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Vulnerability and Restoration Assessment of Masonry Structural Systems

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ABSTRACT: The Masonry structures are complicated 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 robust earthquake resistant design. However, modeling a real structure to a robust quantitative (mathematical) representation is a very difficult and complicated task. This paper presents an approach toward a solution of the problem. A novel methodology for earthquake resistant design of masonry structural systems, either before or after their repair and/or strengthening, is presented. The entire process is illustrated in the case study of a 4-storey historical masonry structure located in the city of Patras, in Greece.

KEYWORDS: Failure modes, Historical Structures, Masonry, Restoration, Seismic Protection Structural Assessment, Structural Modeling,

1 INTRODUCTION

The majority of the main structural systems for historical structures are 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, earthquake is always their number one “enemy” due to their very bad response to earthquakes (Asteris, 2008). 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 behavior 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 (ICOMOS 1931) the Venice Charter 1964 (ICOMOS 1964), etc.), which make the process of analysis more complicated.

Masonry structures are complicated structures and there is lack of knowledge and information concerning the behavior of their structural system under seismic loads. What can only be said is that typically these structures are more massive than today’s structures and that they usually carry their actions primarily in compression. It should be noted here that most of these historical structures were built with specific consideration given mainly to their geometry and aesthetic quality and less to their structural integrity.

Successful modeling of a masonry historical structure is a prerequisite for a reliable earthquake resistant design. Recent methods of analysis should be very carefully applied on masonry structures. For modern structures, with new industrial materials used (reinforced concrete, steel, etc.), the development of a reliable mathematical model is usually possible, due to the fact that, materials and member characteristics are 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 on that field, until engineers become confident about the accuracy of the modeling.

For the purpose of masonry analysis and design, an operationally simple strength criterion is essential. Masonry has a mechanical behavior, which has not yet been fully investigated. Systematic experimental and analytical investigations on the response of masonry and its failure modes have been conducted in the last decades. There have been numerous analytical criteria for masonry structures (Dhanasekar et al. 1985; Naraine and Sinha 1991; Bortolotti et al. 2005). The main disadvantage of existing criteria is that they ignore the distinct anisotropic nature of masonry; even if they do not ignore that, they consist of more than one type of surface leading to additional effort in the analysis process of the masonry structures (Zienkiewicz and Taylor 1991). According to Zienkiewicz et al. (1969) 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 (Page, 1980 and 1981; Samarasinghe, 1980), it is, therefore, timely 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 (1950) and Prager (1959) 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 determining a strength criterion, but there is still a lot ongoing research on that field.

2 STRUCTURAL RESTORATION METHODOLOGY

Structures of architectural heritage present a number of challenges in conservation, diagnosis, analysis, monitoring and strengthening that limit the application of modern legal codes and building standards. Recommendations are desirable and necessary to both ensure rational methods of analysis and repair methods appropriate to the cultural context (Lourenco, 2008).

Restoration of historical and monumental structures requires a collaborative effort of many disciplines, with structural engineering being only one of them. Restoration engineers, however, cannot afford not to encompass all these aspects and, instead, only focus on the details at hand. Our recommendations, if implemented, can affect other parts of the building that are seemingly unrelated. Our approach constitutes a "holistic" approach, taking the responsibility to consider the whole, as well as the parts.

2.1 *Framework of Thought*

Our work has adopted the philosophy, which has resulted from collaboration within the ICOMOS International Scientific Committee of the Analysis and Restoration of Structures on Architectural Heritage (ISCARSAH) 1; in particular, the *ICOMOS Charter: Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage* (ISCARSAH *Principles*) 2. This framework of thought is delineated by the principles of research and documentation, authenticity and integrity, compatibility (both visual and physical), minimal intervention, and reversibility and are in harmony with those that are the foundation of the *Venice Charter* (1964) and *The Secretary of the Interior's Standards for Historic Preservation Projects* (Morton and Hume 1979).

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, 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 many issues for the preservation of historic structures were discussed. The Charter focuses on achieving harmony between the 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-colour 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 behaviour 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 the 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 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. This is essential for future analysis of continuous processes (such as decay processes or slow soil settlements), 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 in the present.

2.3 *Proposed Methodology*

Based on ICOMOS principles and recommendations as well as on other similar works (Syrmakezis et al. 1995 & 1997; Binda et al. 2000, 2005 & 2006; Asteris et al. 2005 & 2012; Theodossopoulos et al. 2002; Lourenco 2006; Asteris 2008; Onaka 2009; Tassios 2010; Giannopoulos and Asteris 2011; Chronopoulos et al. 2012) 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. For the solution of a problem of this nature, one should go through the following eight distinct steps, namely:

Step 1: Historical and experimental documentation

There are some practical aspects that should be followed before carrying out a rigorous analysis, which are listed below (Tassios, 2010).

a) Long experience shows that the structural design document regarding seismic strengthening 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 Design, in order to account for all initial and consecutive construction phases, previous repairs etc.

b) Description of existing and or repaired damages (visible or possibly hidden ones), together with their in-time evolution; monitoring, be it a short term one, may be helpful.

c) Systematic description of the in situ materials, including their interconnections-especially in the case of three leaf masonry walls. Connections of perpendicular walls are thoroughly investigated.

