Earthquake Resistant Design and Rehabilitation of Masonry Historical Structures

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Abstract: A general methodology for the earthquake resistant design and rehabilitation of damaged masonry historical structures is presented. The whole process is illustrated through a case study: The rehabilitation of Zoodohos Pigi Holy Temple in Athens; a structure that suffered extensive damage during the September 1999 Athens earthquake. The procedures followed in the design, together with a description of the actions taken for the rehabilitation of the Temple and its strengthening against future earthquake actions, are reported.

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Introduction

Repairing and strengthening of a historical masonry structure is indeed a difficult and challenging task. Its cultural value and the desire to preserve it for future generations, demands a high level of protection against any possible destruction under future actions, in which earthquake is also included. At the same time, any intervention to fulfill this demand has to not harm what has survived until today. To accomplish this task, a precise understanding of the problems to be faced by the structure, the reasons for them, as well as a sound knowledge of the effect any intervention might have on the structure are needed, so that intervention does not become the cause of future damage to the structure under consideration.

The burden of protecting a historical structure falls mainly on the shoulders of the designer. Definitely, a successful intervention on a monument requires a good comprehension of its structural behavior under static and dynamic (earthquake) loading. For the engineer, to take part in the restoration process of a historical structure through the analysis of its structural system, means mainly to face through this analysis the demanding task of providing the historical structure with the ability 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. And this has to be done under the "tough" restrictions current regulations impose, as well as other limitations, that have their sources in cultural, artistic, and historical needs, stated in a series of scientific Charters [e.g., the Athens Charter (1931), the Venice Charter (1964), etc.], that make the process of analysis even more difficult.

Traditional–historical buildings and monuments are mainly constructed before the ample use of reinforced concrete, with elements and technology based on the experience of the builders alone, without any structural seismic design. Most of these historical structures were built with specific consideration given mainly to their geometry and aesthetic quality and less to their structural integrity. In addition, the aging of these structures and their wearing out due to various causes, such as humidity, ground settlements, pollution, earthquakes, etc., as well as the lack of maintenance, make these structures more vulnerable to future earthquake action than the modern ones. It is therefore of great importance to understand how the structures of yesterday differ from the various types of structures of today.

For the earthquake resistant design of such masonry structural systems, and in order to effectively assess their present condition, the engineer must have a sound understanding of the structural characteristics and expected behavior of the structures, model them for the purpose of analysis, and finally select the appropriate method of repair and strengthening he will use. In this paper, a general methodology for the earthquake resistant design and rehabilitation of masonry historical structures is presented. The procedure is illustrated through the case study of rehabilitation of the Zoodohos Pigi Holy Temple; a structure characterized as an edifice of great historical and cultural significance by the Greek Ministry of Culture.

General Principles for the Earthquake Resistant Design and Rehabilitation of Masonry Historical Structures

Structures of architectural heritage, due to the complexity of their typologies, history, damage, and past repairs, present a number of specific challenges in diagnosis and restoration that limit the ap-

