

Seismic vulnerability assessment of historical masonry structural systems



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ABSTRACT

Masonry structures are complex systems that require a thorough and detailed knowledge and information regarding their behavior under seismic loading. Appropriate modeling of a masonry structure is a prerequisite for a reliable earthquake resistant design or assessment. However, modeling a real structure to a robust quantitative (mathematical) representation is a very difficult, complex and computationally demanding task. This paper presents a methodology for earthquake resistant design or assessment of masonry structural systems. The entire process is illustrated using case studies from historical masonry structures in the European area. In particular, the applicability of the proposed method is checked via analyses of existing masonry buildings in three countries, namely Greece, Portugal and Cyprus, with different seismicity levels, influencing the risk impacting the masonry structures. Useful conclusions are drawn regarding the effectiveness of the intervention techniques used for the reduction of the vulnerability of the case-study structures, through the comparison of the results obtained.

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1. Introduction

The majority of the main structural systems for historical structures are masonry elements, composed of stone, bricks, adobe and mortar. For many old historical masonry structures (including monuments) erected in zones of moderate to high seismicity, earthquake is one of their principal threats due to their limited earthquake resistance capacity [1], let alone other problems associated to the misuse or lack of proper maintenance. A successful intervention on a monument requires a good comprehension of its structural behavior under static and dynamic (earthquake) loading. An Engineer, taking part in the restoration process of a historical structure, through the analysis of its structural system, has to face the demanding task of checking and providing the structure with adequate capacity to withstand future actions with certain limits of damage, while bearing in mind the characteristics and values which make the structure unique and worthy of special attention. This has to be carried out within the conditions imposed by past or current regulations and scientific Charters (e.g. the Athens Charter 1931 [2], the Venice Charter 1964 [3], etc.), which make the whole process of analysis more demanding.

Masonry constructions are typically complex structures and there is lack of knowledge and information concerning the behavior of their structural systems, particularly in what regards their seismic response. Typically, these structures are more massive than today's structures and usually carry their actions primarily in compression.

Successful modeling of a masonry historical structure is a prerequisite for a reliable earthquake resistant design or assessment. For modern structures, with new industrial materials (reinforced concrete, steel, etc.), the development of a reliable mathematical model is possible, due to the fact that materials and member characteristics are more uniform and mostly explicitly known. On the other hand, for the case of masonry, and especially for the traditional plain one, it seems that there is a lot to be done in this field, until Engineers become more confident about the accuracy of the modeling.

For the purpose of masonry analysis and design, an operationally simple strength criterion is essential, taking into account the many uncertainties of the problem. Systematic experimental and analytical investigations on the response of masonry and its failure modes have been conducted in the last decades. Numerous analytical criteria have been proposed for masonry structures [4–6]. The main disadvantage of many existing criteria is that they ignore the distinct anisotropic nature of masonry, not to mention problems

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arising from differences within its thickness, and models not ignoring that behavior consist of more than one type of failure surfaces leading to an additional effort in the analysis process of the masonry structures [7]. According to Zienkiewicz et al. [8], the computation of singular points on failure surfaces may be avoided by a suitable choice of a continuous surface, which usually can represent, with a good degree of accuracy, the real condition.

Since reliable experimental data in the combined-stress state are rising rapidly [9–11], it is, therefore, the right time to examine the validity and utility of existing criteria, and to propose a failure surface of convex shape suitable for the anisotropic nature of masonry material. According to Hill [12] and Prager [13], the failure surface for a stable material must be convex. This, in mathematical terms, is valid if the total Gaussian curvature K of the failure surface is positive.

As can be concluded, various researchers have been working on the earthquake resistant design of masonry structural systems and especially on determining a strength criterion, but there is still a lot ongoing research on this field. In addition, aspects regarding the in- and out-of-plane behavior of 2- or 3-leaf masonry are not yet covered in detail. In the present study, masonry is considered as a single leaf one and is modeled as a homogeneous elastic material.

In this paper the framework of thought for such interventions is first discussed and then the steps of the proposed methodology are outlined. Following these, mathematical modeling issues, including failure criteria, are presented. Possible intervention techniques are described and then the results of the application of the proposed methodology in three case-studies are presented, followed by a comparison of the results and conclusions.

2. General methodology

Structures of architectural heritage present a number of challenges in conservation, diagnosis, analysis, monitoring, repair and strengthening that limit the application of modern codes and building standards. Recommendations are desirable and necessary to both ensure rational methods of analysis and intervention methods appropriate to the cultural context [14].

2.1. Framework of thought

Our research has adopted the rationale resulted from the work developed within the ICOMOS 2001 [15] scientific committee ISCARSAH (International Scientific Committee of the Analysis and Restoration of Structures of Architectural Heritage) and, in particular, by the *ICOMOS Charter: Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage* (ISCARSAH Principles). This framework of thought is delineated by the principles of: research and documentation, authenticity and integrity, compatibility (both visual and physical and/or chemical), minimal intervention and the degree of reversibility, as it is very seldom possible to achieve a fully reversible technique. They are in harmony with those that are the foundation of the *Athens and Venice Charters* and *The Secretary of the Interior's Standards for Historic Preservation Projects* [16].

2.2. ICOMOS recommendations

Differing opinions has been a characteristic of the field throughout its long history in its attempts to establish criteria for rehabilitation of historic and monumental structures. Nevertheless, a widely accepted framework is the Venice Charter [3], which was formulated in May of 1964, as a result of deliberations of many specialists and technicians in the restoration of historic monumental structures. During that congress, among many issues discussed

for the preservation of historic structures, the Charter focused on achieving harmony between the existing structure and the new rehabilitation work performed upon it. According to the Charter, such interventions must follow the following basic principles: material compatibility, conservation of overall lay-out or decoration and mass–color relationship, avoidance of the removal of any part, or additions to the building. The Charter requires detailed documentation of all rehabilitation works by means of critical reports (including drawings and photographs) and recommends its publication. According to ICOMOS recommendations, a thorough understanding of the structural behavior and material characteristics is essential for any project related to the architectural heritage. It is recommended that the work of analysis and evaluation should be done with the cooperation of specialists from different disciplines, such as earthquake specialists, architects, engineers and art historians. In addition, it is considered necessary for these specialists to have common knowledge on the subject of conserving and upgrading or strengthening the historical buildings.

The methodology puts emphasis on the importance of an “Explanatory Report”, in which all the acquired information, the diagnosis, including the safety evaluation, and any decision to intervene should be fully detailed and justified. This is essential for future analysis of continuous processes affecting the structure (such as decay processes or slow soil settlements or other side-effects), or phenomena of cyclical nature (such as the variation in temperature or moisture content) and even phenomena that can suddenly occur (such as earthquakes or hurricanes), as well as for future evaluation and understanding of the remedial measures adopted at present.

2.3. Proposed methodology

Based on ICOMOS principles and recommendations, as well as on other similar works [17–27,1,28,29], a restoration methodology for historical masonry structures has been developed and presented here as a contribution to the solution of this complex problem. A flowchart of the proposed methodology is illustrated in Fig. 1. In the framework of the proposed methodology, the following eight distinct steps are included:

2.3.1. Step 1: Historical and experimental documentation

There are some aspects that should be followed before carrying out a rigorous structural analysis, which are listed below [29].

- (a) Experience shows that the structural analysis regarding the seismic response of a Monument is an integral part of the broader study of the Monument; history and architecture of the Monument are indispensable prerequisites for the structural analysis, in order to account for all initial and consecutive construction phases, previous interventions or additions, etc.
- (b) Description of existing damages and/or previous interventions (visible or possibly hidden ones), together with their in-time evolution; monitoring may be helpful.
- (c) Systematic description of the materials, including their interconnections. Connections of perpendicular walls or of walls and floors should be thoroughly investigated.
- (d) Results of experimental investigations regarding: geometrical data, *in situ* evaluation of the strength of materials, structural properties of masonry walls, dynamic response of the construction, subterranean data, as well as results of possible previous monitoring (displacements, settlements, internal forces, humidity, groundwater level, cracks' opening, seismic accelerations, environmental data, etc.).
- (e) Description of the structural system, in a systematic and detailed way.

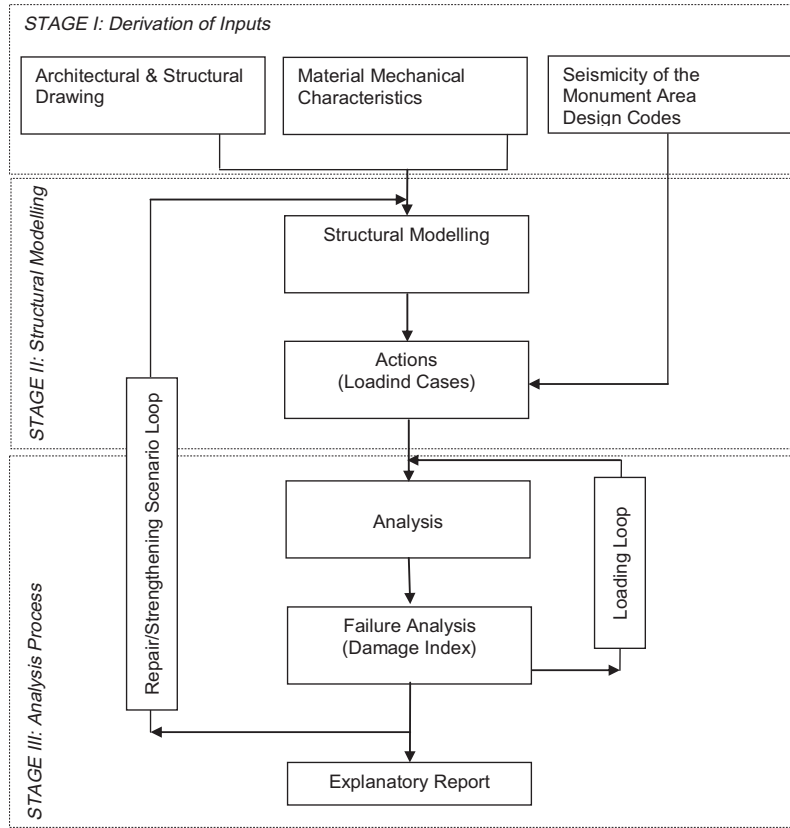


Fig. 1. Flowchart with the applied methodology for vulnerability and restoration assessment.