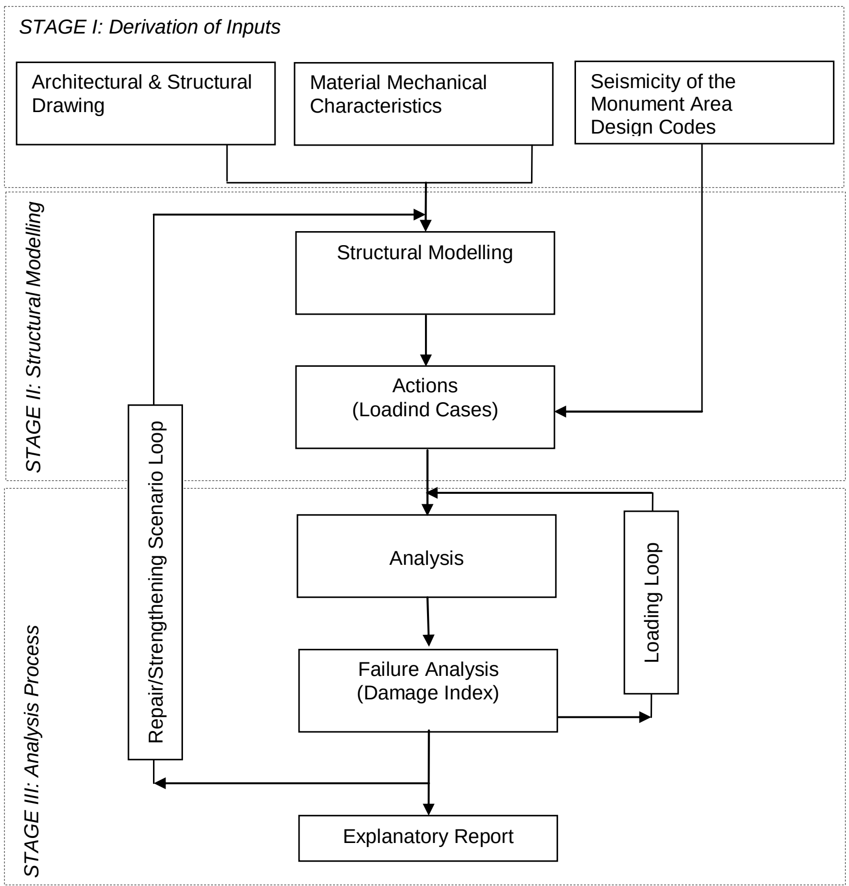


Figure 1: Flowchart with the applied methodology for vulnerability and restoration assessment

c) Results of experimental investigations regarding: geometrical data, internal structure, in situ strength of materials, structural properties of masonry walls, dynamic response of building elements, subterranean data, as well as results of possible previous monitoring installations (displacements, settlements, internal forces, humidity, groundwater level, cracks' opening, seismic accelerations, environmental data etc).

d) Description of the structural system.

f) Description of the soil and the foundation.

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, their modulus of elasticity and Poisson ratio are of primary importance. 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 exist. System resistance such as buckling-effects or local-compression resistance are not considered. Among them the formulae for low-strength stone-masonry proposed by Tassios & Chronopoulos (1986) are combining all parameters affecting the value of f_w.

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

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

where

α is a reduction factor due to non-orthogonality of blocks (α=0.5 for block stones & α=2.5 for rubble stones).

β is a mortar-to-stone factor (β=0.5 for rough blocks & β=0.1 for very smooth-surface stones).

ξ is a factor expressing the adverse effect of thick mortar joints, $\xi=1/[1+3.5(k-k_o)]$, k=(volume of mortar / volume of masonry) & k_o=0.3.

However, for well built masonry structures Tassios (1988) proposed a different compressive strength formula for masonry.

for $f_{bc}>f_{mc}$

$$f_{wc}=\left[f_{mc}+0.4(f_{bc}-f_{mc})\right]\times(1-0.8\sqrt[3]{\alpha}) \qquad (3)$$

and for $f_{bc}<f_{mc}$

$$f_{wc}=f_{bc}\times(1-0.8\sqrt[3]{\alpha}) \qquad (4)$$

where

f_{bc}, f_{mc} are compressive strengths of blocks and mortar respectively.

$\alpha = t_{jm} / h_{bm}$ is the ratio between average (horizontal) joint thickness t_{jm}, and average block height h_{bm}.

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 seems to be generally the most suitable for the analysis. For higher model reliability, specific simulation parameters, such as 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

Different loading cases have to be taken into consideration, including the seismic actions, especially for structures built in seismic areas. Combinations of dead loads, live loads and earthquake loads, have been used, following the general rules provided by codes. Earthquake has to be considered along all unfavorable directions for the building.

Step 5: Analysis

Using input data of the previous steps a Finite Element Analysis is performed and moments (normal-shear) - displacements at the joints of the mesh are calculated. Due to the actual behaviour of plain masonry and the high degree of uncertainty in the previous steps, elastic analysis seems to be the most realistic one for the analysis of such structures, especially before any repair and/or strengthening.