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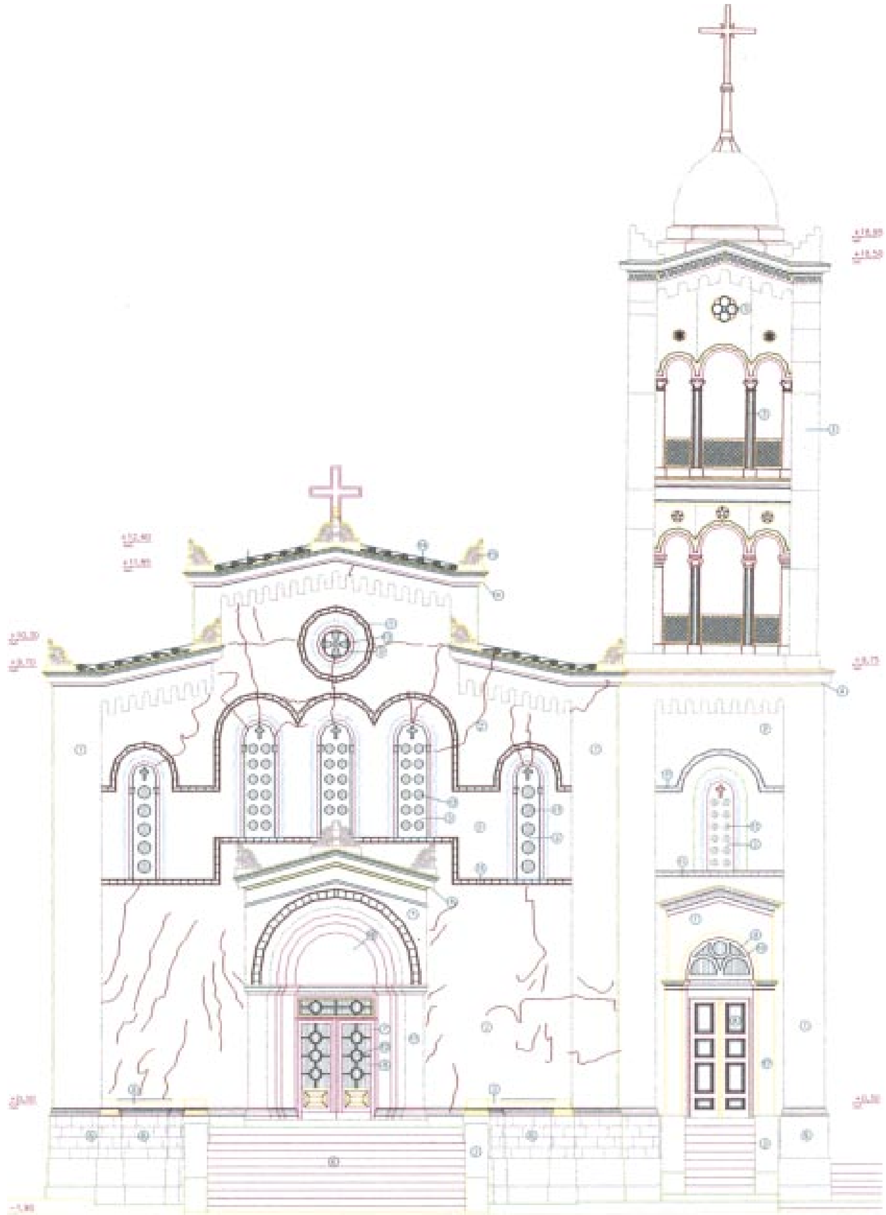

Fig. 1. (Color) Main façade of the church (western view)

plication of modern legal codes and building standards. Masonry structures are complicated structures and there was until today, an unfortunate lack of knowledge and information concerning the behavior of their structural system under seismic loads. Modern methods of analysis, developed generally on abstract mathematical models, should be very carefully applied on masonry historical buildings. 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. Oppositely, for the case of masonry, and especially for the traditional plain one, it seems that there is a long research way to go until reaching a similar level of confidence.

Despite the different structural materials and typology of historical masonry buildings, their damage, resulting from past earthquakes, can be classified in a uniform way, ranging from the damage due to inadequate connection between the walls (separation and out-of-plane collapse of the walls) to damage due to insufficient load-bearing capacity of the walls for carrying the in-plane loads (diagonal cracks and disintegration of the walls, partial or complete collapse of the buildings). Following the same scheme, the technical measures for structural strengthening of historical buildings can be classified into: Technical measures for tying the walls, anchoring and stiffening of the floors (rigid floor diaphragm action should be provided, and the floors should be well anchored into the walls to prevent the out-of-plane behavior), and technical measures for strengthening of masonry walls (these should be uniformly distributed in both orthogonal directions of the buildings and sufficient in number and strength to resist the expected seismic loads).

Successful modeling of a masonry historical structure is a prerequisite for its reliable earthquake resistant design. Nevertheless, the transition from the natural image of the real structure observed, to the mathematical solution of the problem, is a difficult task. The requirements are always put on the real structure, to be finalized in the future, while the analysis is performed on the mathematical model, existing in advance. The real structure fol-