- (f) Description of the subsoil and the foundation, including basic characteristics.

2.3.2. Step 2: Material characteristics

The characteristics of materials composing the structure are basic input data for structural analysis. Namely, the compressive/tensile strength of the materials, the modulus of elasticity and Poisson ratio are of primary importance, at least as far as a linear/elastic analysis is concerned. For the estimation of those parameters, combination of analytical or semi-empirical methods and experimental data have to be used. For the determination of the masonry compressive and tensile strength, several semi-empirical expressions are available in the literature. In the majority of these expressions, global effects contributing to the system resistance, such as buckling-effects or local-compression resistance are not considered.

The formulae for the estimation of the compressive strength, f_w , of low-strength stone-masonry, with a single leaf, proposed by Tassios and Chronopoulos [32] is presented next, and combines several parameters affecting the strength:

$$f_{wc} = \xi \left[\left(\frac{2}{3} \sqrt{f_{bc}} - \alpha \right) + \beta f_{mc} \right] \text{ [in MPa]} \quad (1)$$

$$f_{wt} = \frac{2}{3} f_{mt} \quad (2)$$

where f_{wc} , f_{wt} are the compressive and tensile strength of masonry respectively, f_{mc} , f_{mt} are the compressive and tensile strength of mortar respectively, f_{bc} is the compressive strength of the block/stone material, α is a reduction factor due to non-orthogonality of blocks ($\alpha = 0.5$ for block stones and $\alpha = 2.5$ for rubble stones), β is a mortar-to-stone factor ($\beta = 0.5$ for rough stones and $\beta = 0.1$ for very smooth-surface stones) and ξ is a factor expressing the adverse

effect of thick mortar joints, $\xi = 1/[1 + 3.5(k - k_0)]$, (k = volume of mortar/volume of masonry) and $k_0 = 0.3$.

However, for well-built and regular masonry structures, Tassios [33] proposed another expression for the estimation of the compressive strength, namely:

$$\text{– for } f_{bc} > f_{mc} \quad f_{wc} = [f_{mc} + 0.4(f_{bc} - f_{mc})] \times (1 - 0.8\sqrt[3]{\alpha}) \quad (3)$$

and

$$\text{– for } f_{bc} \leq f_{mc} \quad f_{wc} = f_{bc} \times (1 - 0.8\sqrt[3]{\alpha}) \quad (4)$$

where f_{bc} , f_{mc} are the compressive strength of blocks and mortar, respectively. $\alpha = t_{jm}/h_{bm}$ is the ratio between average bed (horizontal) joint thickness t_{jm} , and average block height h_{bm} .

2.3.3. Step 3: Structural model

The simplest approach to the modeling of complex historic buildings is given by the application of different structural elements, employing truss, beam, panel, plate or shell elements to represent columns, piers, arches and vaults, with the assumption of homogeneous material behavior.

A 3-D finite element model (with elastic materials), as used in this study, as well, seems to be generally the most suitable for the analysis, at least as far as a global assessment is concerned. For higher model reliability, specific simulation parameters, such as the rotation capacity of the wooden floor or roof connection with the masonry wall, the degree of connections between intersected walls and the influence of spandrel beams, have always to be taken into account.

An interesting and straight-forward alternative approach which requires our attention is the use of equivalent frames, as shown in Milani et al. [30]. In any case, the equivalent frame approach is most times hardly applicable (for instance it is not suited for churches, bridges, bell-towers, light houses, etc.). Another Finite Element model which is very efficient and much more advanced for the non-linear analysis of masonry structures is that presented in Milani and Venturini [31], where a full pushover analysis with damaging materials is performed on several masonry churches by means of rigid elements interconnected by damaging interfaces.

2.3.4. Step 4: Actions

Different loading cases have to be taken into consideration, including seismic actions for structures built in seismic areas. Combinations of dead loads, live loads and earthquake demands, have to be used. Earthquake has to be considered along all unfavorable directions for the building. Nevertheless, certain issues are still open, regarding e.g. the poor hysteretic behavior of masonry or the adverse influence of the simultaneous vertical component of the seismic action.

2.3.5. Step 5: Analysis

Using input data of the previous steps a Finite Element Analysis is performed and stresses (normal-shear)–displacements at the joints of the mesh are calculated. Due to the actual behavior of plain masonry and the high degree of uncertainty in the previous steps, elastic analysis is a first valuable tool for such structures, especially before any repair and/or strengthening.

2.3.6. Step 6: Failure criterion and assessment

A failure criterion must be established for the definition of the damaged regions of the structure (as a first insight). Taking into account the conclusions of Step 2 concerning materials' characteristics, such a criterion is proposed, and will be used as an input to carry out the analysis.

These failure results are used as input data for the development of a damage index. Based on this index the possibility of a structure to be damaged beyond a specified level (heavy, moderate, insignificant) for various levels of ground acceleration is determined. This information is important during the analysis and redesign process for a historical structure since it gives the opportunity to investigate different scenarios with different options regarding repair/strengthening.

2.3.7. Step 7: Repairing and/or strengthening decisions and reanalysis

According to the results of Steps 5 and 6, all the damaged regions are repaired and/or strengthened. The method to be used, the extent of the interventions, the type of the materials, etc., could be directly related to the results and are based on semi-empirical expressions for the final mechanical characteristics of masonry (see e.g. [32]).

Last, a new structural analysis has to be performed including all the final materials, loading and structural data. Results of the analysis have subsequently to be used in the process of Steps 5 and 6, leading to a final approval (or rejection) of the decisions already taken for repair or strengthening of the existing structure.

2.3.8. Step 8: Explanatory report

The last step, as a result of the proposed methodology, includes the detailed 'Explanatory Report', where all the collected information, the diagnosis, including the safety evaluation, and any decision to intervene should be fully detailed. This document is essential for eventual future analyses and interventions' measures in the structure.

3. Mathematical issues

3.1. Failure criterion

The proposed quantitative methodology is based on the damage evaluation of masonry. Damage is estimated using a cubic polynomial function. In this method, the failure surface in the stress space can be described by the following equation [34,35]:

$$f(\sigma_x, \sigma_y, \tau) = 2.27\sigma_x + 9.87\sigma_y + 0.573\sigma_x^2 + 1.32\sigma_y^2 + 6.25\tau^2 - 0.30\sigma_x\sigma_y + 0.009585\sigma_x^2\sigma_y + 0.003135\sigma_x\sigma_y^2 + 0.28398\sigma_x\tau^2 + 0.4689\sigma_y\tau^2 = 1 \quad (5)$$

However, this anisotropic failure criterion may apply only to certain types of masonry material. This disadvantage can be reversed if this criterion is expressed in a non-dimensional form, and, as so, it can be applied more generally to other types of masonry materials. This can be achieved by dividing and multiplying (at the same time) each term in Eq. (5) by one material uniaxial strength raised in the sum of the exponents of the variables σ_x , σ_y , τ (as appeared in each term). To this end, it is selected the uniaxial vertical to the bed joints compressive strength denoted with the symbol $f_{wc}^{90^\circ}$. This model was proposed by Asteris and Syrmakizis [36] and Asteris [37].

Thus, Eq. (5) takes the following form:

$$f(\sigma_x, \sigma_y, \tau) = 17.15 \left(\frac{\sigma_x}{f_{wc}^{90^\circ}} \right) + 74.57 \left(\frac{\sigma_y}{f_{wc}^{90^\circ}} \right) + 32.71 \left(\frac{\sigma_x}{f_{wc}^{90^\circ}} \right)^2 + 75.34 \left(\frac{\sigma_y}{f_{wc}^{90^\circ}} \right)^2 + 356.74 \left(\frac{\tau}{f_{wc}^{90^\circ}} \right)^2 - 17.12 \left(\frac{\sigma_x}{f_{wc}^{90^\circ}} \right) \left(\frac{\sigma_y}{f_{wc}^{90^\circ}} \right) + 4.13 \left(\frac{\sigma_x}{f_{wc}^{90^\circ}} \right)^2 \left(\frac{\sigma_y}{f_{wc}^{90^\circ}} \right) + 1.35 \left(\frac{\sigma_x}{f_{wc}^{90^\circ}} \right) \left(\frac{\sigma_y}{f_{wc}^{90^\circ}} \right)^2 + 122.46 \left(\frac{\sigma_x}{f_{wc}^{90^\circ}} \right) \left(\frac{\tau}{f_{wc}^{90^\circ}} \right)^2 + 202.20 \left(\frac{\sigma_y}{f_{wc}^{90^\circ}} \right) \left(\frac{\tau}{f_{wc}^{90^\circ}} \right)^2 = 1 \quad (6)$$

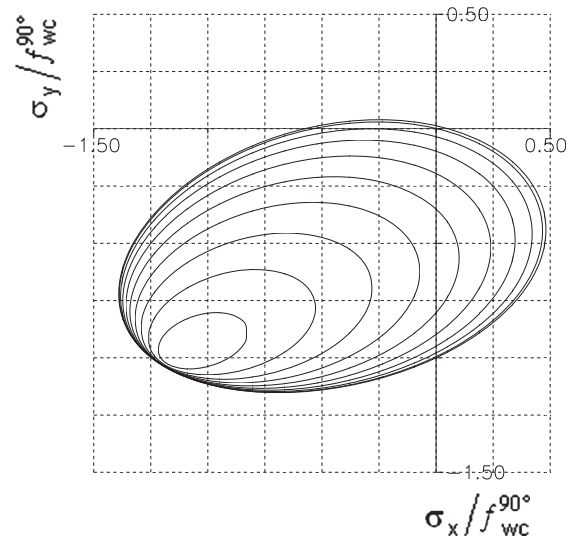


Fig. 2. Non-dimensional failure surface of masonry in normal stress terms [36] (= 0.00 up to 0.45 by step = 0.05).