Step 6: Failure criterion & vulnerability assessment

A failure criterion must be established for the definition of the failed regions of the structure. 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 damage index. Based on this index 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 strengthening decisions.

Step 7: Repairing and/or strengthening decisions and reanalysis

According to the results of step 5, all the failed regions are repaired and/or strengthened. The method to be used, the extent of the interventions, the type of the materials, etc., are directly related to the results and are based on semi-empirical expressions for the final mechanical characteristics of masonry (Tassios & Chronopoulos, 1986).

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

Step 8: Explanatory Report

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

3 MATHEMATICAL ISSUES

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 (Syrmakezis & Asteris 2001; Asteris 2010).

$$\begin{aligned} F = {} & 2.27\sigma_x + 9.87\sigma_y + 0.573\sigma_x^2 + 1.32\sigma_y^2 + \\ & 6.25\tau^2 - 0.30\sigma_x\sigma_y + +0.009585\sigma_x^2\sigma_y + \\ & 0.003135\sigma_x\sigma_y^2 + 0.28398\sigma_x\tau^2 + 0.4689\sigma_y\tau^2 = 1 \end{aligned} \tag{5}$$

Their results showed a good correlation with data from the literature. However, this anisotropic failure criterion applies only to the specific masonry material that he was studying. This disadvantage could be reversed if this criterion is expressed in a non-dimensional form, and, as such, can be applied more generally to a plethora of masonry materials. This can be achieved by dividing and multiplying (at the same time) each term in Eq. 5 by one material monoaxial strength raised in the sum of the exponents of the variables σ_x, σ_y, τ (as appeared in each term). It is selected the uniaxial compressive strength Y' to be across the y-axis, which, in terms of the masonry material corresponds to the uniaxial compressive strength denoted with the symbol $f_{wc}^{90^\circ}$. This model was proposed by Asteris et al. (2009).

Equation 5 can thus take the following form:

$$\begin{aligned} F = {} & 17.15\left(\frac{\sigma_x}{f_{wc}^{90^\circ}}\right) + 74.57\left(\frac{\sigma_y}{f_{wc}^{90^\circ}}\right) + 32.71\left(\frac{\sigma_x}{f_{wc}^{90^\circ}}\right)^2 + \\ & 75.34\left(\frac{\sigma_y}{f_{wc}^{90^\circ}}\right)^2 + 356.74\left(\frac{\tau}{f_{wc}^{90^\circ}}\right)^2 - 17.12\left(\frac{\sigma_x}{f_{wc}^{90^\circ}}\right)\left(\frac{\sigma_y}{f_{wc}^{90^\circ}}\right) + \\ & 4.13\left(\frac{\sigma_x}{f_{wc}^{90^\circ}}\right)^2\left(\frac{\sigma_y}{f_{wc}^{90^\circ}}\right) + 1.35\left(\frac{\sigma_x}{f_{wc}^{90^\circ}}\right)\left(\frac{\sigma_y}{f_{wc}^{90^\circ}}\right)^2 + \\ & 122.46\left(\frac{\sigma_x}{f_{wc}^{90^\circ}}\right)\left(\frac{\tau}{f_{wc}^{90^\circ}}\right)^2 + 202.20\left(\frac{\sigma_y}{f_{wc}^{90^\circ}}\right)\left(\frac{\tau}{f_{wc}^{90^\circ}}\right)^2 = 1 \end{aligned} \tag{6}$$

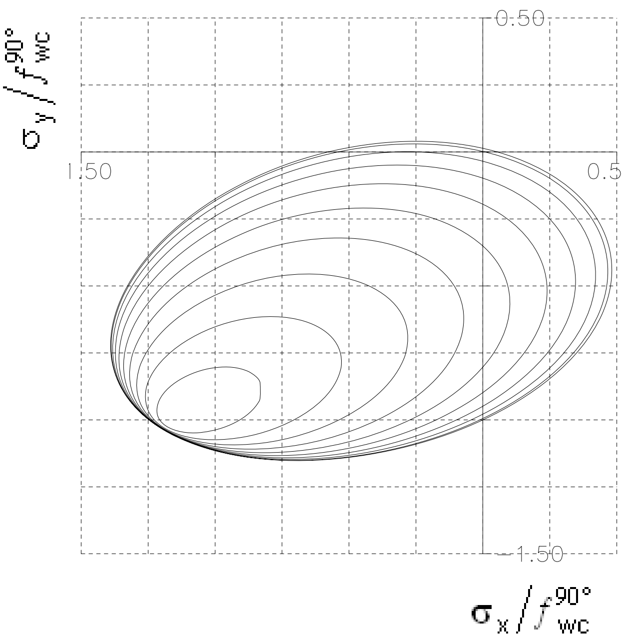


Figure 2: Non-Dimensional Failure Surface of Masonry in Normal Stress Terms (Asteris et al., 2009) ($\tau / f_{wc}^{90^\circ}$ =0.00 up to 0.45 by step=0.05)

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

3.2 *Structural Modeling*

Analytical and experimental studies on the behaviour of masonry walls to in-plane static loads have been the focus of activity of a number of investigators for many years. Masonry exhibits distinct directional properties, due to the influence of mortar joints acting as planes of weakness. Depending upon the orientation of the joints to the stress directions, failure can occur in the joints alone, or simultaneously in the joints and blocks. The great number of the influencing factors, such as dimension and anisotropy of the bricks, joint width and arrangement of bed and head joints, material properties of both brick and mortar, and quality of workmanship, make the simulation of plain brick masonry extremely difficult.

