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EARTHQUAKE RESISTANT DESIGN OF MASONRY STRUCTURAL SYSTEMS

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Abstract. *Masonry structures are complicated structures and there is, currently, a lack of knowledge and information concerning the behaviour of their structural system under seismic loading. Successful modeling of a masonry structure is a prerequisite for a reliable earthquake resistant design. However, modeling a real structure to a robust quantitative (mathematical) representation is a very difficult and complicated task. This paper is presenting a contribution toward a solution of the problem. A new methodology for earthquake resistant design of masonry structural systems, either before or after their repair and/or strengthening is presented. The whole process is illustrated using the case study of a typical 4-storey masonry structure of the city of Patras in Greece.*

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 [1]. 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 behaviour 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 [2] the Venice Charter 1964 [3], 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 [4][5][6]. 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 [7]. According to Zienkiewicz et. al [8] the computation of singular points on failure surfaces may be avoided by a suitable choice of a continuous surface, which usually can represent, with a good degree of accuracy, the real condition.

Since reliable experimental data in the combined-stress state are rising rapidly [9][10][11], 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 [12] and Prager [13] the failure surface for a stable material must be convex. This, in mathematical terms, is valid if the total Gaussian curvature K of the failure surface is positive.

As can be concluded, various researchers have been working on the earthquake resistant design of masonry structural systems and especially determining a strength criterion, but there is still a lot ongoing research on that field.

2 EARTHQUAKE RESISTANT DESIGN OF MASONRY STRUCTURAL SYSTEMS

2.1 Methodology

The proposed methodology includes six 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.

(1) 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. Among them the formulae [14]:

$$f_{wc} = \xi\left[\left(\tfrac{2}{3}\sqrt{f_{bc}} - a\right) + \beta f_{mc}\right] \qquad f_{wt} = \frac{2}{3} f_{mt} \tag{1}$$

(compressive strength) (tensile strength)

are combining all parameters affecting the value of f_w. These parameters are described elsewhere in detail [14].

(2) Structural model
A 3-D finite element model seems to be generally the most suitable for the analysis.

(3) 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, is used, following the general rules provided by codes [18][19]. Earthquake has to be considered along all unfavorable directions for the building.

(4) 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.

(5) Failure criterion
A failure criterion must be established for the definition of the failed regions of the structure. Taking into account the conclusions of step 1 concerning materials' characteristics, such a criterion is proposed, and will be used as an input to carry out the analysis.

(6) 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 extend 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 [14].

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.

2.2 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 [15][16].

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

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. 2 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 $f_{wc}^{90°}$. This model was proposed by Asteris et al. [17].
Eq. 2 can thus take the following form:

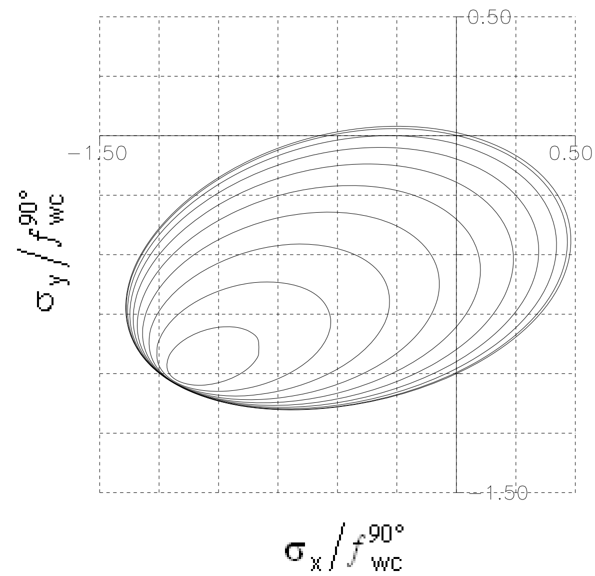


Figure 1: Non-Dimensional Failure Surface of Masonry in Normal Stress Terms
($\tau / f_{wc}^{90°}$ =0.00 up to 0.45 by step=0.05) [17]

$$F = 17.15\left(\frac{\sigma_x}{f_{wc}^{90°}}\right) + 74.57\left(\frac{\sigma_y}{f_{wc}^{90°}}\right) + 32.71\left(\frac{\sigma_x}{f_{wc}^{90°}}\right)^2 + 75.34\left(\frac{\sigma_y}{f_{wc}^{90°}}\right)^2 + 356.74\left(\frac{\tau}{f_{wc}^{90°}}\right)^2 - 17.12\left(\frac{\sigma_x}{f_{wc}^{90°}}\right)\left(\frac{\sigma_y}{f_{wc}^{90°}}\right) + \\ + 4.13\left(\frac{\sigma_x}{f_{wc}^{90°}}\right)^2\left(\frac{\sigma_y}{f_{wc}^{90°}}\right) + 1.35\left(\frac{\sigma_x}{f_{wc}^{90°}}\right)\left(\frac{\sigma_y}{f_{wc}^{90°}}\right)^2 + 122.46\left(\frac{\sigma_x}{f_{wc}^{90°}}\right)\left(\frac{\tau}{f_{wc}^{90°}}\right)^2 + 202.20\left(\frac{\sigma_y}{f_{wc}^{90°}}\right)\left(\frac{\tau}{f_{wc}^{90°}}\right)^2 = 1 \quad (3)$$