Fig. 2 depicts the contour map of Eq. (6), that is the non-dimensional failure surface of masonry in normal stress terms (with $\tau/f_{wc}^{90^\circ}$ taking values of 0 up to 0.45 by steps of 0.05).

It should be noted that a plethora of masonry failure criteria based on homogenization techniques have been proposed in the literature [36–41].

3.2. Structural modeling

Masonry may exhibit distinct properties for different directions, mainly due to the influence of mortar joints (bed and head ones) acting as planes of weakness. Depending on the relative orientation of the joints regarding the loading stress direction, as well as on the dimensions and properties of the units, failure can occur only in the joints, or can affect both the joints and units. The large number of factors influencing the masonry behavior, such as dimension and anisotropy of the bricks, joint width and arrangement of bed and head joints, material properties of both stone and mortar, and quality of workmanship, make the simulation of plain masonry an extremely difficult task, especially in the case of rubble stone masonry.

As referred by Lourenço [42], Asteris et al. [43] and Lourenço [44] the brick masonry models commonly adopted could be grouped in the following three groups, according to the level of refinement:

3.2.1. Macro-modeling (masonry as one-phase material)

Units, mortar and unit–mortar interface are smeared out in a homogeneous continuum (Fig. 3b). No distinction between the individual units and joints is made, and masonry is considered as a homogeneous, isotropic or anisotropic continuum. While this procedure may be preferred for the analysis of large masonry structures, it is not suitable for the detailed stress analysis of a small panel, due to the fact that it is difficult to capture all the possible failure mechanisms. The influence of the mortar joints acting as planes of weakness cannot be addressed.

3.2.2. Simplified micro-modeling (masonry as a two-phases material)

Expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit–mortar interfaces is lumped in discontinuum elements (Fig. 3c). According to these procedures, which are intermediate approaches, the properties of the mortar and the unit/mortar interface (masonry as a two-phase material) are lumped into a common element, while expanded elements are used to represent the brick units. This approach leads to the reduction in computational intensiveness, and yields a model, which is applicable to a wider range of structural systems.

3.2.3. Detailed micro-modeling (masonry as a three-phases material)

Units and mortar in the joints are represented by continuum elements whereas the unit–mortar interface is represented by discontinuum elements (Fig. 3d). While this leads to accurate results, the level of refinement means that any analysis will be computationally intensive, and so will limit its application to small laboratory specimens and structural details. Sutcliffe et al. [45], Asteris et al. [46] and Lourenço et al. [44] have proposed simplified micro-modeling procedures to overcome the problem.

3.3. Damage index

Damage control in a building is a complex task, especially under seismic action. 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

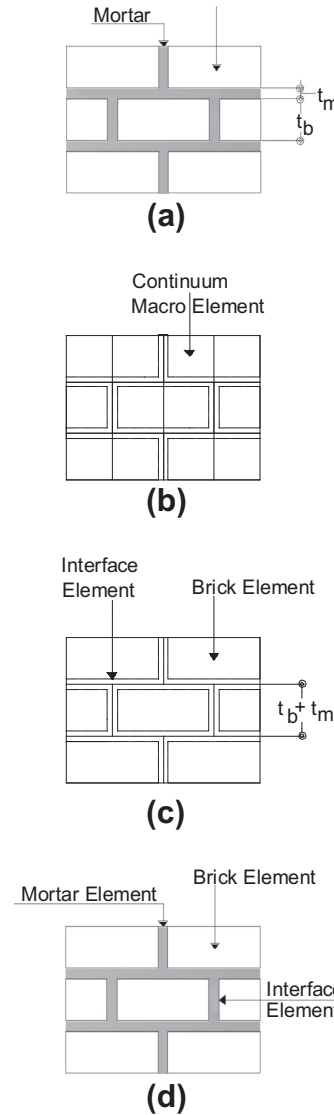


Fig. 3. Masonry modeling strategies: (a) masonry sample; (b) macro-modeling; (c) simplified micro-modeling and (d) detailed micro-modeling.

acceleration, and plastic energy dissipation (viscous or hysteretic). 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 [47]. 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 performance of the building for the different levels of the 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 the limits.

For the case of masonry structures a new damage index is proposed by Asteris [1], which employs as response parameter the percentage of the damaged area of the structure relatively to the total area of the structure. The proposed damage index, [DI], for a masonry structure can be estimated by:

$$[DI] = \frac{A_{fail}}{A_{tot}} \times 100 \quad (7)$$

where A_{fail} is the damaged 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 an interplay between demands and appropriate performance objectives. The Engineer then has to develop 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 safety or collapse prevention, given that ground shaking of specified severity is experienced.

In the past, the practice of meeting performance-based objectives was already included in design practice, but it was rather informal simplistic and non-standard. Some Engineers would characterize performance as life-safety or not; others would assign ratings ranging from poor to good. This qualitative approach adopted for 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 [48]). 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. In fact, other approaches could be used, according to the recent European Codes (EC8 [49]), based on a more engineered (and more detailed) estimation of damage.

3.5. Fragility curves

One of the problems to be faced and resolved at later stages of the global analysis has to do with the quantitative vulnerability assessment of the building as it is (damaged or not) as well as if will be “modified” after interventions. One of the most important tools seems to be fragility analysis, which provides a measure of the safety margin of the structural system above specified structural performance/hazard levels.

A number of methodologies for performing fragility analysis have been proposed in the past which have been used to assess the behavior of structural systems. Simplified methodologies for fragility evaluation have been proposed by Kircher et al. [50] and incorporated in HAZUS99 [51]. These methodologies assume that the spectral ordinates are log-normally distributed, assuming the variability is represented by the logarithmic standard deviation.

The importance of fragility analysis in various stages of risk assessment, loss estimation, and decision making in consequence-based engineering to achieve the desirable long-term objectives of loss reduction and mitigation using the most efficient intervention measures was indicated in Wen and Ellingwood [52]. Fragility functions were developed in Pagni and Lowes [53] to identify the method of repair required for older reinforced concrete

beam-column joints damaged due to earthquake demands. A methodology for the risk assessment of reinforced concrete and unreinforced masonry structures was presented in Kappos et al. [54]. A fragility analysis for the assessment of reinforced concrete structures with soft ground story and short columns was presented in Lagaros [55]. Based on fragility analysis, Omidvar et al. [56] developed fragility curves for unreinforced masonry structures in Iran and showed that their vulnerability is larger than the vulnerability of the similar types in the project Risk-UE. A very interesting analysis procedure, for the case of historical masonry structures has been recently proposed by Milani and Venturini [57]. Namely, the authors proposed a novel 3D homogenized FE limit analysis software for the fragility curve evaluation of entire existing masonry churches.

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 out 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.

Fragility is evaluated as the total probability of a response parameter R exceeding the allowable response value r_{lim} (limit-state), for various earthquake intensities I . In mathematical form, this is simply a conditional probability [58,59] given by:

$$\text{Fragility} = P[R \geq r_{lim}|I] = \sum_j^3 P[R \geq r_{lim}|I, C]P(C = c_j) \quad (8)$$

where $P(C = c_j)$ is the probability that capacity c_j occurs. In the following examples basic steps for the development of the fragility curves, are shortly presented.

4. Technological issues

4.1. General principles

Due to the significant differences regarding the applied approaches and methodologies as well as the relevant basic data (actions, resistances, etc.) concerning the assessment and the redesign of old masonry structures, different techniques and materials could be applied, of various levels, not to mention recent possibilities

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]	$\geq 20\%$	$10\% \leq DI < 20\%$	$< 10\%$
	Collapse prevention	Life safety	Immediate occupancy

related to structural control techniques (mainly passive, e.g. by means of fluid viscous or other dampers).