According to Lourenco (2002) & Asteris et al. (2003), the different analytical procedures could be summarized in the following three levels of refinement for masonry models.

- <u>Macro-modeling (Masonry as an one-phase material)</u>
  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 its failure mechanisms. The influence of the mortar joints acting as planes of weakness cannot be addressed.
- <u>Simplified micro-modeling (Masonry as a two-phase material</u>
  Expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit–mortar interface 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 structures.

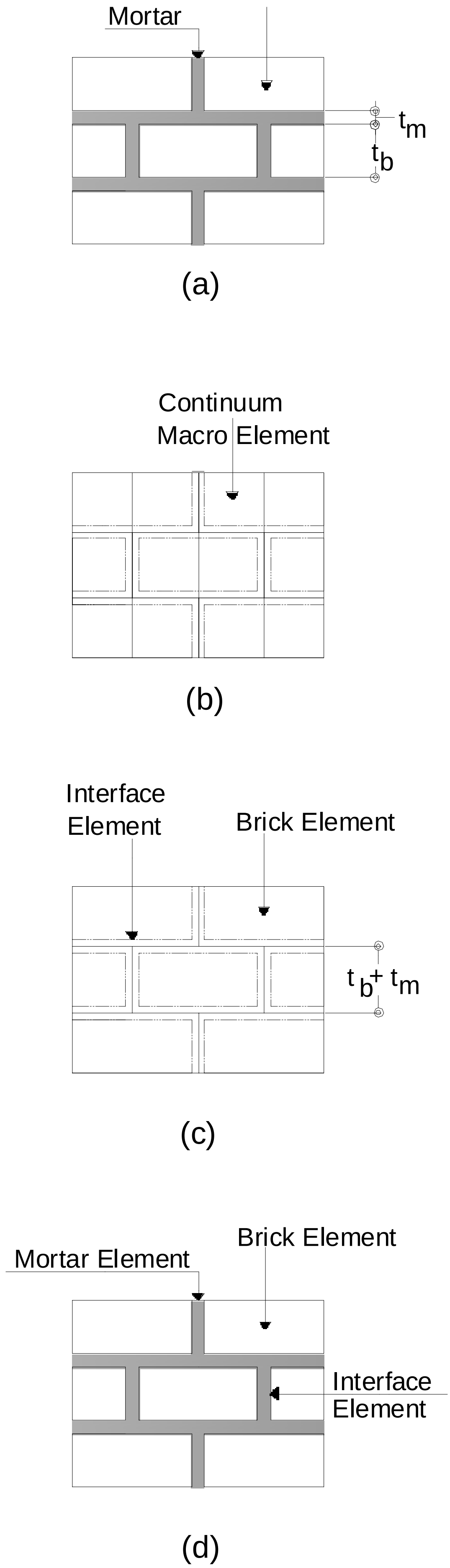


Figure 3: Masonry modeling strategies: a) Masonry sample; b)Macro-modeling; c) Simplified micro-modeling; d) Detailed micro-modeling

- Detailed micro-modeling (Masonry as a three-phase material
  Units and mortar in the joints are represented by continuum elements whereas the unit–mortar interface is represented by discontinuum elements (Fig. 4d). 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. (2001) and Asteris et al. (2003), have proposed simplified micro-modeling procedures to overcome the problem.

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 et al., 1987). 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 is proposed by Asteris (2008), which employs as response parameter the percentage of the failed area of the structure to the total area of the structure. The proposed damage index, [DI], for a masonry structure can be estimated by:

$$[\mathrm{DI}] = \frac{\mathrm{A}_{\mathrm{fail}}}{\mathrm{A}_{\mathrm{tot}}} \times 100 \qquad (7)$$

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

4 CASE STUDY

The methodology described before is illustrated in a comprehensive form, through the case-study of a 4-storey masonry structure of the city of Patras in Greece.

Step 1: Historical and experimental documentation

The building was built at the beginning of the 20th century and has been characterized recently as a historical building. The structural system is composed by porous stones and mortar; the floor system is consisted by wooden boards mounted on wooden beams spanning one direction. The building has suffered several earthquakes during its service life, but has never been repaired or strengthened. A typical plan view is shown in Fig. 4.

