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Earthquake resistant design of masonry structural systems

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Abstract

Masonry structures are complicated structures and there is, till today, a lack of knowledge and information concerning the behaviour of their structural system under seismic loads. Successful modelling of a masonry historical structure is a prerequisite for a reliable earthquake resistant design. Nevertheless, the transition from the natural image of the real structure observed, to the mathematical solution of the analysis is a very difficult task. This paper presents as a contribution to the solution of the problem. A methodology for the earthquake resistant design of structural systems of historical damaged structures either before, or after their repair and/or strengthening is presented. The whole process is illustrated using the case study of one typical masonry structure, the "Casarma" tower of the Sitia city in Creta.

1. Introduction

Earthquake resistant design of a historical structure, through the analysis of its structural system, is one fundamental step of the whole restoration process. Repairing and strengthening of a historical masonry structure, is

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indeed a difficult and challenging task. Its cultural value and the desire to preserve it for future generations, demands a high level protection against any possible future destruction under future actions, in which earthquake is also included. At the same time, any intervention to fulfill this demand, has not to harm what has survived till 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 historical monument, prerequisites a good comprehension of its structural response, either under static, or under dynamic (earthquake) loading.

For the engineer, to take part to 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 sometimes unique and worthy of special attention. It is necessary that several steps be taken, in order to understand the structural behaviour of the historical structure, taking also into account their priority at national (as well as, sometimes, international) level.

2. General principles for the earthquake resistant design of masonry structural systems

For the earthquake resistant design of masonry structural systems, the engineer must have a sound appreciation of its structural characteristics and its expected behaviour, in order that he may effectively assess the present condition of the structure, model it for the purpose of analysis, and finally make the appropriate choice of the methods of repair and strenghtening he will use. It is therefore of great importance to understand how the structures of yesterday differ from the various types of structures of today.

Masonry structures are complicated structures and there is an unfortunate lack of knowledge and information concerning the behaviour of their structural system under seismic loads. What can be said about these structures is that they are typically more massive than todays' 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 modelling of a masonry historical structure is a prerequisite for a reliable earthquake resistant design. Modern methods of analysis, developed generally on abstract mathematical models, should be very carefully applied on historical (e.g. 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. 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.

The transition from the natural image of the real structure observed, to the mathematical solution of the analysis is a very difficult task. The requirements are always put on the real structure, to be finalised in the future, while the analysis is performed on the mathematical model, existing in advance. The real structure follows 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.

Analysis of a historical structure (e.g. masonry structure), has many similarities, as well as many differences, compared to the analysis of a modern structure (e.g. reinforced concrete shear wall structure). Similarities refer generally to the general assumptions, and to the mathematical models used. Differences, refer to the material properties, the structural system characteristics (e.g. monolithic connections), the distribution of inertial (e.g. earthquake) loads along the height of the building, etc. For an adequate modelling of a masonry structure, proper modification of the mathematical models already used for R.C. 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 earthquake resistant design is mentioned here, due to the relatively large masses of the vertical walls, compared to the masses on the horizontal (usually wooden) diaphragms in a historical building.

3. Methodology

A contribution to the solution of this complex problem, is the systematic methodology proposed by the authors[7]. Following this proposal, the proper solution of the problem should include seven distinct steps. Detailed architectural and structural drawings, describing the existing status of the structure, are always prerequisites for the application of the proposed methodology. These steps are shortly described as following:

Step 1: Material characteristics determination.The characteristics of materials composing the structure are basic input data for a reliable structural analysis. Namely, the compressive-tensile strength of the materials, their modulus of elasticity and Poisson ratio are of primary importance.

Step 2: Structural simulation. A 3-D finite element model seems to be generally the most suitable 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 3: Actions. Loadings foreseen by the codes for the relevant use of the structure, have to be taken into consideration. An appropriate seismic loading has also to be taken into account, especially for structures built in seismic areas.

Step 4: Analysis. Using the data of the steps 1,2,3, FEM linear elastic analysis is performed and stresses (normal-shear) - displacements at the joints of the mesh are calculated.

Step 5: Failure criterion. Taking into account conclusions of step 1 concerning materials' characteristics, a failure criterion is established. On the basis of the FEM analysis results, this criterion is used for the definition of the failed regions of the structure.

Step 6: Repairing and/or strengthening decisions. Decisions have to be taken concerning repair and/or strengthening of the existing structure. The methods to be used, the extend of the interventions, the type of the materials, etc., are directly related to the results of step 5.

Step 7: 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 5, leading to a final approval (or rejection) of the decisions already taken in step 6, concerning repair and/or strengthening of the existing structure.

4. Illustrative example

The methodology following the steps of the previous paragraph is illustrated in a comprehensive form, through the case-study of the Casarma tower, in Sitia, Creta. A short description of the actions undertaken for each one of the aforementioned steps, is given.