Therefore, emphasis should be given on certain basic issues and aspects, as follows:

- (a) Analysis and verification has to cover not only the superstructure of the building but also its basement (if any) and its foundation; after all, in many cases, major problems are faced due to high-action-effects or adverse ground conditions, heavy new adjacent buildings or deep excavations, etc.
- (b) Problems of structural irregularity, in plan and/or in elevation, have to be properly identified and faced, since local demands under a severe earthquake could be excessive and difficult to be 'captured' by means of a conventional redesign (based, e.g., on elastic/linear approaches, in terms of forces or stresses).
- (c) The overall structural integrity and proper connection (and function) between all primary load-bearing elements (of all kinds, vertical and horizontal ones) should be studied in detail.
- (d) All predominant weaknesses of the old masonry, i.e. those due to cavity or double-leaf walls or due to premature out-of-plane failures, should be eliminated (as far as possible).
- (e) Finally, the extent or the degree of structural 'improvement' or upgrading (strengthening) should be carefully examined and decided, based on clear and quantitative performance levels (target structural behavior/level of expected damage), compatible with the overall 'value', importance and use of the building.

To this end, various techniques and materials (ranging from lower to higher levels) have been used (successfully or not) and reported in the technical literature, while an inventory of some of the possible structural interventions could be found in the Eurocode 8 – Part 3 (CEN, 2005) [49] (informative Annex C).

Nevertheless, normative (or even informative) guidelines or rules regarding mechanical characteristics, modeling, re-dimensioning, minimum/maximum requirements, etc. are still missing.

Within this framework, a combination of simple and conventional interventions is commonly applied for old masonry buildings, with emphasis on the following two principles:

- i. Proper justification of the type of the foreseen intervention and quantitative estimation of its expected effect on the structural response, at global and/or local level, on a type-by-type basis and not collectively (i.e. not for the combined effect of all of them).
- ii. Proper selection of materials and techniques, again on a case-by-case basis, combined with clear quantitative requirements and provisions regarding the quality control and the assessment of the workmanship and the real effectiveness of the intervention scheme.

4.2. Structural interventions, repair and strengthening techniques

Structural 'improvement' and strengthening (beyond repair), from an engineering point of view, could be classified in quite a few main categories for the superstructure of old masonry buildings, as it is shortly presented here below; techniques for interventions on the foundations and/or the sub-soil of buildings, such as under-pinning (in successive steps), instatement of root- or micro-piles and soil improvement by grouting (of various types) are beyond the scope of this paper (see, also [60–62]).

4.2.1. Techniques for the upgrading of diaphragmatic action of roofs and floors

Although a diaphragmatic action is not a prerequisite for an adequate seismic response of quasi-regular and low-rise masonry buildings, it is well recognized that an increased horizontal strength and stiffness of roof and floors is favorable, especially if this is combined with proper connections between these elements and the masonry walls in all directions. In addition, the need for 'diaphragms' could be increased, in cases of heavy roofs and floors and/or discontinuities in plan or in elevation.

In this respect, double or even triple planking may be used together with horizontal bracing elements, made of wood or steel, while in the majority of cases the intervention is supplemented with the instatement of horizontal frames or trusses with dense or continuous connections and anchorages to the walls, combined with ties, belts, etc.

Special attention should be given at all connections between the horizontal and the vertical load-bearing elements, while in many cases there is a need for an enhancement of the characteristics of the masonry layers at eaves as well as at floors level, not to mention possible additional problems due to 'free' façade walls or pediments.

4.2.2. Techniques for the upgrading of structural connections, between intersecting walls or walls and roofs/floors

For any connection to be effective, the conjugate structural elements should present adequate mechanical characteristics; therefore, local (at least) strengthening of the diaphragms as well as of the related zones of the walls (horizontal and vertical ones) is needed (see the previous and the next paragraph). Various techniques and materials could be used for the improvement of connections or the instatement of new ones, such as:

- Partial rebuilding in successive steps, in turns, etc.
- Instatement of external steel ties, straps, etc.
- Instatement of 'blind' connectors, anchors, dowels, etc., especially in the cases of separation cracks between walls.
- Construction of belts, vertical and/or horizontal, external or partly embedded in the masonry walls, etc.

Nevertheless, most of these techniques have not been studied in detail, while certain bad experiences are reported. In any case, the materials and the techniques should be carefully selected and applied, covering all related problems and connections, in an almost uniform way throughout the whole of the building.

4.2.3. Techniques for the enhancement of the response and the characteristics of masonry walls (in-plane and out-of-plane)

The construction of external buttresses, with adequate foundation and connection to the walls is seldom the case in structural interventions, even for "isolated" buildings or structures. In addition, the instatement of new elements aiming at "transforming" the masonry into a really confined or reinforced one is not the case at all. Therefore, old masonry traditional buildings have to "survive" with a limited dissipation capacity and ductility, offered through the techniques according to the previous paragraphs and certain additional ones for the walls themselves, as follows:

- Partial or local rebuilding and recovering, reestablishment of the structural continuity.
- Closing/building of openings, doors and windows, with proper connectors, shear keys, etc.
- Deep repointing/re-jointing (on both faces of the wall, with a limited thickness), probably combined with the insertion of small diameter deformed or helical reinforcement in bed joints.

- Overall upgrading of cavity or double-leaf walls or of thick rubble stone masonry, by means of grout injections and impregnation, securing a reliable degree of monolithic and continuous response.
- Combination of the above technique and that of inserting of dense transversal (with respect to the wall's planes) connectors, made of masonry units and/or special ties, leading to enhanced effectiveness.
- Application of stitching belts in turns, vertical and horizontal, or of mortar coats with light reinforcement, on both faces of the wall, if this is permitted.

4.3. Use of innovative materials for the protection of monuments

Innovative materials, such as dampers, shape memory alloys and FRPs, provide a viable solution for the protection of monuments. Such solutions are inconspicuous and reversible, which are two very important attributes that any method used for the retrofitting of monuments should possess. The application of SMA and other innovative devices in protecting monuments are reported by Biritognolo et al. [63], Croci [64], and more recently by Chrysostomou et al. [65–69], Casciati et al. [70], El-Borgi et al. [71]; Casciati and Faravelli [72]. In Chrysostomou's publications, it has been shown that such methods have beneficial effects in reducing the earthquake loads on the structures as well as modifying their dynamic characteristics in a favorable way. The effectiveness of FRPs for the strengthening of masonry monuments was studied by many researchers both analytically and experimentally (see [73]) and was demonstrated by Croci [64] who has used aramidic and glass FRPs for the strengthening of the vaults of the Basilica of St Francis of Assisi.

5. Case studies

In this section, the application of the proposed method in the case of representative masonry structures in three different regions (namely: Crete, Greece; Aveiro, Portugal; and Askas, Cyprus) will be presented. The methodological steps applied to each structure, as well as the specific characteristics of each masonry structure (and the country's regulatory framework) will be presented in detail. The seismic activity occurring in each country will be also presented, because the aforementioned countries show a multitude of (distinct) levels of seismic activity, which constitute the primary risk factor affecting the masonry structures. In particular, analyses have been performed by means of elastic FEMs, while stresses have been used in an automatic manner, while fragility curves have been derived and compared.

5.1. Case study 1: Historical masonry structure in Greece

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 a step-by-step approach:

5.1.1. 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 ties which are at the levels of the top floor and roof deck at

the four corners of the building. The ties, until today, and despite the corrosion and the subsequent decay, offered a certain degree of consolidation. The floors and roof are wooden structures. The roof is hipped, based on the perimeter bearing walls of the building. Moreover, two additional buildings are attached laterally to the main one, as shown in Fig. 4.

The main building has dimensions of $15.2 \times 15.85 \text{ m}^2$, the west building $2.65 \times 5.05 \text{ m}^2$ and the east $5.65 \times 7.00 \text{ m}^2$. The height of the main building is 10.87 m and the maximum height including the roof is equal to 13.30 m. The ground floor shows almost completely symmetry with a wide main hallway that divides the building into two symmetrical wings. As regards the morphological description, complete symmetry is observed in the facades, which are characterized by elaborated 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 have limited capacity. Several local problems or defect are identified, but not very seriously damages by seismic or geological reasons. As a summary of the damage that has been found in the structure, it includes 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, and corrosion of embedded iron ties at the corners of the walls.

5.1.2. Step 2 – Material characteristics

The materials composing the structure are: natural stone, reinforced concrete elements, wooden roof, timber planking and steel 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: $f_{wk} = 3.050 \text{ MPa}$, $\gamma = 22 \text{ kN/m}^3$, $\nu = 0.3$.

5.1.3. Step 3–5 – Structural modeling

The program used to simulate the structure is the software SAP2000 v14 Nonlinear. With this software an appropriate FEM model to calculate the response of the structure was formed. The development of the finite elements mesh was such that the ideal concentration of masses at the nodes simulates well the real mass distribution. This ensures a reliable simulation of the inertial loads for dynamic analysis. To fully determine the deformation of the system, six degrees of freedom for each node were considered. The six degrees of freedom correspond to three translations, 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 Fig. 5.

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 mesh 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 the anisotropic behavior of the masonry structure was better simulated. Specifically, condensation occurred on the following areas: locations of concentrated loads, perimeter of the openings, corner areas (wall compounds). For the simulation model 5745 nodes, 5197 surface elements, and 120 beam elements were used.

For the analysis of the structure both, static and dynamic loads were taken into account. For static loads apart from the weight of materials, permanent and live-loads were also added. More specifically, dead loads were considered at floors and roofs,



Fig. 4. South facade of the building.