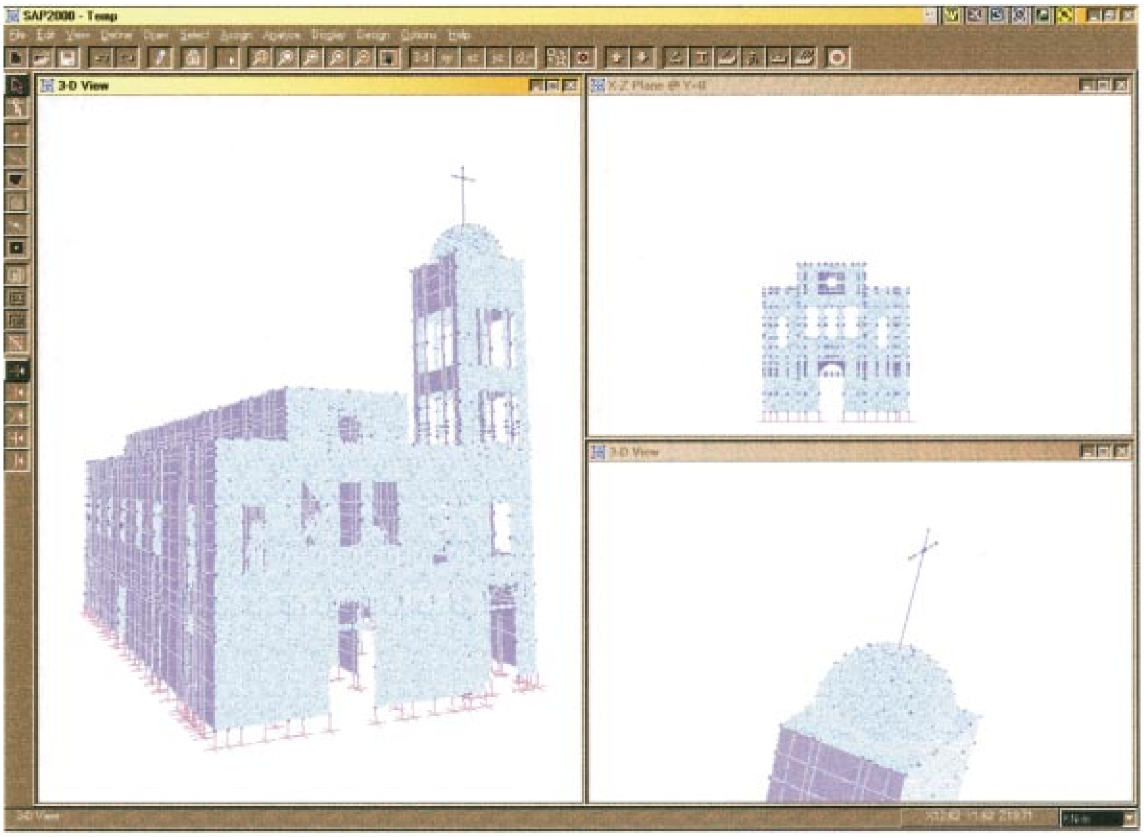

Fig. 2. (Color) Three-dimensional discertization of the Holy Temple

lows the (complicated) physical laws governing its response, but will not necessarily follow the mathematical laws of the model chosen. Oppositely, the model follows its mathematical laws, but not necessarily the physical laws of the real structure.

The analysis of a historical structure (e.g., masonry building) has many similarities, as well as many differences, compared to the analysis of a modern structure (e.g., reinforced concrete shear wall building). Similarities refer to the general assumptions and the mathematical models used; while differences refer to the material properties, the structural system characteristics (e.g., monolithic connections), and the distribution of inertial (e.g., earthquake) loads along the height of the structure. For an adequate modeling of a masonry structure, proper modification of the mathematical models already used for reinforced concrete shear wall buildings, as well as proper modification of models concerning connections is required. As an example, the inadequacy of the lumped mass system model for the earthquake resistant design of a masonry structure is mentioned here, due to the relatively large masses of the vertical walls, compared to the masses of the horizontal (usually wooden) diaphragms in a historical building.

The complex geometry—most of the historical buildings usually have—generate sizeable variations in stiffness due both to heterogeneities in masonry work and to abrupt changes in cross section among the component elements, making it impossible to carry out accurate computations through the application of conventional material strength techniques. In such cases, it is concluded that only the use of the finite element method (FEM) enables the deriving of credible computational results. Implementation of this method allows not only the ascertaining of the overall functioning of the structure, but also the determining of the values of stresses existing in the most "sensitive" parts of the structure (Bruno 2001; Asteris 2003).

Structural Analysis and Repair Methodology

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

- Step 1: Preparation of detailed architectural and structural drawings, describing the current condition of the structure.
- Step 2: Determination of material characteristics. Mechanical characteristics of the materials composing the structure are the basic input data needed for the analysis. In particular, the compressive and tensile strength of the materials, their modulus of elasticity, and Poisson's ratio, are of primary importance.
- Step 3: Structural simulation. A three-dimensional (3D) FEM seems to be generally the most suitable one for the analysis. For a higher model reliability, specific simulation parameters, like the rotation capacity of the wooden floor connection with the masonry wall, the rigidity degree of connections between intersected walls, the influence of spandrel beams, etc., have always to be taken into account.
- Step 4: Actions. Loadings foreseen by the codes for the relevant use of the structure have to be considered. An appropriate seismic loading has also to be taken into account, especially for structures built in seismic areas.
- Step 5: Analysis. Using the data of Steps 1–4, FEM elastic (or elastoplastic) analysis is performed and stresses (normal and shear), as well as displacements at the nodes of the mesh elements, are calculated.
- Step 6: Failure criterion. Taking into consideration conclusions