Fig. 1 depicts the contour map of Eq. 3, 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).

The resulting F value denotes the type of failure (Table 1), e.g.:
F<1 no failure of the element (type 1)
F≥1 failure of the element (type 2 to 5)

No	Type of masonry failure		Failure criterion
1	No failure		$F < 1$
2	Failure under biaxial tension/tension		$F \geq 1$ & $\sigma_x > 0$ & $\sigma_y > 0$
3	Failure under biaxial tension/compression		$F \geq 1$ & $\sigma_x > 0$ & $\sigma_y < 0$
4	Failure under biaxial compression/tension		$F \geq 1$ & $\sigma_x < 0$ & $\sigma_y > 0$
5	Failure under biaxial compression/compression		$F \geq 1$ & $\sigma_x < 0$ & $\sigma_y < 0$

Table 1: Non-dimensional masonry failure criterion under biaxial stress state

3 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. 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. 2.

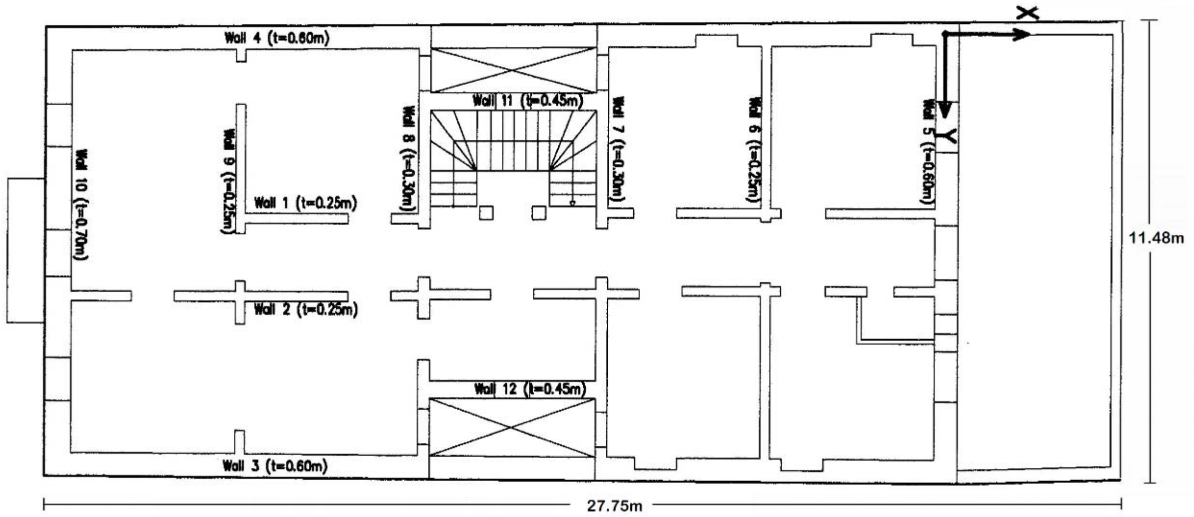


Figure 2: Typical plan view of the examined building

The new methodology for earthquake resistant design of masonry structural systems either before, or after their repair and/or strengthening is presented, by a short description of all steps.

1. 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 2 have been used for the analysis. Taking into account these and using semi-empirical expressions [14], the values of masonry compressive and tensile strength, have been calculated.

Material	Strength		Elastic Modulus	Poisson ratio
	Compressive	Tensile		
porous stone (f_{bc})	10 MPa	-	-	-
mortar (f_{mc} & f_{mt})	0.75 MPa	0.15 MPa	-	-
masonry (f_{wc} & f_{wt})	1.13 MPa	0.20 MPa	1130 MPa	0.3

Table 2: Mechanical characteristics of all materials used

2. For the simulation of the structural characteristics, a 3-D finite element model was built, using the Sofistik design software package (Fig. 3). All masonry walls were model using a 4-noded shell element. About 7800 element were needed to model the structure. For the determination of the strains in each element, six degrees of freedom (6 DoF) were con-

sidered. This refers to the motion of a rigid body in three-dimensional space; translation in three perpendicular axes combined with rotation about three perpendicular axes.