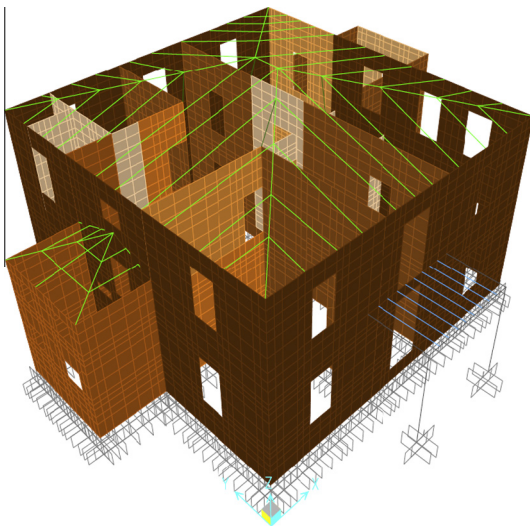


Fig. 5. Colors on the model based on material and thickness of the profiles (southwest corner). (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.)

whereas live loads were considered only at the floors since at the roofs they are negligible. More specifically, live loads were taken equal to 2.5 kN/m^2 , while for the dead loads it was assumed: unreinforced concrete floor 3.00 kN/m^2 , wooden floor 0.6 kN/m^2 , roof tiles 0.5 kN/m^2 . 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 2000 [74]).

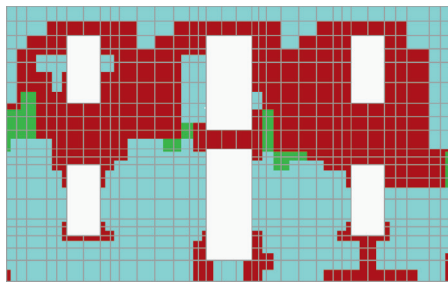
It is well known that Greece is one of the most seismically active countries in the world and the most active in Europe. The long documented seismic history of Greece reports many catastrophes due to earthquakes. It could be stated that it is an “ideal seismological laboratory” for the structural engineer. Namely, earthquakes in Greece are strongly related to everyday life, within the country's course in history. The strong earthquakes that have occurred in this, relatively limited, area of the eastern Mediterranean have affected the history, tradition, religion, arts, building habits, political, social and economic status for a very long time. 5% of the seismic

activity worldwide is exhibited in Greece, whereas 50% of the seismic activity in Europe is exhibited in Greece. It is worth mentioning that the vast majority of the past earthquakes are exhibited within the eastern Mediterranean region. The high seismic activity exhibited in this region is due to the fact that it is located at the boundary of the Africa-Eurasia convergence [75]. Within this framework, the Anatolian plate rotates counterclockwise. As a consequence, the Aegean microplate moves fast towards SW. The external Aegean area is subject to a general compressional stress field and the inner Aegean area experiences a general extensional stress field.

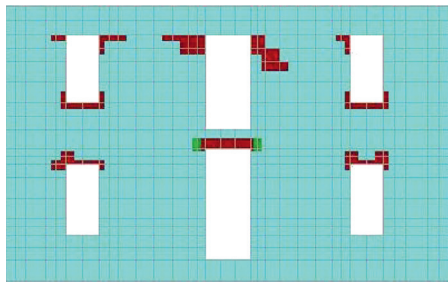
Greece often hosts large magnitude earthquakes, whilst a moderate or small magnitude earthquake is felt every 2–3 days on average. Major shallow earthquakes ($M > 8$, return period of about 1000 years), which can cause such extensive destructions, occur rarely. Although the majority of these earthquakes are shallow, only a few have been recorded as «devastating» for the human environment or for life loss (e.g., the 1881 Chios, 1953 Cephalonia, 1999 Athens earthquakes). This is due to the fact that the majority of these earthquakes are located in the sea and thus most of the energy released is effectively dissipated prior to reaching the populated areas.

5.1.4. Step 6 – Determination of the seismic vulnerability

Failure analysis of the structure: The failure analysis for the existing structure as well as for the studied interventions' scenarios was based on the failure criteria explained in previous sections. In addition to the main computer program used for the analysis (SAP2000), a special computer program, capable of producing a “visual” representation of the failed regions within the structure, has been developed from scratch. The program takes the SAP2000 analysis results as input and gives statistics for the number of failure points, as well as of the type of failure, providing a general view of the probable damage level and the main type of damages within the structure. As an example, the failed points of the internal wall of the temple are depicted in Fig. 6. These diagrams have been proven very useful for the extraction of the required conclusions about the general type of failures in the structure, as well as for decision making concerning the type and the extent of interventions. It can be concluded from Fig. 6 that the particular wall has failed mainly under biaxial tension



(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

Fig. 6. Typical failure areas for the front facade of the structure before and after interventions (PGA = 0.40g).

compression. The analysis concerns a range of peak ground accelerations between 0.08g to 0.40g and masonry tensile strength ranging from 0.05 MPa to 0.55 MPa. Failure results refer to a percentage of the overall failure, as well as to the overall picture, as such of Fig. 6 (type, extent and position of damage).

Probabilistic analysis – Fragility curves: The results concerning the failure areas of the structure were analyzed 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 structure damage for the three structural performance levels (insignificant, moderate and heavy damage) have been determined and the results are presented in Table 2. Figs. 7 and 8 show the damage or fragility curves of the structure before and after interventions, respectively.

These figures show that the fragility curves are important tools in evaluating and ranking the efficiency 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 = 0.20g is reduced by 44% (that is, from 71% probability of damage to 40% probability of damage, as can be seen in both in Table 2 and Figs. 7–9) for one of the studied scenarios.

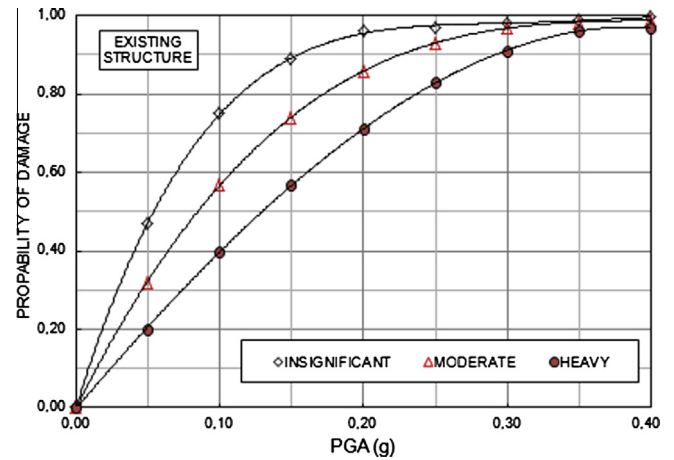


Fig. 7. Fragility curves for the existing structure (before intervention).

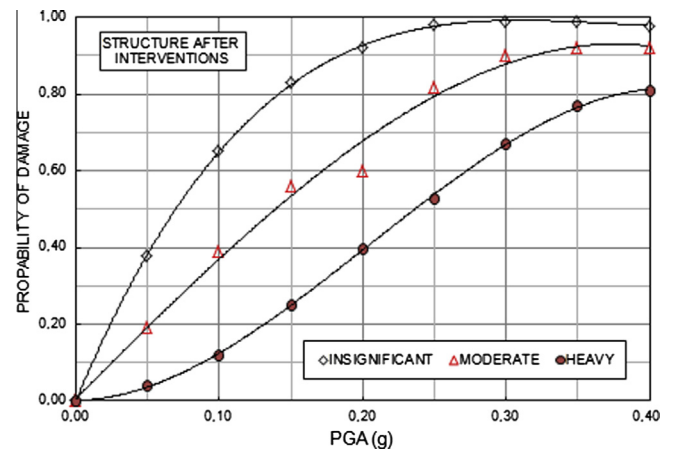


Fig. 8. Fragility curves for the repaired structure (after intervention).

In another paragraph, exhaustive and comparative commentary of the results from Table 2 and Figs. 7–9 will be discussed in more detail, but first another two case studies will be fully presented.

5.2. Case study 2: Residential masonry building in Portugal

A two-story adobe masonry building located in Aveiro, Portugal, was evaluated following the methodology described herein. The various steps followed in this process are thoroughly discussed in this section.

5.2.1. Step 1 – Identity of the structure

Adobe masonry was one of the most predominant materials used in construction in the 19th century and in the first half of

Table 2
Probability of exceeding the damage state for the structure in Greece.

Case	Damage state	Peak Ground Acceleration (PGA) (g)							
		0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
Before intervention	Insignificant	0.47	0.75	0.89	0.96	0.97	0.98	0.99	0.995
	Moderate	0.32	0.57	0.74	0.86	0.93	0.97	0.98	0.99
	Heavy	0.2	0.4	0.57	0.71	0.83	0.91	0.96	0.97
After intervention	Insignificant	0.38	0.65	0.83	0.92	0.98	0.99	0.99	0.9785
	Moderate	0.19	0.39	0.56	0.71	0.82	0.9	0.92	0.92
	Heavy	0.04	0.12	0.25	0.4	0.53	0.67	0.77	0.81

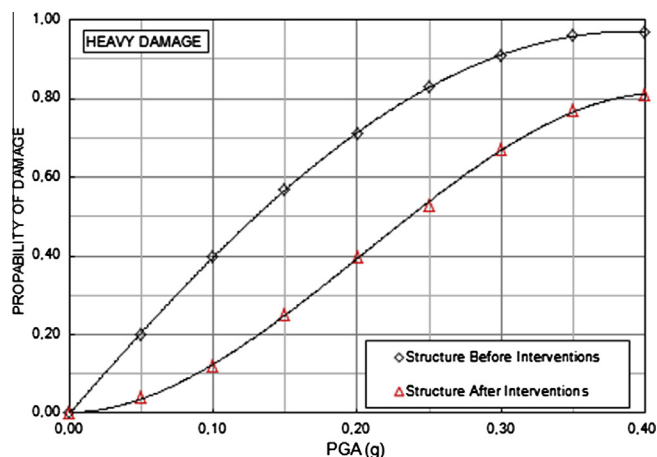


Fig. 9. Fragility curves for the case of heavy damage (before and after intervention).

the 20th century in Portugal, more specifically in the region of Aveiro [76]. The non-consideration of a seismic provisions associated with the current state of degradation of many of these constructions, may characterize adobe buildings as considerably vulnerable to seismic action.

