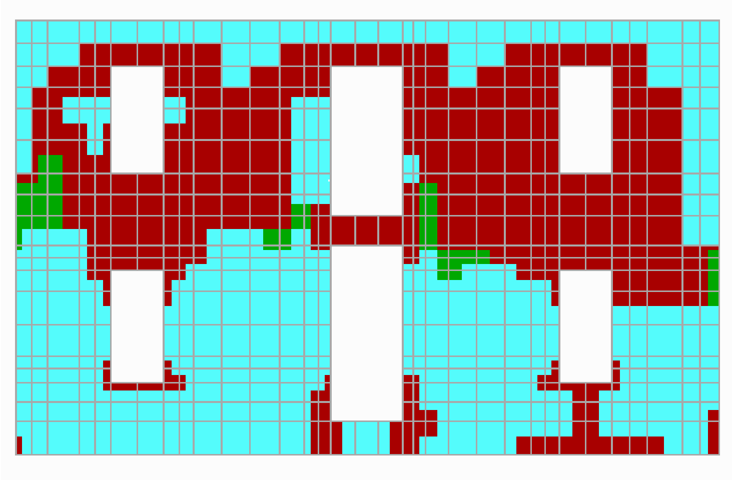
Figure 3 - Colours on the model based on material and thickness of the profiles (southwest corner)

The geometrical simulation was done by isotropic surface members (shell elements) and isotropic linear members (frame elements), which are considered to represent with sufficient reliability the properties of the real body. The model used to analyze the building is spatial. The discretization of the finite element network was through flat quadrilateral and triangular elements. Depending on the geometry and loading conditions prevailing at each region of the model the condensation of the data was chosen. In this way it was better simulated the anisotropic behaviour of the masonry structure. Specifically, condensation occurred on the following areas: Locations of concentrated loads, perimeter of the openings, corner areas (wall compounds). For the simulation model were used 5197 surface members, 5745 nodes and 120 beam elements.

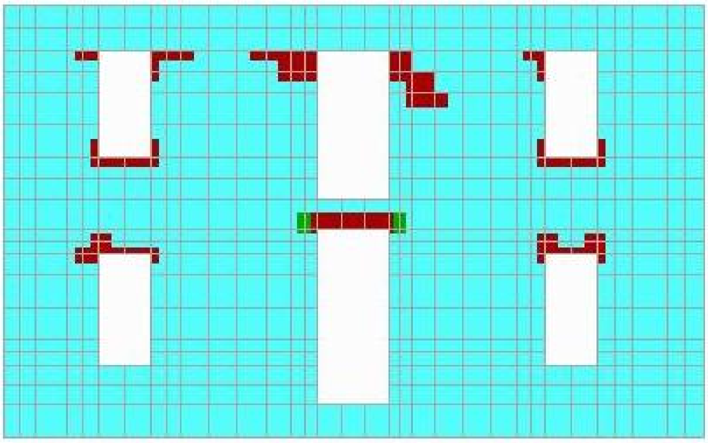
For the simulation of the model were taken into account both, static and dynamic loads. For static loads apart from the weight of materials permanent and moving loads were added. More specifically, dead loads were considered at floors and roofs, whereas live loads only at the floors since at the roofs are negligible. More specifically, live loads were taken equals to 2,5kN/m^2 while dead loads as follows: unreinforced concrete floor: 3.00kN/m^2, wooden floor: 0.6kN/m^2, roof tiles: 0.5kN/m. For dynamic loading, seismic design actions were taken into account. Due to the lack of data of recent seismic events the seismic loads were considered according to Greek Seismic Code (EAK 2003).

Step 6 – Determination of Seismic Vulnerability

Failure analysis of the structure: The failure analysis of the structure so for the existing structure as well as for the studied interventions' scenarios, was based on the failure criteria explained at a previous paragraph. The analysis concerns a range of Peak Ground Accelerations between 0.08g to 0.40g and masonry tensile strength ranging from 0.05MPa to 0.55MPa. Failure results refer to percentage of the overall failure, as well as to picture, as such of the Figures 4, distinctly the type, extent and position of damage.



a) Front facade of the structure before interventions



b) Front facade of the structure after interventions

FAILURE UNDER BIAXIAL TENSION

FAILURE UNDER BIAXIAL COMPRESSION

FAILURE UNDER HETEROSEMOUS STRESSES

NON FAILURE

Figure 4 - Typical failure areas for the front facade of the structure before and after interventions (PGA=0.40 g)

Probabilistic analysis - Fragility curves: The results concerning the failure areas of the structure were analysed with probabilistic methods. Especially the Probability Distribution Function and the associated Probability Density Function were estimated for each level of Peak Ground Acceleration applied at the structure. Using these Probability Distribution Functions, the probabilities of damage of the structure for the three structural performance levels (insignificant, moderate and heavy damage) have been determined. Figures 5 and 6 show the fragility curves of the structure before and after interventions.