1. In situ inspection showed that masonry stones were porous stones. Experiments have been performed using conventional specimens for stones and special tests for mortar, and the following have been estimated: compressive strength of porous stones f_{bc}= 10.0 Mpa, compressive strength of mortar (scratch-width method) f_{mc}= 0.75 Mpa, tensile strength of mortar (fragments-test method) f_{mt}= 0.15 Mpa. Taking into account the above mentioned values, using semi-empirical expressions[2,7], the values of masonry compressive and tensile strength, have been calculated: f_{wc}=1.13 MPa, f_{wt}=0.20 MPa, modulus of elasticity E_w=1000f_{wc}, Poisson ratio ν=0.30.

2. For the simulation of the structural characteristics, a 3-D finite element model consisting of 4540 joints and 4130 shell elements has been created.

3. For the case considered, earthquake action along the two main axes of the building have been taken into account, in both directions (left-right, right-left). Consequently 5 action combinations have been used. Both vertical and horizontal loads have been applied as nodal ones.

Base seismic coefficient has been calculated following modified Eurocodes and Greek codes requirements, equal to $\varepsilon=0.40g$. For the calculation of ε, the maximum expected ground acceleration, soil conditions, type of foundation, structural damping, importance factor, behaviour factor, dynamic characteristics of the structure etc., have been taken into account.

4. By FEM analysis biaxial stresses σ_x and σ_y (homosemous or heterosemous), shear stress τ_{xy}, as well as displacements and rotations have been calculated. The programme SAP-90 used for the analysis, provided numerical, as well as graphical, output of the results (Fig. 2).

5. A special computer programme called "FAILURE" has been developed in the Institute of Structural Analysis and Aseismic Research of NTUA for the failure analysis of the structure[7]. The program is using the FEM analysis results and the mechanical characteristics of the materials for the determination of the failure regions of the structure. The failure regions can be automatically plotted directly on the shape of the corresponding wall. This programme gives also for each of the walls or for the whole structure and for each loading case, 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.

As an example, the failed points for one of the walls (south-easterly wall) of the tower are depicted on Fig. 3. These diagrams have been proved very useful for the extraction of the required conclusions about the general type of the failures in the structure, as well as for decision making concerning the type and the extent of interventions.

On table I, failure statistics for the whole structure before interventions for all loading combinations are given.

On Table II, data summarising the statistics of failed points for the whole structure for the loading combination 2 are given. It can be concluded from this table (as well as from an other four similar tables) that main source of failure for the structure is the biaxial tensile stress state.

6. Following last conclusion appropriate decisions for the repair and/or strengthening process of the structure have been taken.

7. 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.

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Taking into account the same value of behaviour factor q (equal to 1.50) the same value of base seismic coefficient ε equal to 0.40g has been used.

On Fig.4 a graphical output of the analysis after the proposed interventions is depicted. Comparison of Figures 2 and 4 shows the significant decrease of the stress level.

Table I: Failure statistics for the whole structure before interventions for all loading combinations.

Loading case	Failed joints TOP	Failed joints BOTTOM
1	0/4540 (0.00%)	0/4540 (0.00%)
2	332/4540 (7.30%)	255/4540 (5.62%)
3	479/4540 (10.60%)	411/4540 (9.06%)
4	278/4540 (6.12%)	322/4540 (7.09 %)
5	413/4540 (9.09%)	450/4540 (9.91%)

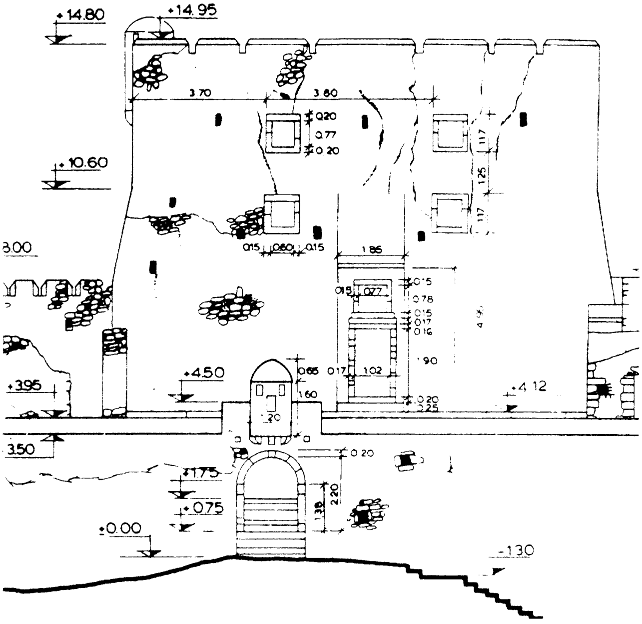

Figure 1: The front facade of the tower (wall S-E).