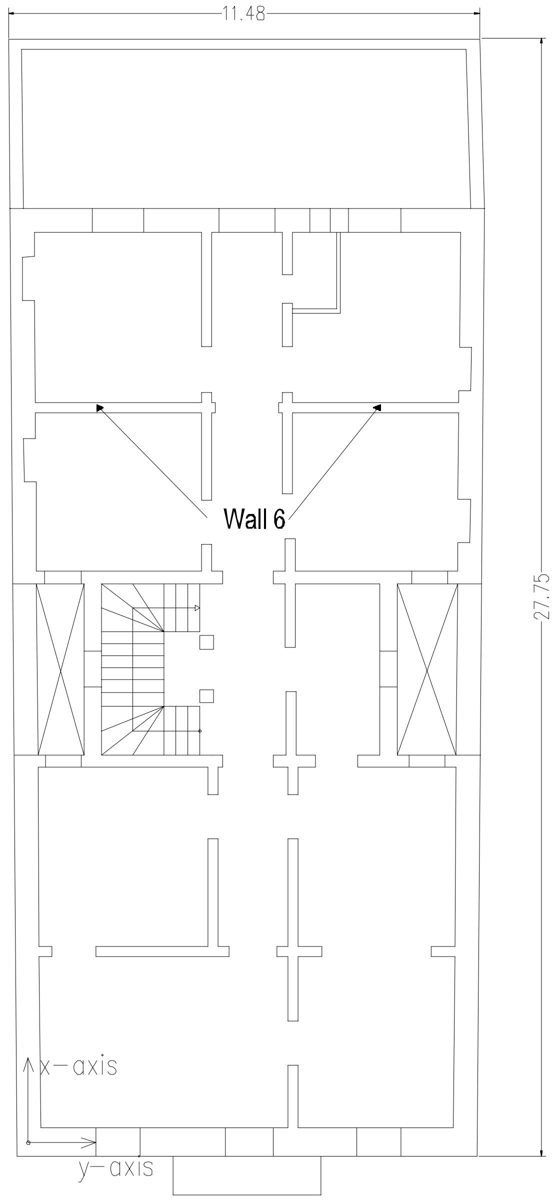


Figure 4: Typical plan view of the examined building

Step 2: Material characteristics

In situ inspection showed that masonry stones were porous stones. Several experiments have been performed in the literature for the determination of the mechanical behavior of stone and mortar; the values shown in Table 1 have been used for the analysis. Taking into account these and using semi-empirical expressions (Tassios & Chronopoulos 1986), the values of masonry compressive and tensile strength, have been calculated.

Table 1. Mechanical characteristics of all materials used.

Material	Strength (Mpa)		Elastic Modulus (MPa)	Poisson ratio
	Compressive	Tensile		
Porous stone	10	-	-	-
mortar	0.75	0.15	-	-
masonry	1.13	0.20	1130	0.30

Step 3: Structural model

Although important improvements have been achieved in analysis techniques in the last decades, the preparation of any analytical model of the historical structure confronts some difficulties. The geometry is a lot more elaborate than for modern buildings and in many cases is very difficult to distinguish between structural and non-structural (decorative) elements. There is also an uncertainty about the materials employed for its construction; as a consequence, some information related to the mechanical properties of the materials is not accurate.

The development of the computational (numerical) model starts with the generation of a 3D geometry model of the historical structure based on the drawings and information taken by previous data. For the simulation of the structural characteristics of the historical structure under study, a 3-D finite element model was developed, using the *Sofistik* design software package (Fig. 5). All masonry walls were modelled using a 4-noded shell element. About 7800 elements were needed to model the structure. For the determination of the strains in each element, six degrees of freedom (6 DoF) were considered. This refers a) to the motion of a rigid body in three-dimensional space and b) translation in three perpendicular axes combined with rotation about three perpendicular axes.

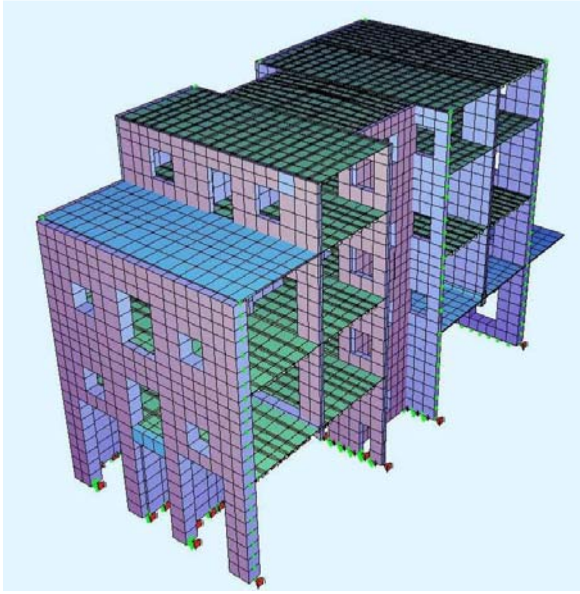


Figure 5: The 3-D FEM model of the building

Step 4: Actions

Nominal values of dead and live loads were specified in the Greek Loading Codes (LC 1945), which are still in effect today. The seismic loads were also specified in the Greek Earthquake Code (EAK 2000).

(a) Dead loads (G)

LC1: Self-weight of masonry walls, wooden floor and roof.

