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Structural analysis methodology for historical buildings

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1 Introduction

The present paper describes a methodology for the structural analysis process of historical damaged buildings either before, or after their repair and/or strengthening. The paper is presented as a final output of a research project undertaken by the authors in the Institute of Structural Analysis and Aseismic Research of NTUA, in the period 1992-94, entitled "Rehabilitation of Historical Buildings in Creta". The project was aiming to present a complete methodology for the structural analysis of such buildings, and apply it in two real examples, two historical buildings in the cities of Chania and Sitia. In the paper the methodology proposed is illustrated using the case study of one of the buildings (in Chania), a 3-story (2 floors + basement) masonry building with wooden horizontal diaphragms (floors, roof), of about 15x15 m plan dimensions. On Fig.1 the front facade of the building is depicted.

2 General Methodology Description

The proposed methodology includes seven steps for the structural analysis of a historical building. Detailed architectural and structural drawings, describing the existing status of the structure, are always prerequisites for the application of the proposed methodology.

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. 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. Among them the formulae:

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Fig. 1 The front facade of the building (drawing by Architect Mrs A. Violaki).

$$f_{wc} = \xi\left[\left(\frac{2}{3}\sqrt{f_{bc}} - \alpha\right) + \beta f_{mc}\right] \quad , \quad f_{wt} = \tfrac{2}{3} f_{mt}$$

(Tassios-Chronopoulos, 1985) are combining all parameters affecting the value of f_w. These parameters are, "α" a factor describing the influence of the blocks' shape and the type of the construction, "β" a factor describing the kind of the masonry (stone or brick), "ξ" a factor describing the bed-joint width and the volume of the included mortar, "f_{bc}" the compressive strength of stones, and "f_{mc} \ f_{mt}" the compressive\tensile strength of mortar.

Values of f_{bc}, have to be estimated experimentally. The values of mortar's strength f_{mc} (compressive) and f_{mt} (tensile), have to be estimated by the specially adopted method of fragments' and scratch-width tests. Modulus of elasticity, if not otherwise defined, can be estimated as the 1000 multiple of f_{wc}. Poisson ratio, can be taken equal to 0.30.

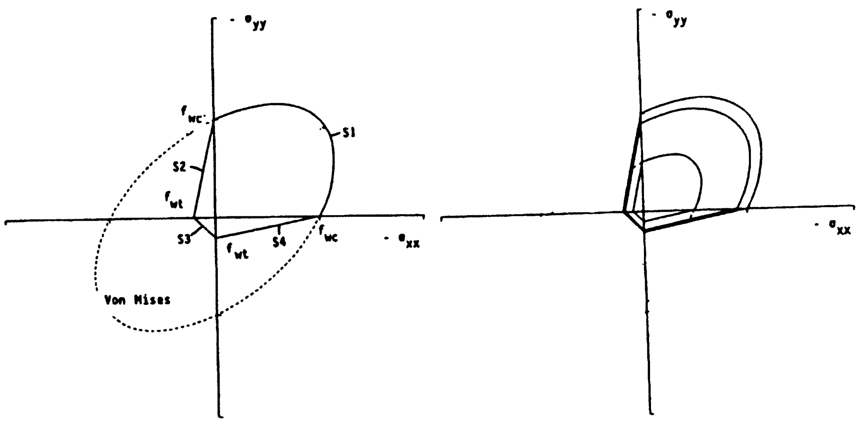
Step 2: Structural simulation.
A 3-D finite element model seems to be generally the most suitable for the analysis. For a higher model reliability, specific simulation parameters, like the rotation capacity of the wooden floor connection with the masonry wall, the rigidity degree of connections between 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 building, have to be taken into considerations. An appropriate seismic loading has also to be taken into account, especially for structures built in seismic areas.

Combinations of dead loads, live loads and earthquake action, is used, following the general rules provided by codes. Earthquake has to be considered along all unfavourable directions for the building. Base seismic coefficient along height distribution is taken into account following the Eurocodes' requirements, appropriately modified.

Step 4: Analysis.
Using the data of the steps 1,2,3, FEM linear elastic analysis is perfomed and stresses (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 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.



(a) (b)
Fig. 2 The proposed failure surface.

For the plane ($\sigma_x,\sigma_y \neq 0$, $\tau_{xy}=0$) the proposed failure criterion is based on the combination of a specific masonry plane failure line for the cases biaxial tension (BT), biaxial tension-compression (BTC), biaxial compression-tension (BCT) and Von Mises failure ellipse for the case of biaxial compression (BC) (Fig. 2a). Therefore the final result is a 3-D failure surface (Fig. 2b) which is constituted by four families of surfaces (S1,S2,S3 and S4) for each one of the previous cases.