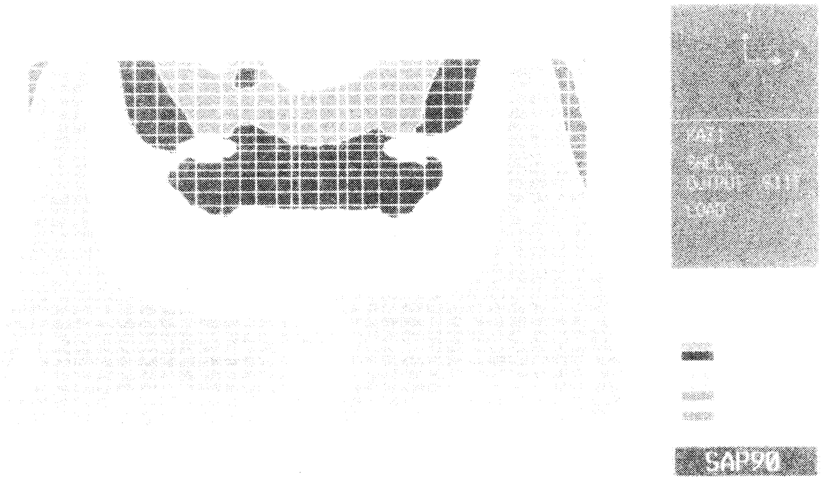

Figure 2: Typical graphical output of the analysis results before interventions (facade wall S-E, loading case 2).

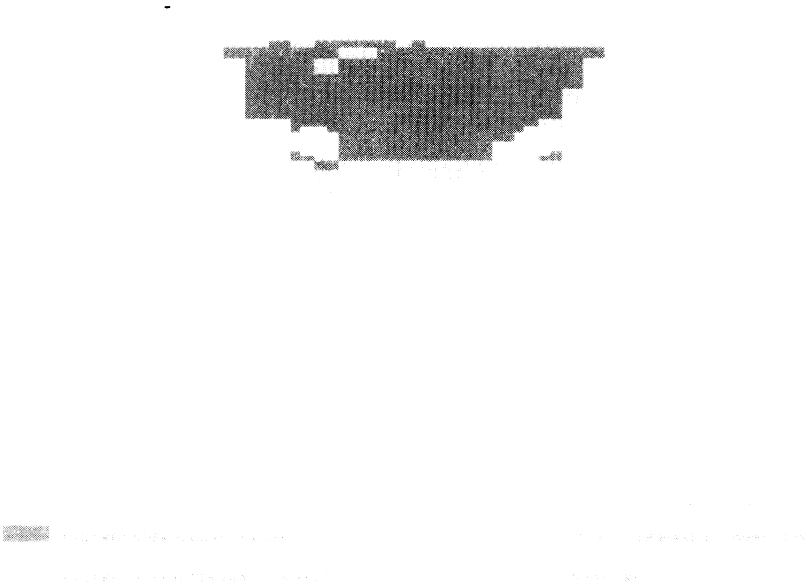

Figure 3: Example of failure results before interventions for the case of wall of Fig. 2.

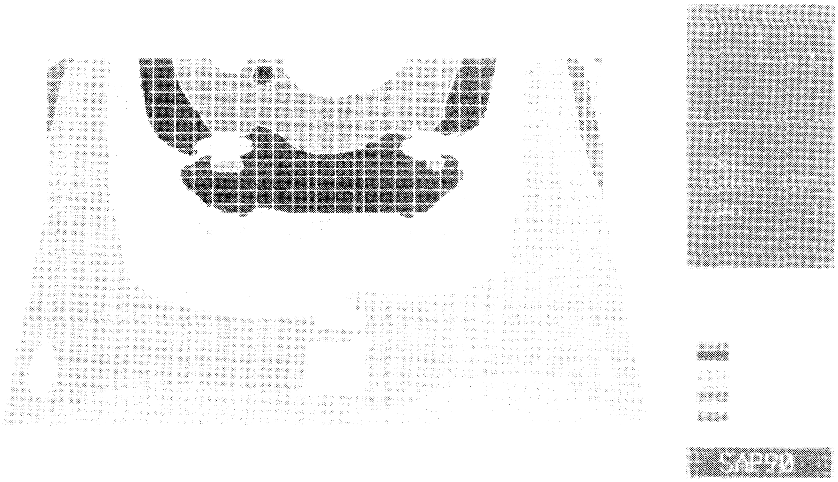

Figure 4: Typical graphical output of the analysis results after interventions (facade wall S-E, loading case 2).

