Syrmakezis, C.A., **Asteris, P.G.**, & Sophocleous, A.A. (1997). Earthquake resistant design of masonry tower structures. Proceedings, Fifth International Conference on Structural Studies of Historical Buildings, *STREMA 97,* Vol. 3, pp. 377-386, San Sebastian, Spain, 25-27 June, 1997.

Earthquake resistant design of masonry tower structures

C.A. Syrmakezis, P. G. Asteris , A. A. Sophocleous
National Technical University of Athens, Institute of Structural Analysis and Aseismic Research, Zografou Campus,
GR-15773, Athens, Greece
EMail: isaarsyr@central.ntua.gr

Abstract

The paper refers to the problem of protection of masonry tower structures against earthquake hazard. This specific category includes several important types of historical structures: chimneys, bell towers, lighthouses, minarets etc. For the complicated response of these structures, there is a lack of knowledge and information concerning their behaviour under seismic actions. In this paper the peculiarities of the response of such structures are investigated. Namely the compression-nontension response and the load distribution along the height of these structures, modifying both significantly the response compared to response of regular building structures, is examined. The structures under consideration present a high degree of vulnerability against earthquakes and usually strengthening is required for their protection. In this paper methods of strengthening and an appropriate analysis and design process, taking into account all these parameters, are proposed. The whole process is illustrated using the case-study of one typical masonry tower structure, the bell tower of Agia Paraskevi in Metsovo, Greece.

1 Introduction

Masonry structures are in general complicated structures and there is an unfortunate lack of knowledge and information concerning the behaviour of their structural system under seismic loads. For a successful protection of the masonry tower structures (including several important types of historical structures: chimneys, bell towers, lighthouses, minarets etc.), several specific questions have to be answered, beyond the general difficulties and peculiarities mentioned above.

Recently a general methodology concerning the structural analysis and redesign of masonry structures has been presented[4, 5]. In this paper a specified version of this methodology is applied for the tower masonry structures. Namely, two analysis methods as well as their specific features for the type of structures considered are proposed: a) Limit State analysis and b) Finite element analysis.

2 Limit State analysis

For the Limit State Analysis, non-linear stress/strain relationships have to be defined. For each one of the basic materials (masonry, concrete, steel) an analytical polynomial expression, with the following general form is used:

$$\sigma = \alpha \frac{f_k}{\gamma_m} \sum_{i=1}^{r} c_i \varepsilon^i \tag{1}$$

where:

- α : material constant, to be chosen according the codes.
- f_k : characteristic strength of the material.
- γ_m : partial safety factor.
- c_i : coefficients for analytical polynomial expression of the material stress/strain relationship.
- r : number of terms to be used.

Following equation 1 distribution of normal stresses σ_{zz} in compression area of the cross-section (Figure 1), is:

$$\sigma_{zz}(x,y) = \alpha \frac{f_k}{\gamma_m} \sum_{i=1}^{r} c_i\, \varepsilon^i_{zz}(x,y) = \alpha \frac{f_k}{\gamma_m} \sum_{i=1}^{r} c_i \left[\frac{x}{x_c} \varepsilon_c \right]^i \tag{2}$$

For various values of parameters ε_c and x_c (strain distributions) the corresponding stress distributions give a total compressive force N and a bending moment M of the form:

$$N = \alpha \frac{f_k}{\gamma_m} \sum_{i=1}^{r} c_i \left[\frac{\varepsilon_c}{x_c} \right]^i I_i \tag{3}$$

$$M + N\left(x_c - b_0\right) = \alpha \frac{f_k}{\gamma_m} \sum_{i=1}^{r} c_i \left[\frac{\varepsilon_c}{x_c} \right]^i I_{i+1} \tag{4}$$

where I_i is the i-rank moment of inertia of the compressed area as per n-n axis.