This building typology is usually fairly regular in height and plant and according to the Portuguese Building Census Survey of 2011 [77], mostly comprised by one and two stories structures, and seldom three stories. For what concerns the masonry walls, the adobe bricks in the region of Aveiro were produced based on raw materials locally available, such as earth and lime. When certain type of soils were used, organic fibers were also added in order to improve its quality during the drying process [78,79]. The traditional roof structure is composed by a simple wooden trust system with a hipped shape and ceramic tiles covering. A mesh of wooden beams composes the floors with wooden planks for the covering. Interior walls were frequently made with another solution, usually lighter and thinner such as “tabique” (wattle and daub).

The adobe structure considered in this study has two stories and a vertically and horizontally regular shape as depicted in Fig. 10. Its length is about 14.35 m, with 9.40 m of width and a height of 6 m. The roof rises an additional 1.50 m. The ground floor has a wide-open configuration as it was used mainly for storage purposes, whilst the upper floor contains several division walls, composed by “tabique”.

A technical inspection into the building revealed a significant degradation of the structural elements. These include fissures in the vicinity of the openings (windows and doors), weak connection between the various horizontal structural elements and adjacent walls, cracks on the inner division walls and advanced degradation of the wooden elements.

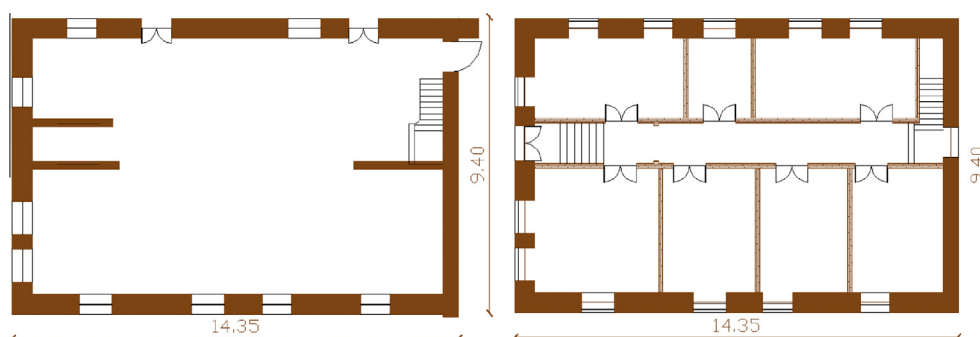


Fig. 10. Drawings of the ground (left) and second floor (right) of the adobe masonry building (dimensions in meters).

5.2.2. Step 2 – Material characteristics

The predominant construction materials are the adobe bricks and wooden beams and planks. In the work of Silveira et al. [78,79], the mechanical properties of adobe bricks taken from structures located in the region of Aveiro are thoroughly investigated. These authors propose a mean compressive strength of 1.32 MPa and a mean tensile strength of 0.17 MPa. Notwithstanding the value and importance of such results, the mechanical properties of the adobe bricks can considerably differ from the properties of the adobe wall, as discussed by Varum et al. [76]. Thus, a decision was made to employ the mechanical characteristics derived from experimental tests on a number of sample adobe walls carried out by Varum et al. [76]. This study indicated a mean compressive strength of 0.33 MPa and a mean secant modulus of elasticity of 600 MPa.

5.2.3. Step 3–5 – Structural modeling

The same modeling approach described in the previous case study was employed herein, in order to allow a direct comparison between both results. Hence, SAP2000 v14 was employed to create a FEM model to assess the response of the structure against increasing levels of seismic action. The adobe masonry walls were modeled using isotropic surface members (shell elements) and the wooden beams were represented through isotropic linear members (frame elements). It was decided not to model the inner division walls, as their reduced stiffness does not influence significantly the structural response of the building and therefore, only their weight was accounted in the numerical model. A representation of the model of the masonry building is illustrated in Fig. 11.

For what concerns the continuity between the various structural elements, based on the aforementioned technical inspection, it was concluded that due to the poor connections between each element, no flexural moment could be transferred. Hence, the appropriate nodes have been configured not to withstand any bending moments.

Besides the weight of the structure (dead-load), a distributed permanent load of 2.0 kN/m² was added to simulate the weight of the division walls, non-structural components and contents; as well as a distributed load of 2.0 kN/m² to model the live-loads. Additional lateral loads were employed to simulate the seismic action. These loads were proportional to a set of peak ground accelerations and were applied based on a response spectrum specific for this region and according to the first three modes of vibration of the structure.

5.2.4. Step 6 – Determination of the seismic vulnerability

For the evaluation of the seismic vulnerability, a set of peak ground accelerations between 0.05g and 0.40g was considered. For each acceleration level, a set of analyses was carried out

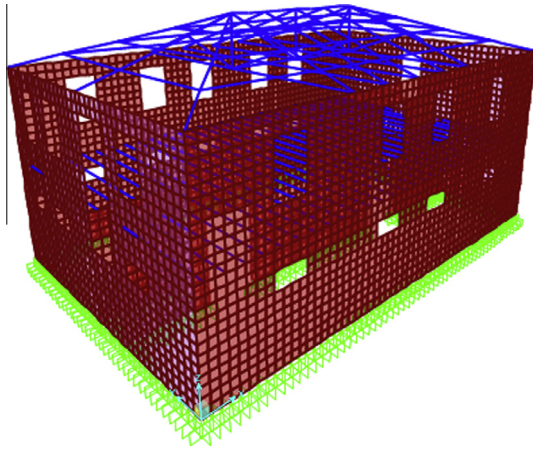


Fig. 11. Numerical model created on SAP2000 of the adobe masonry building.

considering various loading combinations, directions of the lateral loads and a range of tensile strength of the masonry wall (from 0.02 MPa to 0.20 MPa). The failure analysis for each surface element was performed according to the failure criteria described in Section 3.1. These results were plotted and a lognormal cumulative function was derived for each damage state (insignificant, moderate and extensive damage) as illustrated in both Table 3 and Fig. 12.

The same evaluation process was employed after the introduction of the retrofitting/strengthening techniques described in Section 4.2. This retrofitting intervention improved considerably the structural behavior of the masonry building, as indicated by the lower percentage of the overall failure. The resulting fragility model for the retrofitted structure is depicted in Fig. 13.

It should be indicatively mentioned that the probability of heavy damage from a seismic motion with demand represented by $\text{PGA} = 0.20\text{g}$ is reduced by 23% (that is, from 90% probability of damage to 69% probability of damage, as can be seen in both Table 3 and Figs. 12 and 13) for one of the studied scenarios (strengthening scenarios).

5.3. Case study 3: Historical masonry structure in Cyprus

The church of Agios Ioannis Prodromos was used as a case study to apply the methodology outlined in this research. In the following, the various steps of the methodology are outlined in detail.

5.3.1. Step 1 – Identity of the structure

The church of Agios Ioannis Prodromos (Fig. 14) in the village of Askas, Cyprus, contains a vast cycle of important and rare Byzantine wall paintings dating from the 15th and 16th centuries. Robert Gowing et al. from the Courtauld Institute Conservation of Wall Painting Department performed an exploration to clarify the

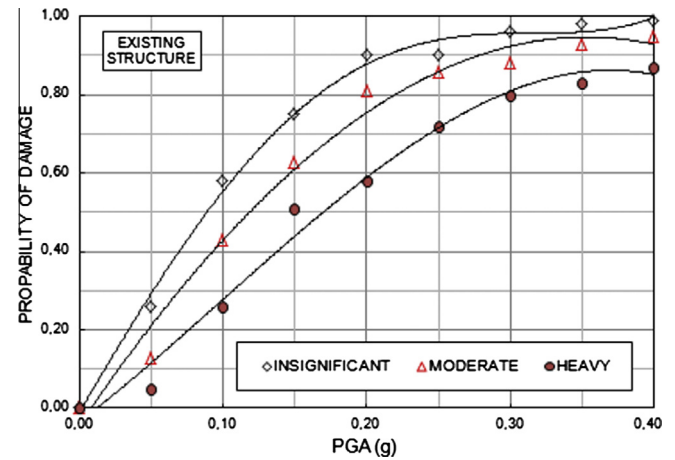


Fig. 12. Fragility model for the original structure.