In this respect it has to be mentioned here that masonry is at least an orthotropic material. Mechanical characteristics are essentially different along the two principal directions, ie perpendicular and paraller to bed joints. During the work presented in this paper, a simplified assumption was made, taking into account isotropic material. Consequently, no modification of the biaxial failure envelope has been considered for the planes $\tau_{xy} \neq 0$.

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. 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 (BC, BCT, BTC, BT). This information provides a general view for the probable damage level and the main type of damages 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, based on semi-empirical expressions for the final mechanical characteristics of masonry (Tassios-Chronopoulos, 1985).

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.

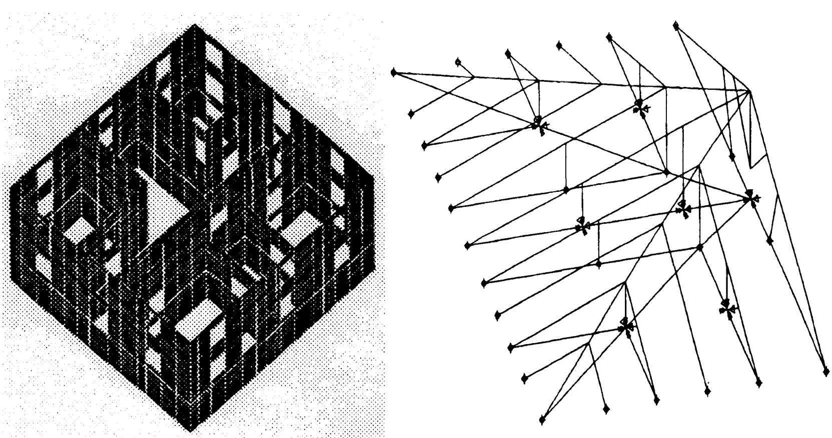


Fig. 3 The 3-D FEM model of the building and the roof.

3 Illustrative example

The methodology following the steps of the previous paragraph is illustrated in a comprehensive form, through the case-study of the building under consideration.

1. In situ inspection showed that masonry stones were limestones and porous stones. Experiments have been perfomed using conventional specimens for stones and special tests for mortar, and the following have been estimated: compressive strength of limestones f_{bc}=25.0 Mpa, compressive strength of porous stones f_{bc}= 7.0 Mpa, compressive strength of mortar (scratch-width method) f_{mc}= 0.725 Mpa, tensile strength of mortar (fragments-test method) f_{mt}= 0.145 Mpa. The following values of parameters α, β ,ξ have been considered:

Material	α	β	ξ
limestone	2.5	0.5	0.90
porous stone	1.5	0.5	1.15

Taking into account the above mentioned values, the values of f_{wc} have been calculated: limestone f_{wc}=1.30 MPa, porous stone f_{wc}=0.90 MPa, mean value f_{wc}=1.10 MPa, modulus of elasticity E_w=1000f_{wc}=1100 MPa.

2. For the simulation of the structural characteristics, a 3-D finite element model consisting of 2788 joints, 2216 shell elements and 3 frame elements has been created. For the simulation of the wooden roof of the building (half part) a 3-D truss model has been created, using 57 joints and 82 elements (Fig. 3).

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. These combinations have been used twice, once with, once without, horizontal diaphragms. 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 ε=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. 4).

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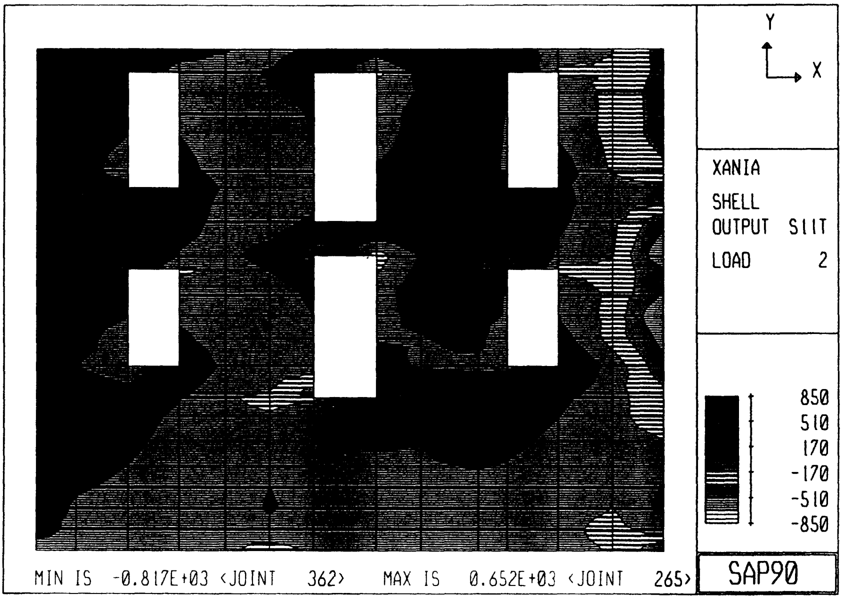

Fig. 4. Typical graphical output of the analysis results before interventions.