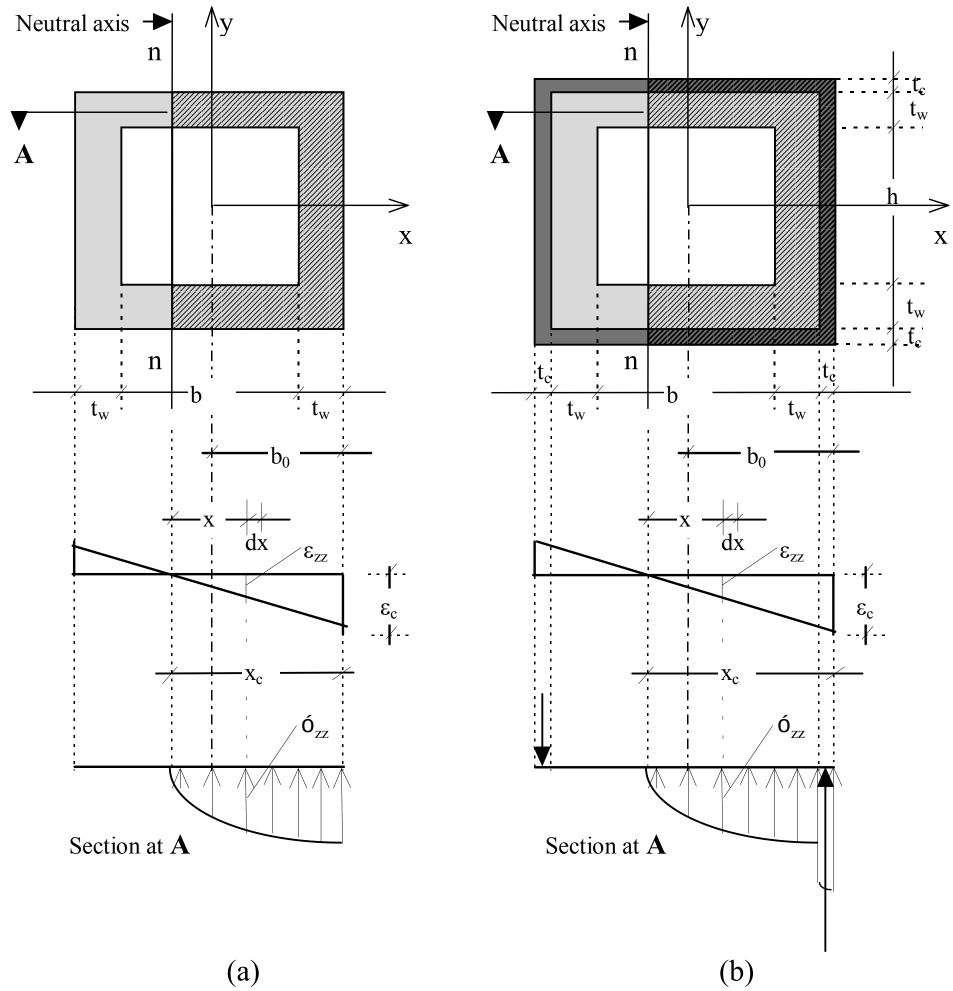


Figure 1: Strain and stress distributions for: a) plain section and b) jacketed section (external reinforced concrete jacket).

To define a cross-section design strength resistance (design axial force N_d and bending moment M_d), the development of its interaction diagrams is needed. It can be evaluated, using equations 3 and 4.

Limit State Analysis can be mainly used for the redesign of an existing masonry tower structure to be strengthened using modern materials (e.g. reinforced concrete jacket).

For the investigation of the existing status of the masonry structure, before any intervention, the method can apply on condition that overturning stability is fulfilled. It is to be noted that overturning stability, very often, does not exist for such structures, especially in earthquake prone areas, where high seismicity provides high values of horizontal seismic forces, leading to overturning collapse.

3 Finite Element Elastic Analysis

The finite element elastic analysis can also be used. Through this method one can: a) verify the results obtained from the previous method, and b) investigate the systematic collapse pattern, using specific failure criteria. An appropriate model is required, modeling with sufficient accuracy the real structure, as far as its geometry and mass-stiffness distribution is concerned.

For the failure analysis, a special computer programme called "FAILURE" has been developed in the Institute of Structural Analysis and Aseismic Research of NTUA[4, 5]. 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. This programme gives also for each one of the walls or for the whole structure and for each loading case, statistics for the number of failed points and the type of failure. This information is very useful as it provides a general view for the probable damage level and the main type of damages of the structure, allowing the conclusions, concerning the necessary measures to be applied.

4 Case study

The case study refers to a typical masonry tower structure, the bell tower of Agia Paraskevi in Metsovo, Greece. The section of the tower is rectangular 4×4 m at the base (Figure 2), and is slightly reduced along the 23 meters of its height.

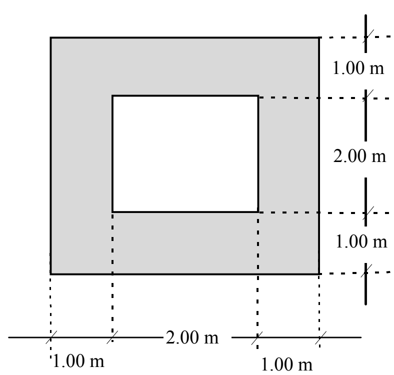


Figure 2: The section at the base of the bell tower of Agia Paraskevi in Metsovo, Greece.