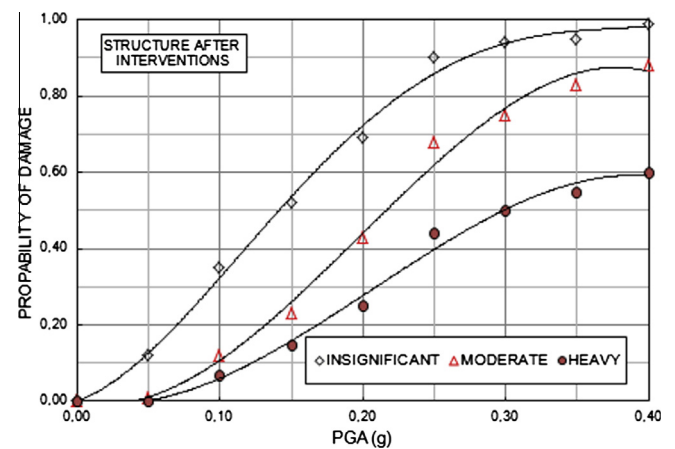


Fig. 13. Fragility model of the retrofitted structure.

various construction phases of the church. According to Gowing's report [80], the earliest building phase appears to consist primarily of a large semi-domed apse and the surrounding east wall. The painted decoration provides the only clue with a proposed date, based on stylistic examination, of around the middle of the 16th century. Gowing reports that extensive rebuilding appears to have occurred, around the beginning of the 17th century, involving the complete enlargement of the body of the church. Constructed as a three-aisled basilica plan church, the design accommodated the original apse and east wall, retaining their painted decoration. The third phase that was noted by Gowing is dated to 1952. This involved the raising of the outer walls to increase the height of the aisles. The exterior changes are visible on the south and east walls with a noticeable change in the construction type. The new

Table 3
Probability of exceeding the damage state for the structure in Portugal.

Case	Damage state	Peak Ground Acceleration (PGA) (g)							
		0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
Before intervention	Insignificant	0.26	0.58	0.75	0.9	0.9	0.96	0.98	0.99
	Moderate	0.13	0.43	0.63	0.81	0.86	0.88	0.93	0.95
	Heavy	0.05	0.26	0.51	0.58	0.72	0.8	0.83	0.87
After intervention	Insignificant	0.12	0.35	0.52	0.69	0.9	0.94	0.95	0.99
	Moderate	0.01	0.12	0.23	0.43	0.68	0.75	0.83	0.88
	Heavy	0	0.07	0.15	0.25	0.44	0.5	0.55	0.6



Fig. 14. The church of Agios Ioannis Prodromos at Askas, Cyprus.

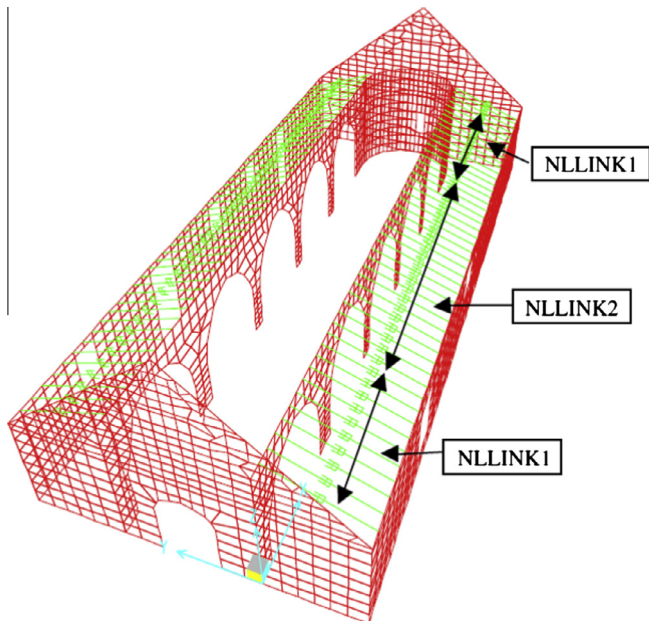


Fig. 15. Position and properties of the dampers.

roof is noticeably lower in pitch as a result of maintaining the old peak height and the increased outer walls.

5.3.2. Step 2 – Material characteristics

The walls of the church consist of diabase stone (unit weight 30 kN/m^3) build in random rubble with the use of mud, and brick

gallets, which are used to fill the gaps between the large stones and to complete the horizontal string line of each bed joint. The modeling of such a construction presents many problems especially in establishing the modulus of elasticity of the matrix. The methodology used in this research was to vary the modulus of elasticity of the shell elements representing the wall until the periods calculated by the eigenvalue analysis matched, as closely as possible, the measured ones. An original effective modulus of elasticity of 2 kN/mm^2 was used for a trial analysis and the final modulus of the updated finite element model was 1.84 kN/mm^2 .

The church has two roofs made out of wood and covered with clay tiles. A modulus of elasticity of 5.8 kN/mm^2 and a unit weight of 3.6 kN/m^3 were used for the elements representing the wooden sections. The weight of the clay tiles was considered to be 0.75 kN/m^2 .

5.3.3. Step 3–5 – Structural modeling

Following the measurements on the church and the establishment of the first few periods of the structure, a computational model was developed so as to model the measured behavior of the church as closely as possible. The program SAP2000NL was used with shell elements and beam–column elements. The beams, rafters and purlins of the roof were modeled using isotropic beam elements, while the walls were modeled with isotropic shell elements. Strong-axis moment releases were applied on the ends of the beams since this was considered to represent more accurately the actual connection of the beams to the main rafter, the external walls, and the arches within the church. The eigenvalue analysis of this model resulted in the following periods of vibration: 0.21 s, 0.17 s, 0.15 s, 0.13 s and 0.11 s, which closely matched the ones measured using ambient vibration analysis and triaxial accelerometers and were 0.23 s, 0.17 s, 0.16 s, 0.13 s and 0.11 s. This matching was effected by changing the effective modulus of elasticity, as explained in step 2 above.

5.3.4. Step 6 – Determination of the seismic vulnerability

In order to study the effects of dampers on the response of the church, it was decided to use time history analysis, and to monitor the interstorey drift of crucial locations. Since this is a monument with very sensitive paintings it was decided to use an interstorey drift of 0.1% as the limit of acceptability of the results for the Maximum Considered Earthquake (MCE, which is obtained by multiplying the ground acceleration of Design Basis Earthquake, DBE, by 1.5). In the absence of an earthquake record in Cyprus, it was decided to use a record from the 1999 Athens earthquake (Metro, 90°) which was scaled to a peak ground acceleration of $0.15g$ for the DBE, which results in $0.225g$ for the MCE. The same record was used in both horizontal directions in the analysis.

Linear dampers were used connecting the inner and outer walls (Fig. 15). Ninety-six dampers were used, of which forty-six had an effective damping of 0.15 kN s/m (NLLINK1) and the rest fifty had an effective damping of 0.20 kN s/m (NLLINK2, Fig. 15). An analysis without dampers was performed to find the locations at which the maximum displacements occurred. This analysis formed the basis for the comparison with the analysis when dampers were introduced. The base shear was also recorded for both analyses. It should be noted that no stiffness was used for the damper, so as to behave as a damping device only. This was checked in the model

Table 4

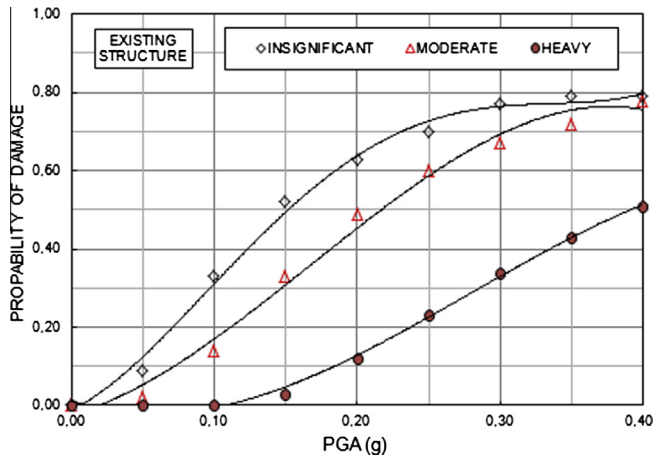
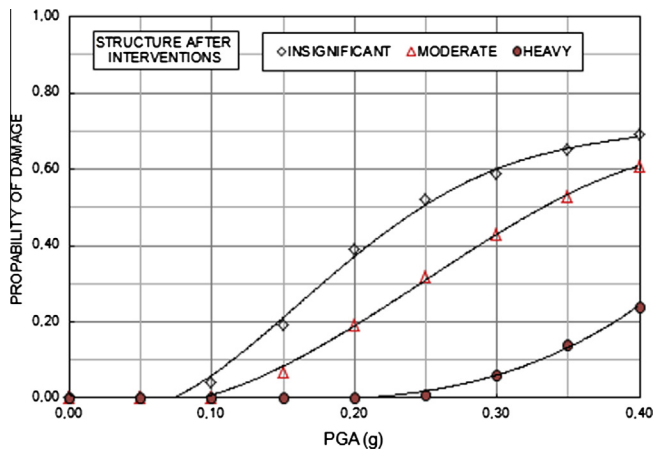
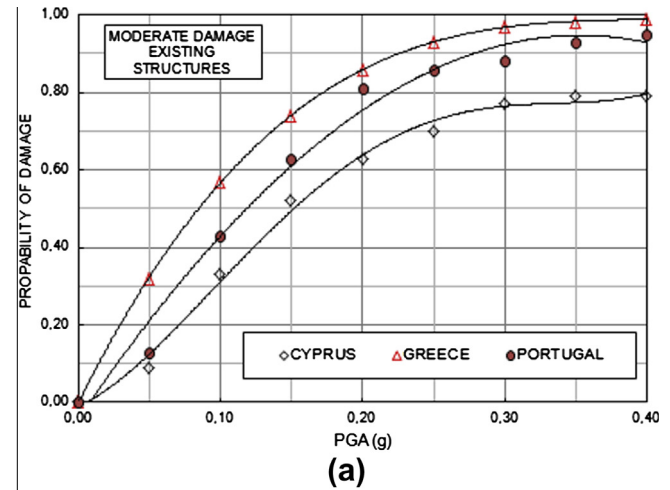
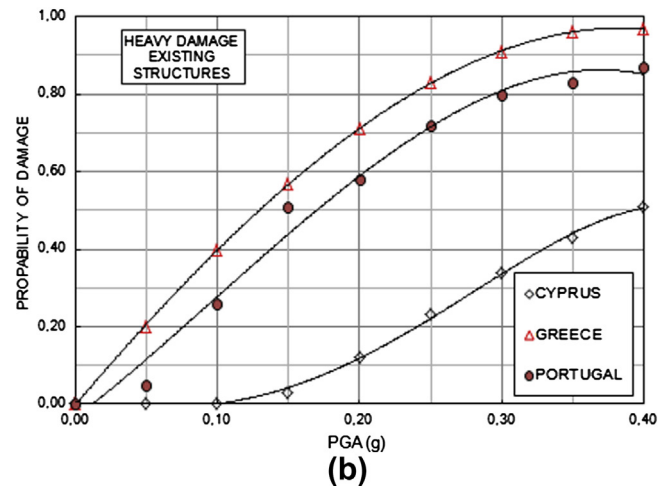
Maximum absolute values of selected locations for the MCE analysis.