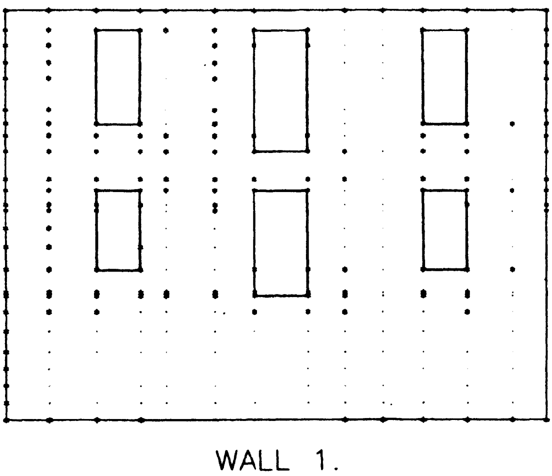

Fig. 5a. Example of failure results before interventions.

5. The failed regions have been plotted using the "FAILURE" computer programme.

As an example, the failed points for one of the vertical walls (wall 1, facade) of the building are depicted on Fig. 5a. On Fig. 5b, the position of the points failed are shown with the values of their stresses, in comparison with the failure surface for a specific level τ / f_{wc} of the failure solid. These diagrams are 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.

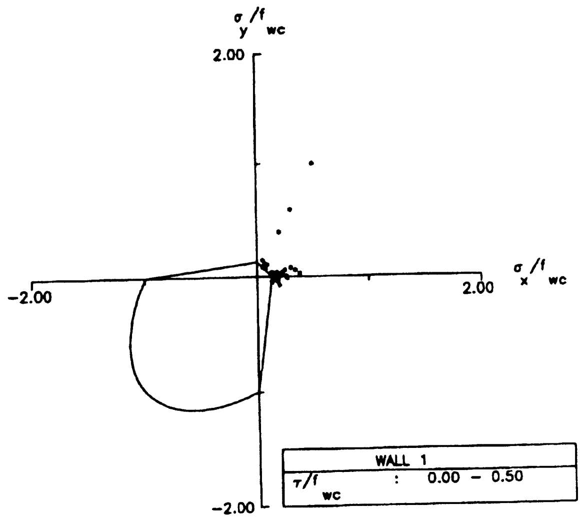

Fig. 5b. Example of failure results before interventions.

On Table I, data are given summarising the statistics of failed points for the whole structure. Following the results of this table the conclusion is that the main source of failure for the structure is the biaxial tensile stresses (BT).

Table I. Failure statistics for the whole structure before interventions.

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-----------------------------------------------
            STATISTICS FOR WHOLE STRUCTURE
-----------------------------------------------
TOTAL NUMBER OF JOINTS......................2788

TOTAL NUMBER OF FAILED JOINTS............... 818

      TOTAL NUMBER OF FAILED JOINTS
      UNDER BIAXIAL COMPRESSIVE STRESSES....  19

      TOTAL NUMBER OF FAILED JOINTS
      UNDER COMPR/TENS STRESSES............. 381

      TOTAL NUMBER OF FAILED JOINTS
      UNDER BIAXIAL TENSILE STRESSES........ 418
      ----------------------------------------
                            TOTAL........ 818
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6. Following the last conclusion, it was decided to use lightly reinforced rendering 2.5 cm thick on both sides of the building's walls, following a deep rejointing. The replacement of deteriorated stones (especially at the cracked areas), as well as the filling - sealing of the cracks has been also proposed.

7. For the reanalysis of the structure, the new data concerning values of material characteristics, loading and structural layout have been evaluated. The strength of the new composite material is modified as in Table II.

Table II. Strength characteristics before and after interventions.

Strength (Mpa)	Before	Influence rejointing	Influence of reinforced rendering	After
f_{wc}	1.10	1.50	0.00	1.65
f_{wt}	0.15	1.70	1.05	0.41

Also the new width of the masonry after the application of the structural interventions has been evaluated using the following expression:

$$t_{w,eq} = t_w + 2t_m(E_m / E_w)$$

where t_w, t_m the width of the wall and the reinforced rendering respectively, and E_w, E_m, the corresponding moduli of elasticity.