These figures show that the fragility curves are of primary importance in evaluating and ranking the efficacy of the remedial proposals, to address the seismic protection of masonry structural systems. It should be indicatively mentioned that the probability of heavy damage from a seismic motion with demand represented by PGA=0.24g is reduced by 35% (that is, from 81% probability of damage to 52% probability of damage, as can be seen in Figures 5 and 6) for the case of one from the studied repairing scenarios.

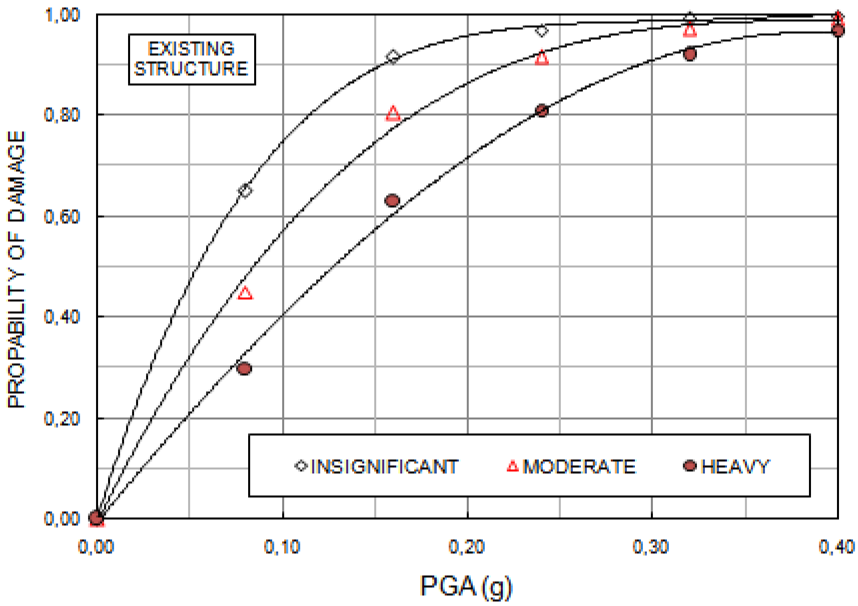


Figure 5 - Fragility curves for the existing structure (before intervention)

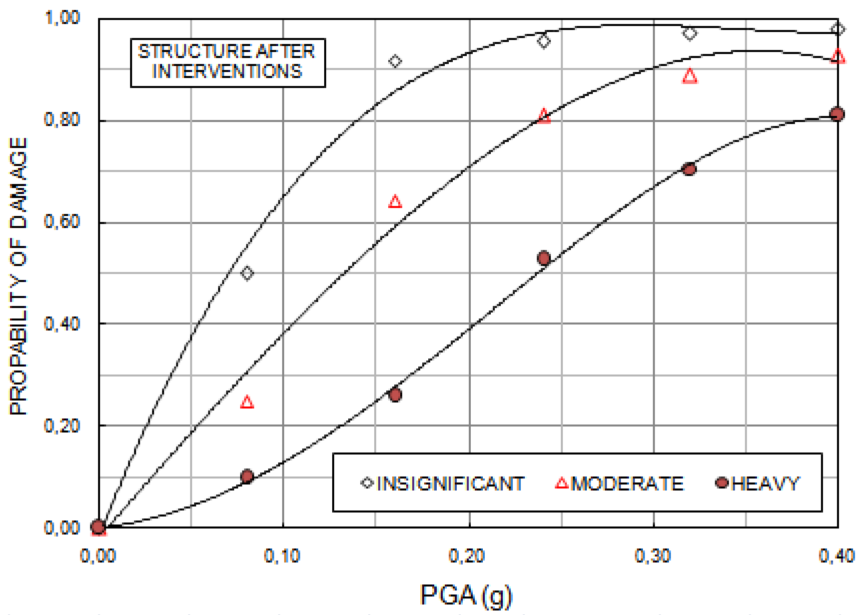


Figure 6 - Fragility curves for the repaired structure (after intervention)

5 CONCLUSION

A general-purpose methodology for the quantitative estimation of the seismic vulnerability of historical structures has been proposed. The main purpose this study was to examine the effectiveness of fragility analysis in order to assess the seismic performance of masonry structural systems. It was clearly shown that the fragility curves, which comprise a basic component of the proposed methodology, are of particular importance in assessing the efficacy of the remedial proposals, and, thus, are instrumental in the decision to select the optimum one.

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