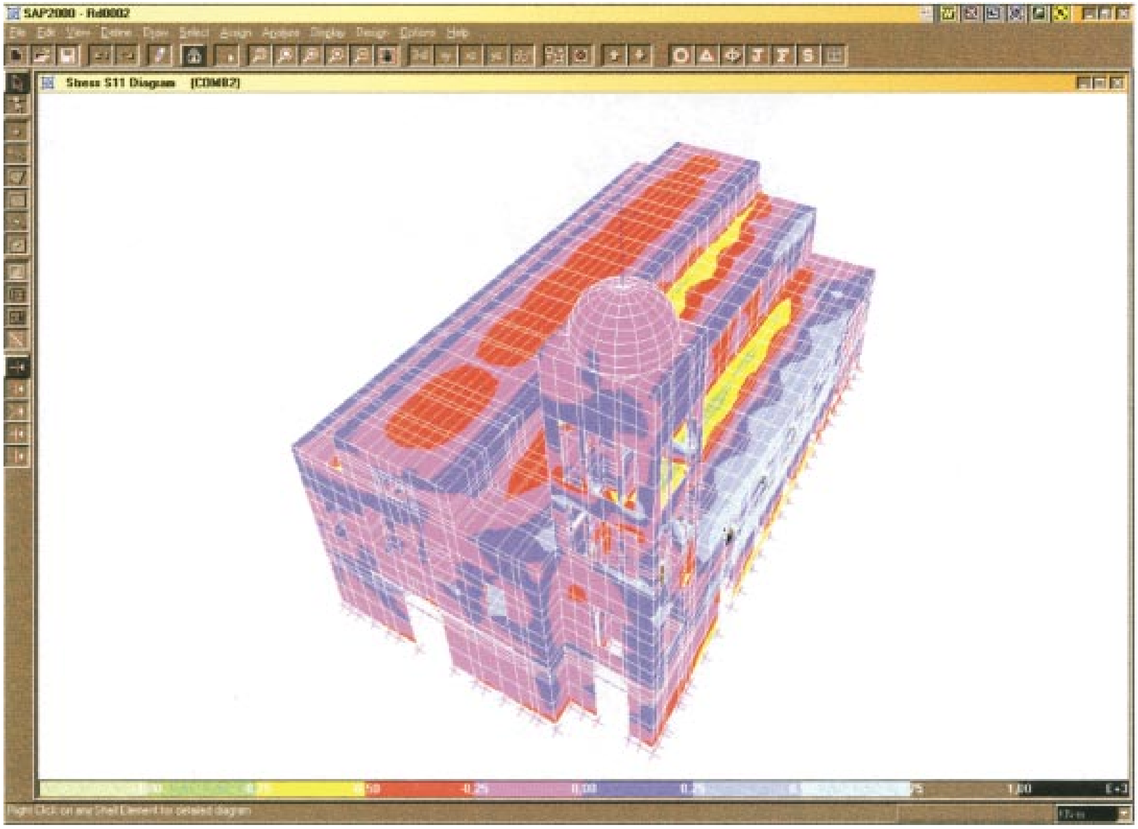

Fig. 3. (Color) Typical graphical output of the analysis results before interventions (biaxial stress σ_{xx}—loading case 2)

made in Step 2, concerning material characteristics, a failure criterion is established and used for the definition of the failed regions of the structure.

- Step 7: Repairing and/or strengthening decisions. Decisions have to be made concerning repair and/or strengthening of the existing structure. The methods to be used, the extend of interventions, the type of the materials, etc., are mainly related to the results of Step 6. It has to be noticed, however, that structural analysis is not always sufficient to give reliable judgements since, sometimes, there are too many uncertainties on material characteristics, inner cracks and discontinuities, permanent deformations and accumulation of stresses in plastic zones, etc., which may impair the results of calculations (Croci 2001). For this reason, qualitative (and subjective) criteria based on the observation of the structure and the historical knowledge of the technologies, phenomena, events, etc. must also be considered before taking any repairing and/or strengthening decisions.
- Step 8: Reanalysis. Within the frame of a final redesign, a new structural analysis has to be performed, using the new material, loading and structural data. Results of the analysis have subsequently to be used in the process of Step 6, leading to a final approval (or rejection) of the decisions already taken in Step 7, concerning repair and/or strengthening of the existing structure.