Case	Base shear X (kN)	Base shear Y (kN)	Displac. X (mm)	Displac. Y (mm)	Damping force (kN)	Inter. drift (%)
With dampers	904	1299	2.47	13.27	–	0.33
w/o dampers	872	557	2.32	4.07	3.4	0.10
% Difference	–3.5	–57	–6.1	–69	–	–70

Table 5

Probability of exceeding the damage state for the structure in Cyprus.

Case	Damage state	Peak Ground Acceleration (PGA) (g)							
		0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40
Before intervention	Insignificant	0.09	0.33	0.52	0.63	0.7	0.75	0.78	0.8
	Moderate	0.02	0.14	0.33	0.49	0.6	0.67	0.72	0.78
	Heavy	0	0	0.03	0.12	0.23	0.34	0.43	0.51
After intervention	Insignificant	0	0.04	0.19	0.39	0.52	0.59	0.65	0.69
	Moderate	0	0	0.07	0.19	0.32	0.43	0.53	0.61
	Heavy	0	0	0	0	0.01	0.06	0.14	0.24

**Fig. 16.** Fragility curves for the existing structure (before intervention).**Fig. 17.** Fragility curves for the repaired structure (after intervention).**Fig. 18.** Fragility curves for the existing structures: (a) case of moderate damage and (b) case of heavy damage.

and indeed there was no additional stiffness introduced since the periods of vibration of the structure remained the same as in the case without dampers.

The results without dampers have shown that the displacements in the x-direction were below the required limit for the MCE excitation, therefore no dampers were introduced in the longitudinal direction. The results for the analyses with and without dampers for the MCE analysis are shown in Table 4, along with the calculated interstorey drift. The height of the arches is 3970 mm and it was used to calculate the interstorey drifts.

From the results, it is clear that significant reductions of the order of 57% and 70% of the base shear and the interstorey drift in the

y-direction, respectively, were obtained by the introduction of the dampers, achieving the limit of 0.1% that was specified. This will provide sufficient protection to the church and its invaluable paintings.

Following the same procedure as in the previous two case studies, the values of the seismic vulnerability are estimated (Table 5 and Figs. 16 and 17). The effect of the linear dampers on the response of historical and monumental structure can be also depicted from Figs. 16 and 17. The probability of insignificant damage from a seismic motion with demand represented by $PGA = 0.20g$ is reduced by 100% (that is, from 12% probability of damage to 0% probability of damage, as can be seen in Figs. 16 and 17, when the structure is fitted with fluid viscous dampers. This is a considerable reduction, which indicates that fluid vis-

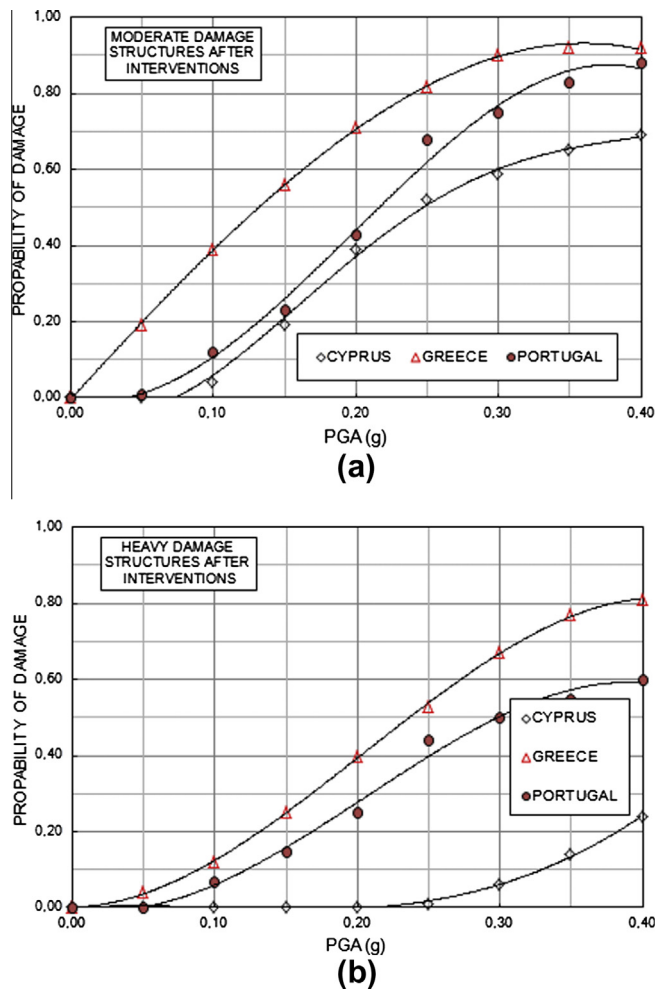


Fig. 19. Fragility curves for the structures after interventions: (a) case of moderate damage and (b) case of heavy damage.

cous dampers can be effective in seismic protection of monumental structures in regions that are at high risk from earthquakes.

5.4. Discussion and comparison of the results

Using the results of all three cases presented above, the fragilities curves were plotted in the same figure for both the existing and the repaired structures (Figs. 18 and 19 respectively). These curves are especially useful since they provide benchmarking (ranking) of the seismic vulnerability of the structures. A ranking such as this helps civil authorities to optimize decisions on choosing among a plethora of structures, which ones present the higher levels of vulnerability and are in need of immediate strengthening.

In order to better present the benchmarking intervention techniques (using the results presented in the above three paragraphs [Tables 5.1.1, 5.2.1 and 5.3.2]), the table embedded in Fig. 20 has been produced. This table shows the decrease of seismic vulnerability for each level of damage (damage level) and for each value of the PGA.

On the basis of this table, the following can be concluded:

- As the value of the PGA increases, the achieved seismic vulnerability reduction is decreased.
- In the case of severe damages, the use of modern intervention techniques (such as the use of innovative materials [i.e. linear dampers] applied in the case of repaired structure in Cyprus) leads to greater reduction of seismic vulnerability compared with traditional retrofitting techniques applied in the other two cases.

6. Conclusions

The vulnerability and assessment of historical masonry structures (before and after structural interventions) remains a considerable challenge from the engineering point view, despite the substantial effort that has taken place in research in the last two decades.

According to the analysis of results for the strengthened structures provided here, it can be concluded that the methodology followed, has been proved helpful to the analysis of existing masonry historical buildings.

Furthermore, it has been shown that the proposed approach offers a ranking method, which helps civil authorities to optimize decisions on choosing, among a plethora of structures, which ones

Case Study	Damage state	Peak Ground Acceleration (PGA) (g)							
		0,05	0,10	0,15	0,20	0,25	0,30	0,35	0,40
Greece	Insignificant	19,1%	13,3%	6,7%	4,2%	0,0%	0,0%	0,0%	0,0%
	Moderate	40,6%	31,6%	24,3%	17,4%	11,8%	7,2%	6,1%	7,1%
	Heavy	80,0%	70,0%	56,1%	43,7%	36,1%	26,4%	19,8%	16,5%
Portugal	Insignificant	53,8%	39,7%	30,7%	23,3%	0,0%	2,1%	3,1%	0,0%
	Moderate	92,3%	72,1%	63,5%	46,9%	20,9%	14,8%	10,8%	7,4%
	Heavy	100,0%	73,1%	70,6%	56,9%	38,9%	37,5%	33,7%	31,0%
Cyprus	Insignificant	100,0%	87,9%	63,5%	38,1%	25,7%	21,3%	16,7%	13,8%
	Moderate	100,0%	100,0%	78,8%	61,2%	46,7%	35,8%	26,4%	21,8%
	Heavy			100,0%	100,0%	95,7%	82,4%	67,4%	52,9%

Fig. 20. Seismic vulnerability reduction of the three studied structures after interventions.

present the higher levels of vulnerability and are in need of immediate strengthening. It also helps the practicing engineer to choose the optimal repairing scenario among a number of competing scenarios.

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