LC6: Additional dead load for the roof = 2 kN/m^2

(b) Live loads (Q)

LC2: 1st storey Live load = 3,5 kN/m^2

LC3: 2nd storey Live load = 3,5 kN/m^2

LC4: 3rd storey Live load = 3,5 kN/m^2

LC5: Roof Live load (snow & wind) = 1,0 kN/m^2

(c) Seismic loads (E)

The seismic action was examined at X & Y direction and at 45^o of X-direction.

LC7: Seismic load – X direction: ε_X= 0,08g / 0,12g / 0,16g

LC8: Seismic load – Y direction: ε_Y = 0,08g / 0,12g / 0,16g

LC9: Seismic load – 45^o of X direction: $\varepsilon_{45}{}^o = \varepsilon_X (\sqrt{2})/2 + \varepsilon_Y (\sqrt{2})/2$

According to the Greek Seismic Code, the seismic zone at the city of Patras is category B, which corresponds to a ground acceleration of 0.08g. However, in Paragraph 5 of the code is highlighted that for parapets and independent masonry walls, the stability and seismic analysis, must be carried out considering a value twice the one indicated; hence the ground acceleration is taken 0.16g for the analysis.

Based on the different loads the following combination actions have been used.

Combination without earthquake

LC21: G+Q=(LC1+LC6)+(LC2+LC4+LC4+LC5)

Combination with earthquake

LC31: G+Q+E_x=(LC1+LC6)+(LC2+LC4+LC4+LC5)+E_x

LC32: G+Q+E_y=(LC1+LC6)+(LC2+LC4+LC4+LC5)+E_y

LC33: G+Q+E_{45^o}= (LC1+LC6)+(LC2+LC4+LC4+LC5)+E_{45^o}

LC41: G+Q-E_x=(LC1+LC6)+(LC2+LC4+LC4+LC5)–E_x

LC42: G+Q-E_y=(LC1+LC6)+(LC2+LC4+LC4+LC5)–E_y

LC43: G+Q-E_{45^o}=(LC1+LC6)+(LC2+LC4+LC4+LC5)-E_{45^o}

(X-direction is perpendicular to the front view of the building (longitudinal direction), and Y-direction is parallel to the front view of the building (transverse direction)).

Step 5: Analysis

Analysis comprises: Modal analysis, stress calculation and failure evaluation.

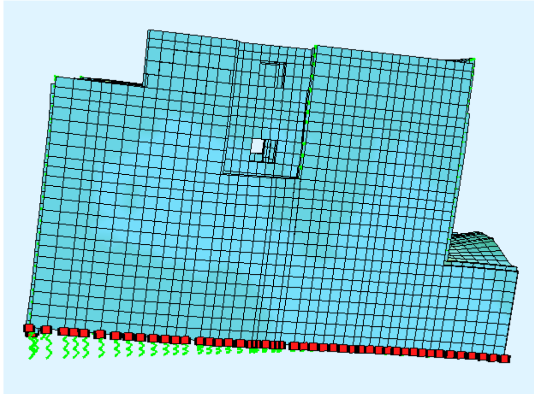
Modal analysis

For the modal analysis, the resulted fundamental modes are lower than expected; such difference can be attributed to the developing of cracks and foundation flexibility in the real structure, which were not taken into account for this study case. The Table 2 shows the natural periods for the first 10modes, where the mass contribution for the fundamental period is given as well.

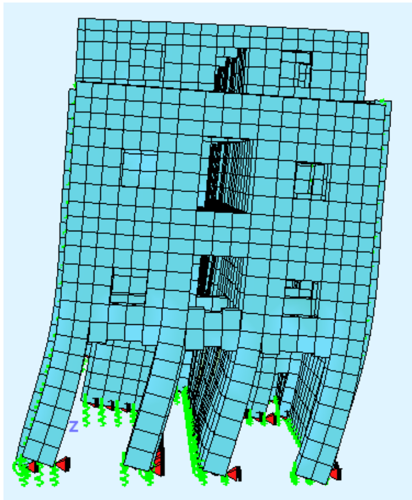
Table 2. Fundamental periods and mass contribution.

Mode	Period [sec]	Mass contribution [%]
1	0.517	31.49
2	0.302	17.75
3	0.274	30.21
4	0.164	58.41
5	0.156	11.93
6	0.132	14.08
7	0.128	13.40
8	0.117	10.92
9	0.109	12.37
10	0.101	8.79

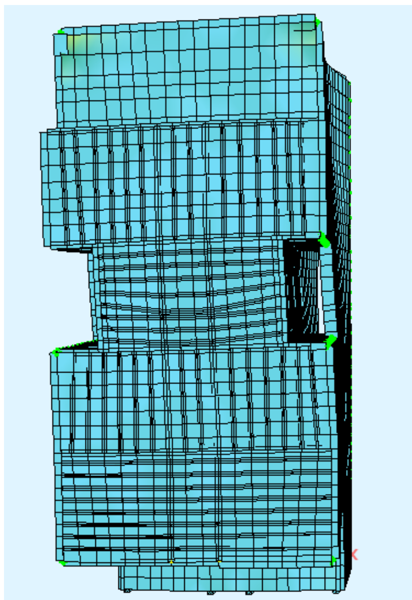
The first mode of vibration acts along Y-direction exciting one side of the structure (Fig. 6b). This mode has an effective mass of 31.49%, and is an important mode for the structure. The third mode of vibration acts along X-direction, with a mass contribution of 30.21% (Fig. 6a). Last, the fourth mode is related in Y-direction having a torsional effect on the structure (Fig. 6c). This mode is the most important for the response of the structure against seismic actions because it affects 58.41% of the total mass. The remained modes are less important, due to the lower mass contribution and basically excite the structure in various torsional modes.