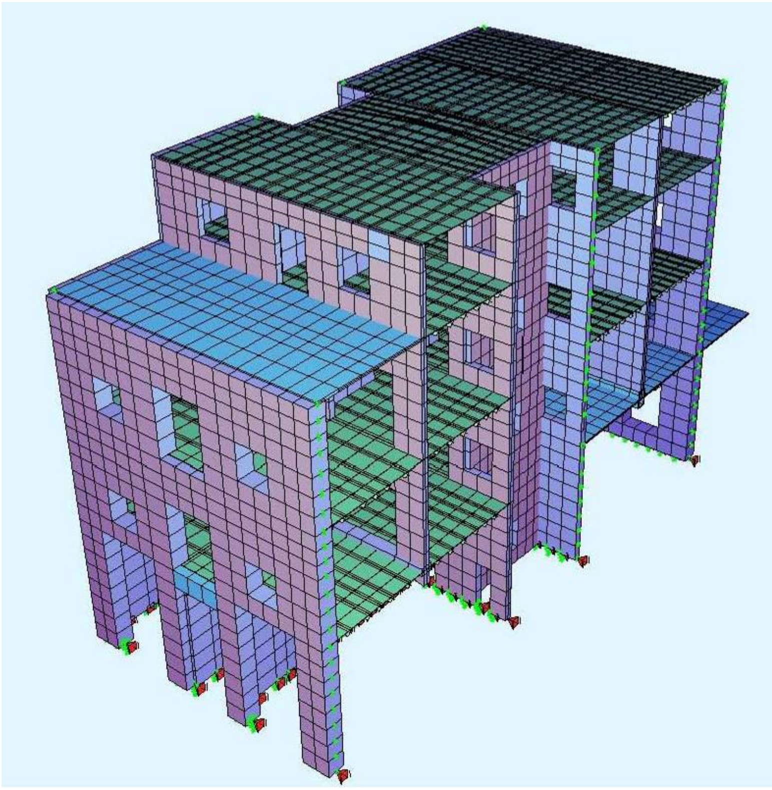


Figure 3: The 3-D FEM model of the building

3. Nominal values of dead and live loads were specified in the Greek Loading Codes [18], which are still in effect today. The seismic loads were also specified in the Greek Earthquake Code [19].

(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: 1^{st} storey Live load = 3,5 kN/m^2
LC3: 2^{nd} storey Live load = 3,5 kN/m^2
LC4: 3^{rd} 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.

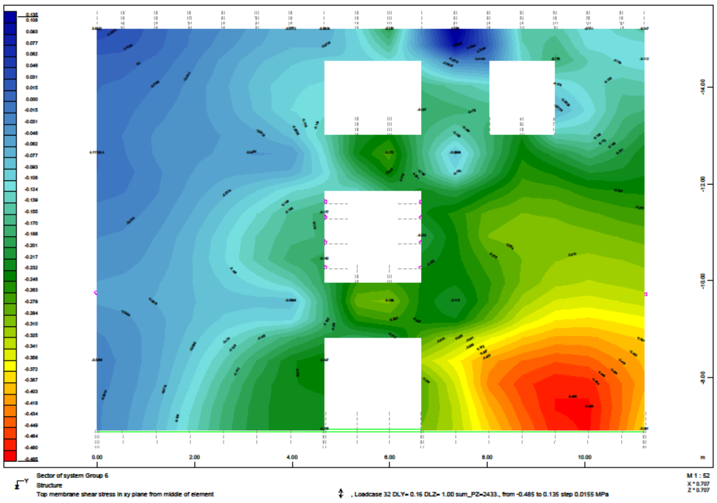
Based on the different loads the following combination actions have been used.

Combination with 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).
Y-direction is parallel to the front view of the building (transverse direction).

4. 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. 4 for the biaxial stresses σ_x and σ_y and the shear stress τ_{xy}.