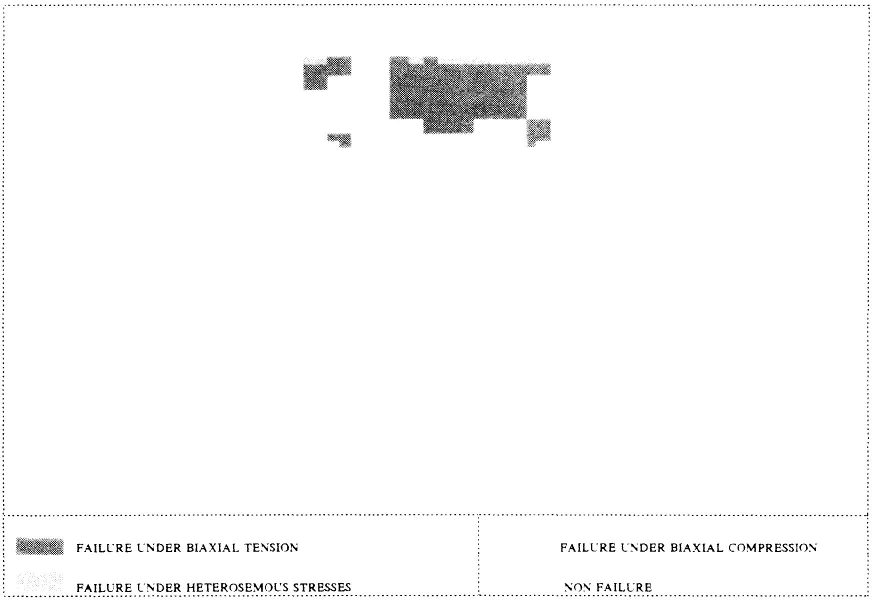

Figure 5: Example of failure results after interventions for the case of wall of Fig. 4.

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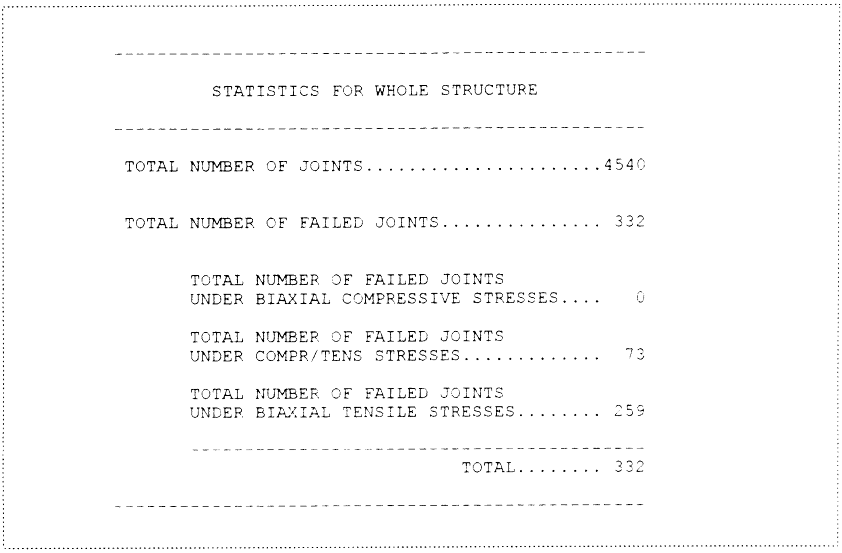
Table II: Failure statistics for the whole structure before interventions for the loading combination 2.



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          STATISTICS FOR WHOLE STRUCTURE

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 TOTAL NUMBER OF JOINTS......................4540


 TOTAL NUMBER OF FAILED JOINTS............... 332


       TOTAL NUMBER OF FAILED JOINTS
       UNDER BIAXIAL COMPRESSIVE STRESSES....   0

       TOTAL NUMBER OF FAILED JOINTS
       UNDER COMPR/TENS STRESSES.............  73

       TOTAL NUMBER OF FAILED JOINTS
       UNDER BIAXIAL TENSILE STRESSES........ 259

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                              TOTAL........ 332

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Table III: Failure statistics for the whole structure after interventions for all loading combinations.

Loading case	Failed joints TOP	Failed joints BOTTOM
1	0/4540 (0.00%)	0/4540 (0.00%)
2	28/4540 (0.06%)	41/4540 (0.09%)
3	69/4540 (1.52%)	90/4540 (1.98%)
4	34/4540 (0.07%)	2/4540 (0.00 %)
5	92/4540 (2.03%)	62/4540 (1.36%)

For the re-application of the failure criterion of step 5, the new analysis results are used. On Fig. 5, the failed points for one of the walls (S-E) of the tower is presented.

On Table III the new statistics data are given. The comparison with Table I is proving a significant improvement of the building's structural behaviour and consequently the suitability of the measures proposed.

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