(a) along X-direction



(b) along Y-direction



(c) rotational about Y-direction

Figure 6: The three primarily modes of vibration for the tested building

Stress calculation

Carrying out the Finite Element Analysis, biaxial stresses σ_x and σ_y, shear stress τ_{xy}, as well as displacements and rotations have been calculated, using all the different load combinations described previously. The *Sofistik* software package provides numerical, as well as graphical, output of the results. The results for a typical masonry wall (Wall 6) is shown schematically in Fig. 6 for the biaxial stresses σ_x and σ_y and the shear stress τ_{xy}.

Step 6: Failure criterion & vulnerability assessment

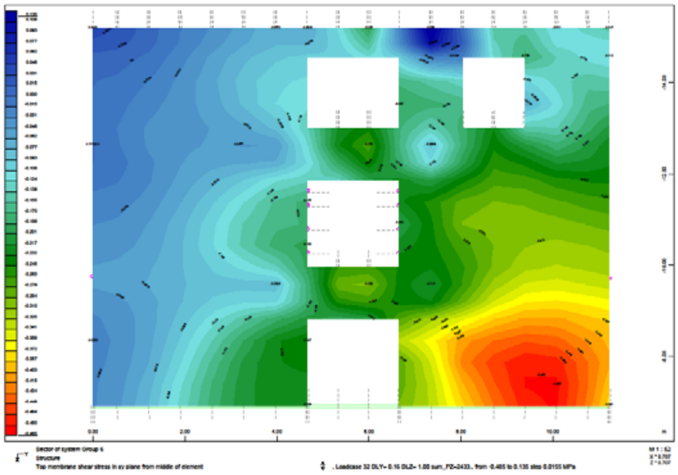
Based on the masonry failure criterion under biaxial stress (Syrmakezis and Asteris 2001; Asteris 2010), a special-purpose computer program, capable of producing a "visual" representation of the failed regions within the structure, has been developed. The program gives statistics for the number of failure 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 a typical wall of (Wall 6) are depicted on Fig. 7. These diagrams have been proven to be very useful for the extraction of the required conclusions about the general type of failures in the structure, as well as for the decision making concerning the type and the extent of interventions. Furthermore, these diagrams are particularly important for confirming the robustness of the proposed structural modeling of the historical structure – the thus obtained failures should correspond to the actual failures of the structure. Indeed, the obtained failures (Fig. 7) are in exact correspondence with the actual (real) failures of the historical structure before its restoration.

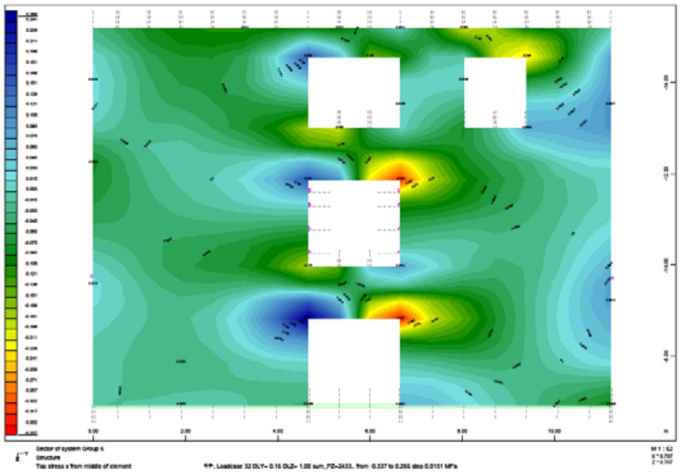
Table 3 shows statistics for the number of failure points and the type of failure. This information provides a general view for the probable damage level and the main type of damages of the structure. The total elements that have not failed (dark blue color) oscillate from 39.4% until 58.5% of all elements of Wall 6 before intervention. Corresponding values will be provided later, after strengthening the wall.

Table 3. Damage index and type of failure for a typical masonry wall before interventions (Wall 6)