Taking into account a new higher value of behaviour factor q (1.50 instead of 1.05) for the new quasi-reinforced masonry, a new value of base seismic coefficient ε equal to 0.31g (instead of 0.40g) has been defined.

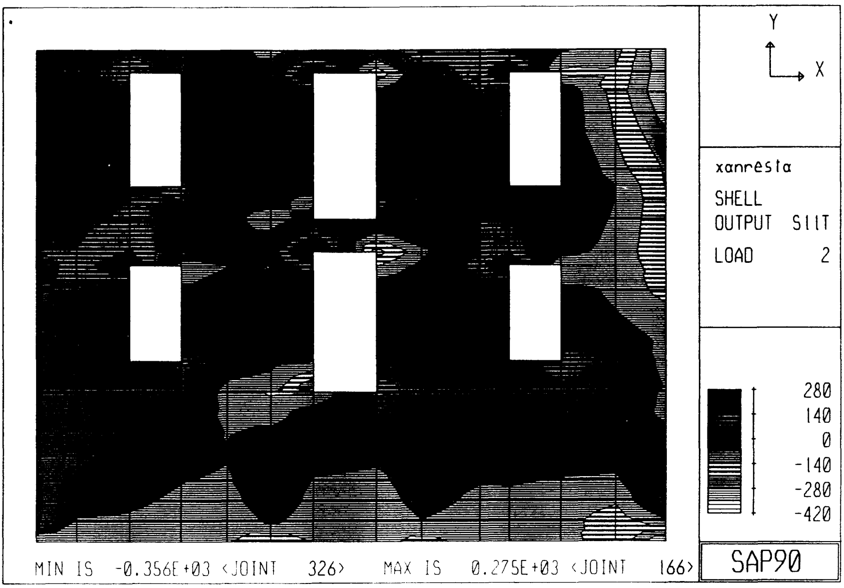

Fig. 6. Typical graphical output of the analysis results after interventions.

On Fig.6 a graphical output of the analysis after the proposed interventions is depicted. Comparison of Figures 4 and 6 shows the significant decrease of the stress level.

For the re-application of the failure criterion of step 5, the new analysis results are used. On Fig.7, the failed points of the facade wall 1, (taking now into account the proposed interventions), is presented.

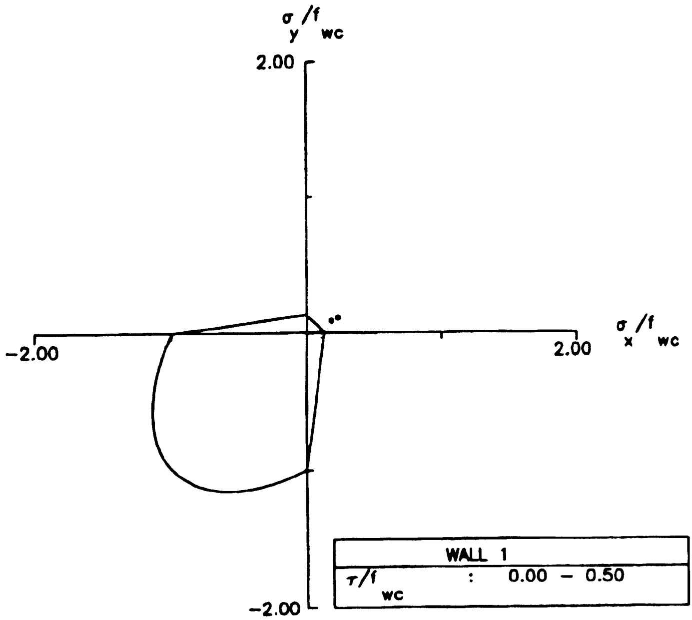

Fig. 7. Example of failure results after interventions.

Table III. Failure statistics for the whole structure after interventions.

STATISTICS FOR WHOLE STRUCTURE	
TOTAL NUMBER OF JOINTS......................	2788
TOTAL NUMBER OF FAILED JOINTS...............	36
TOTAL NUMBER OF FAILED JOINTS UNDER BIAXIAL COMPRESSIVE STRESSES....	1
TOTAL NUMBER OF FAILED JOINTS UNDER COMPR/TENS STRESSES............	16
TOTAL NUMBER OF FAILED JOINTS UNDER BIAXIAL TENSILE STRESSES........	19
TOTAL........	36

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.

Finally, on the following Table IV, the failure statistics for two loading cases (enumerated as loadings 2,3), including earthquake action, are presented, on a comparative basis, before and after interventions.

Table IV. Failure statistics for the whole structure before and after interventions.

Loading case	failed points before	failed points after
2	818/2788 (29.3%)	36/2788 (1.3%)
3	797/2788 (28.6%)	64/2788 (2.3%)

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