Table 1. Loading Case Combinations

Loading case	Combinations
1	$1.35G+1.50Q$
2	$1.00G+0.30Q+1.00Ex+0.30Ey$
3	$1.00G+0.30Q+0.30Ex+1.00Ey$
4	$1.00G+0.30Q-1.00Ex-0.30Ey$
5	$1.00G+0.30Q-0.30Ex-1.00Ey$

Note: G= dead loads, Q= live loads, and E= earthquake loads.

Illustrative Example

The above methodology is illustrated here in a comprehensive form, through the casestudy of rehabilitation of the Zoodohos Pigi Holy Temple in Athens (Fig. 1); a structure that has suffered extensive damage during the September 1999 Athens earthquake.

Designed and built in 1845, the temple is characterized by obvious Renaissance elements (arched openings and decorative tiles), Romanic elements (small arches at the eaves of the roof)

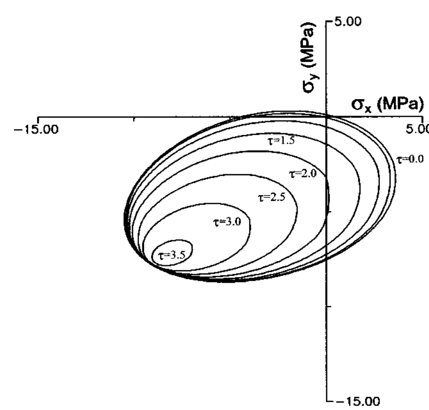

Fig. 4. Failure surface of masonry in normal stress terms

Fig. 5. (Color) Failure results of internal wall, before interventions

and Byzantine elements (analogy to the windows and ceramic stripes around them). The belfry of the temple was built in 1900, has external dimensions of 4.50×4.50 m, 20.00 m in height, and is situated to the north–west side of the temple. In 1950, a fourth nave to the north side of the primary temple, a basement under the nave, and a garret were supplemented. Nowadays, the floor plan of the temple is parallelogram rectangular, with total external dimensions 18.00×30.00 m, and is projected in three levels: Basement, ground floor, and garret (women's quarters).

Damage Description

Extensive damage was caused to the structure during the September 1999 Athens earthquake of moment magnitude $M_w = 5.9$, which occurred at a distance of about 18 km north of the city. The earthquake caused 143 fatalities, 700 injuries, and more than 70.000 people became homeless. The mostly damaged area was located in the north–western suburbs of the city. Thirty buildings collapsed and thousands suffered major or minor damage.

The main damage to the Zoodohos Pigi Holy Temple is located in the central and south nave, where extensive cracks at the cross vaults appear. Extensive cracks can also be observed at the brick arch openings of the north and west side perimeter masonry walls of the structure. The crack pattern formation on the main façade of the Temple has been depicted in Fig. 1. The lack of maintenance of the roof, in conjunction to the absence of insulation and protection against humidity, results in the wearing out of the vaulted structure, making it more vulnerable to future seismic activity. The very good seismic behavior of the belfry, however, has to be noticed.

The main characteristics of the temple's geometry concerning its seismic behavior are:

- The altering of symmetry of the perimeter load-bearing masonry walls of the original structure, due to the later construction of the fourth nave in 1950.
- The bad connection of the roofs' vaults with the perimeter masonry walls of the structure.
- The large dimensions of the structure, which result in large overall horizontal displacements during seismic excitation.
- The lack of horizontal diaphragms at the level of the roof, and their presence at the level of the garrets' floor, which introduce

Fig. 6. (Color) failure results of internal wall, after interventions

high levels of stress and concentrated out-of-plane loads to the perimeter masonry walls in the event of an earthquake.

Analysis and Rehabilitation Process

A short description of the actions undertaken, for each of the aforementioned steps of the previous section, is given below.

Step 1: Determination of the current condition of the structure and drawing up the necessary architectural surveys.