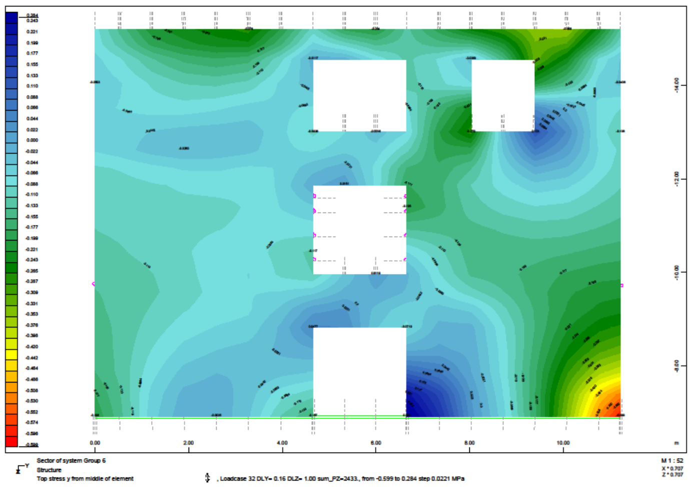
Type of failure	Loading Combination					
	LC31	LC32	LC33	LC41	LC42	LC43
No failure	42,6	58,5	55,6	39,4	46,5	47,5
Under biaxial tension	3,9	5,6	4,9	5,3	4,9	4,9
Under biaxial tension/compression	7,0	6,7	6,0	7,4	6,7	6,3
Under biaxial compression/tension	3,9	2,8	2,8	3,5	3,2	5,3
Under biaxial compression	42,6	26,4	30,6	44,4	38,7	35,9



(a) Contours of normal stress σ_x



(b) Contours of normal stress σ_y



(c) Contours of normal stress τ_{xy}

Figure 6: Contours of normal stresses before interventions

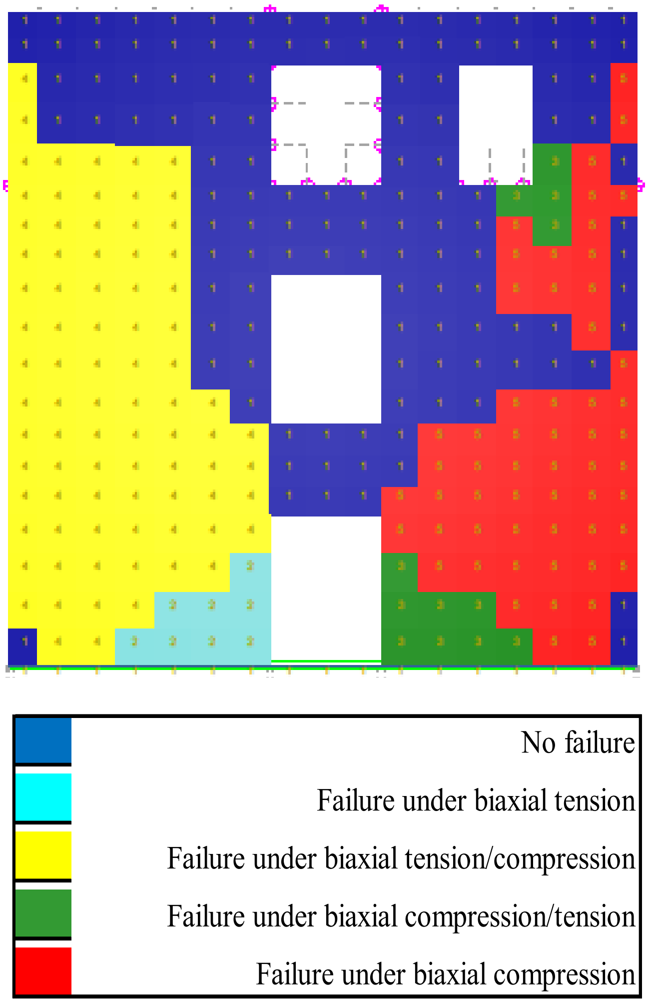


Figure 7: Illustration of failed elements and type of failure for a typical masonry wall before interventions (Wall 6)

Step 7: Repairing and/or strengthening decisions and reanalysis

Following the last conclusion, appropriate decisions for the repair and/or strengthening process of the structure have been taken. It was decided to strengthen most of the walls by concrete jacketing the one side of the masonry walls with a thickness of 8 cm and provision of appropriate additional reinforcement (typically Φ10/15). For the reanalysis of the structure, the new data concerning values of material characteristics, loading and structural layout have been evaluated. The strengths of the new composite materials are modified as following: f_{wc}=1.51 MPa, f_{wt}=0.35 MPa. The results of the analysis after the proposed interventions have shown a significant decrease of the stress levels and thus a significant decrease of the failed elements within the wall.

After intervention, the total elements that have not failed (dark blue color) oscillate from 94.7% until 97.9% of the total elements (Table 4), demonstrating the effectiveness of the strengthening.

Table 4. Damage index and type of failure for a typical masonry wall after interventions (Wall 6)

Type of failure	Loading Combination					
	LC31	LC32	LC33	LC41	LC42	LC43
No failure	97,5	97,2	97,9	97,5	94,7	96,1
Under biaxial tension	1,4	0,4	0,4	1,8	3,9	3,2
Under biaxial tension/compression	1,1	2,5	1,8	0,7	1,4	0,7
Under biaxial compression/tension	0,0	0,0	0,0	0,0	0,0	0,0
Under biaxial compression	0,0	0,0	0,0	0,0	0,0	0,0

5 CONCLUSIONS

The vulnerability and restoration assessment of historical masonry structures remain 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 results of the analysis of the rehabilitated structure provided here, it can be concluded that the methodology followed for the rehabilitation of a masonry historical building has proven to be effective.

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