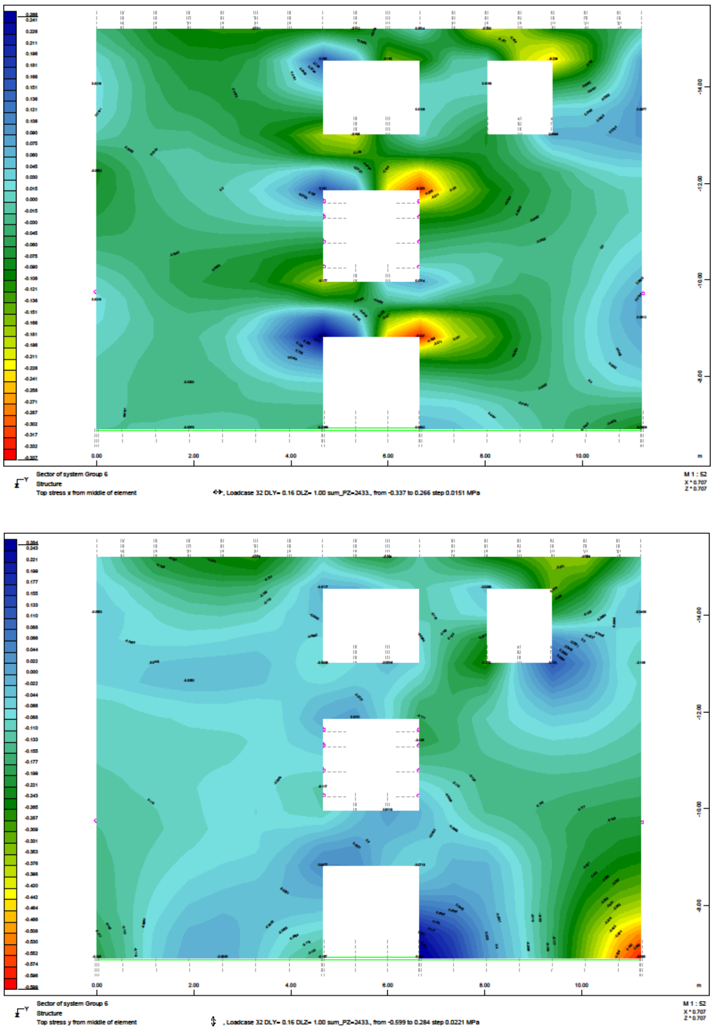


Figure 4: Typical graphical output for biaxial stresses σ_x and σ_y and shear stress τ_{xy} respectively before interventions

5. A spreadsheet has been established for the failure check of each element. 1st column denotes the number of element; 2nd – 9th columns are the internal forces and stress; 10th – 12th columns are the biaxial stresses σ_x and σ_y, shear stress τ_{xy}; last two columns show the application of the failure criterion proposed by Asteris et. al [17]. This spreadsheet 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. This is shown in Table 3 for a typical masonry wall (Wall 6).

All the different types of failures have been inserted back to the Sofistik model and a corresponding illustration of the walls is obtained (Fig. 6). 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.

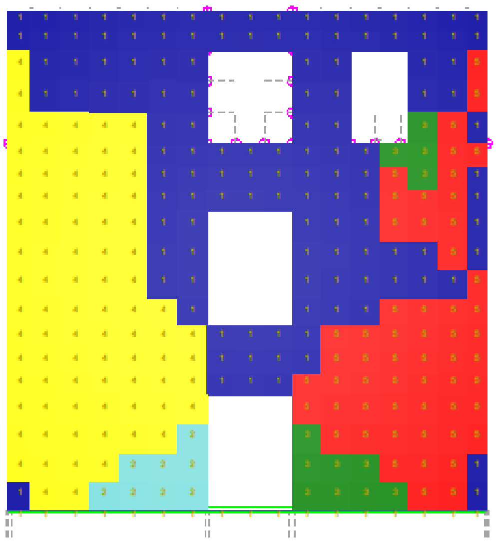


Figure 6: Illustration of failed elements and type of failure
for a typical masonry wall before interventions (Wall 6)

Type of Failure

	1	2	3	4	5
LC31	42.6	3.9	7.0	3.9	42.6
LC32	58.5	5.6	6.7	2.8	26.4
LC33	55.6	4.9	6.0	2.8	30.6
LC41	39.44	5.28	7.39	3.52	44.4
LC42	46.5	4.9	6.7	3.2	38.7
LC43	47.5	4.9	6.3	5.3	35.9

Table 3: Percentage (%) of failed elements and type of failure
for a typical masonry wall (Wall 6)

6. 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.

4 CONCLUSIONS

In this paper, the earthquake resistant design of masonry structural system is discussed, in terms of finding an appropriate failure criterion and then applying an efficient strengthening strategy. A review of numerous failure criteria from various researchers are discussed. A non-dimensional anisotropic masonry failure criterion under biaxial stress state is presented and used in an real masonry structure. The results of the analysis have shown that this criterion is applied successfully and useful conclusions have been provided for the earthquake resistant of the structure. The graphical determination of all failed areas for each wall is important. Especially the categorization of those areas according to the type and intensity of stresses is particularly useful, because the designer can decide on the length and type of intervention. The reason is that some areas may fail due to biaxial compression and others due to biaxial tension and thus different treatment would be need for each case. The case study has shown that the decisions for strengthening each masonry wall separately, based on the failure criterion applied, is successful when comparing the two walls before and after intervention.

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