Step 2: Material characteristics determination. The evaluation of material properties was performed through laboratory experiments, using samples of materials (stones and mortar) extracted from the structure. The following values have been estimated: Compressive strength of stones $f_{bc}=44.55$ MPa, compressive strength of mortar (scratch-width method) $f_{mc}=0.74$ MPa, tensile strength of mortar (fragments-test method) $f_{mt}=0.185$ MPa. Taking into account the above mentioned values, and using semi-empirical expressions (Hendry 1990; Drysdale et al. 1994), the values of masonry's mechanical characteristics have been calculated as: $f_{wc}=9.00$ MPa, $f_{wt}=0.22$ MPa, modulus of elasticity $E_w=1000f_{wc}$, and Poisson's ratio $\nu=0.15$.

Step 3: Structural simulation. For the structural modelling of the building, a 3D finite element mesh, consisting of 6,189 shell elements (to model the plane members of the structure, such as walls, vaults, floors, etc.) and 42 frame elements (to model the line members of the structure, such as columns), has been used. The discertization of the structure is shown in Fig. 2.

Step 4: Actions. Earthquake action, along the two main axes of the building, has been considered in both directions (left–right and right–left). Consequently, five action combinations have been used according to the Eurocodes aseismic regulations (European Committee for Standardization 1988), shown in Table 1. Both vertical and horizontal loads have been applied at the model as nodal ones. The base seismic coefficient has been calculated, following modified Eurocodes requirements, as equal to $\varepsilon=0.40g$. For the calculation of the maximum expected ground acceleration, soil conditions, type of foundation, structural damping, importance factor, behavior factor, dynamic characteristics of the structure etc., have also been taken into account.

Step 5: Analysis. Biaxial stresses σ_x and σ_y (homosemous or heterosemous), shear stress τ_{xy} within the elements, as well as displacements and rotations at the nodes have been calculated. The computer program used for the analysis provided numerical as well as graphical output of the results, a small part of which is presented in Fig. 3.

Step 6: Failure criterion. A masonry failure criterion under biaxial stress state recently proposed (Asteris 2000; Syrmakezis and Asteris 2001), has been used (Fig. 4), in order to determine the failed regions of the structure. 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. 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 the internal wall of the temple are depicted on Fig. 5. 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. 5 that the particular wall has failed mainly under biaxial tension compression.

Step 7: Repairing and/or strengthening decisions. Following the conclusions of the previous Step, and keeping in mind Step 6 of the previous section, appropriate decisions for the repair and/or strengthening process of the structure have been made. The rehabilitation solution in this case is mainly based on the idea of enlarging the structural stiffness of the building by increasing the physical and mechanical properties of some component members of the structure. In particular, it was decided to:

- Fill in the cracks of the masonry with cement injections. Grouting is widely used in current practice and it is an accepted opinion that it is one of the most important means of repair of historic masonry structures (Miltiadou 1985). Injecting the grout into the stone–masonry walls has one major advantage as a strengthening method: The intervention is invisible and is therefore ideal for structural strengthening of historical buildings, where the principles of preservation and restoration of historical monuments should be respected, which severely limit the application of many other technical interventions and do not permit the reconstruction or replacement of structural elements. By grouting the stone–masonry walls with cement grout, the lateral load-bearing capacity of the existing masonry of the building has been significantly improved.
- Proper fixing of the masonry walls intersections, using healthy natural stones and metal stirrups (steel connectors).
- Insert a layer of bentonite cement (of 10 cm thickness), reinforced with polypropylene fibers, on the roof of the structure and under the ceramic tiles, in order to prevent the vaulted structure of the cross vaults from humidity and reinforce the structure against future earthquake actions.

Step 8: Reanalysis. For the reanalysis of the structure, the new data concerning values of material characteristics, loading, and structural layout have been evaluated. The tensile strength of the composite material (masonry) has been modified as: $f_{wt} = 0.57$ MPa. The same value for the base seismic coefficient (that is equal to $0.40g$) has been used.

For the reapplication of the failure criterion of Step 6, in order to define the failed regions of the rehabilitated structure, the new analysis results produced in Step 8, are used. The new failed regions, after the rehabilitation of the internal wall of the temple, are presented in Fig. 6, and these are directly compared to the failure results of Fig. 5.

Conclusions

According to the results of the analysis of the rehabilitated structure, it can be concluded that the methodology followed for the rehabilitation of a typical masonry historical building has proven to be effective. The type and extent of the repairs and interventions performed seems to result in a safe behavior of the rehabilitated structure to future seismic actions, without a change in the architectural aspect of the historical building. Although substantial effort has been made in the last decades in order to improve the knowledge in this particular field of structural engineering, more experimental research is still needed in order to obtain data for reliable assessment of the seismic resistance of historical buildings, either existing or strengthened.

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