The mechanical characteristics of the masonry were evaluated after experimental investigation of the materials as following:

- mean compressive strength: f_{wc}=4.625 MPa
- mean tensile strength: f_{wt}=0.300 MPa

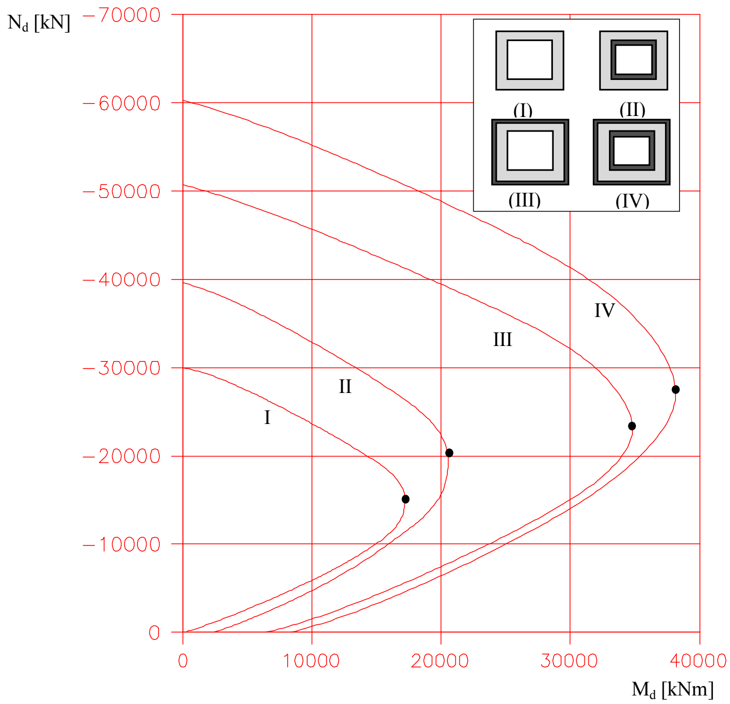


Figure 3: Axial force/bending moment interaction diagrams for plain and jacketed (reinforced concrete jacket inside or/and outside) section the tower.

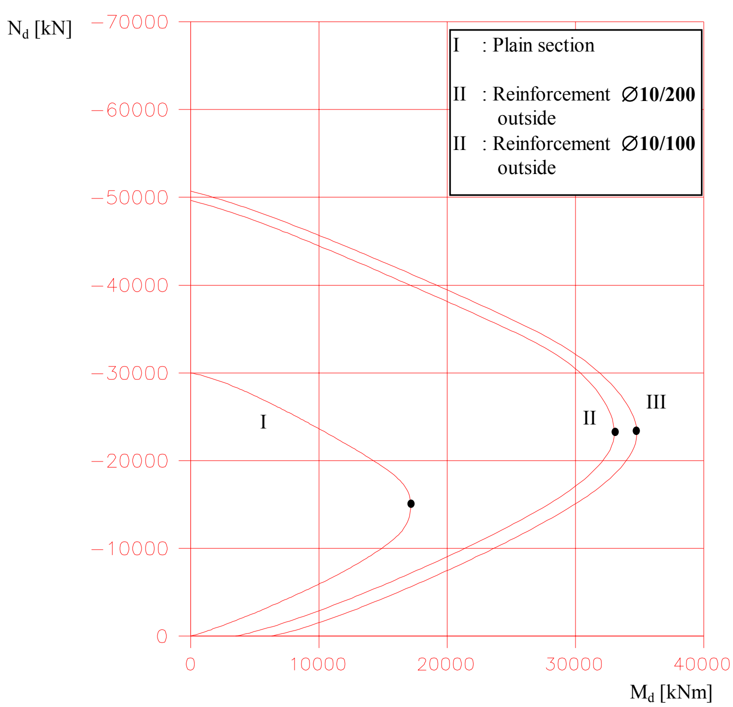


Figure 4: Axial force/bending moment interaction diagrams for jacketed (reinforced concrete jacket outside) section of the tower.

On Figure 3, four curves N_d-M_d are given, representing:

Curve	I	:	plain section.
Curve	II	:	section jacketed with internal reinforced concrete jacket.
Curve	III	:	section jacketed with external reinforced concrete jacket.
Curve	IV	:	section jacketed with both side reinforced concrete jacket.

On each curve, a maximum M_d value point can be marked. This point corresponds to a "balanced" failure, where steel and masonry both, reach their limiting values simultaneously. The line linking all these points is an almost straight line. Above this line each combination of N_d-M_d corresponds to failure in which the masonry reaches its limit value, while steel remains below the yield level (compression failure area). Below the line, pairs N_d-M_d correspond to failure in which steel also has yielded (tensile failure area).

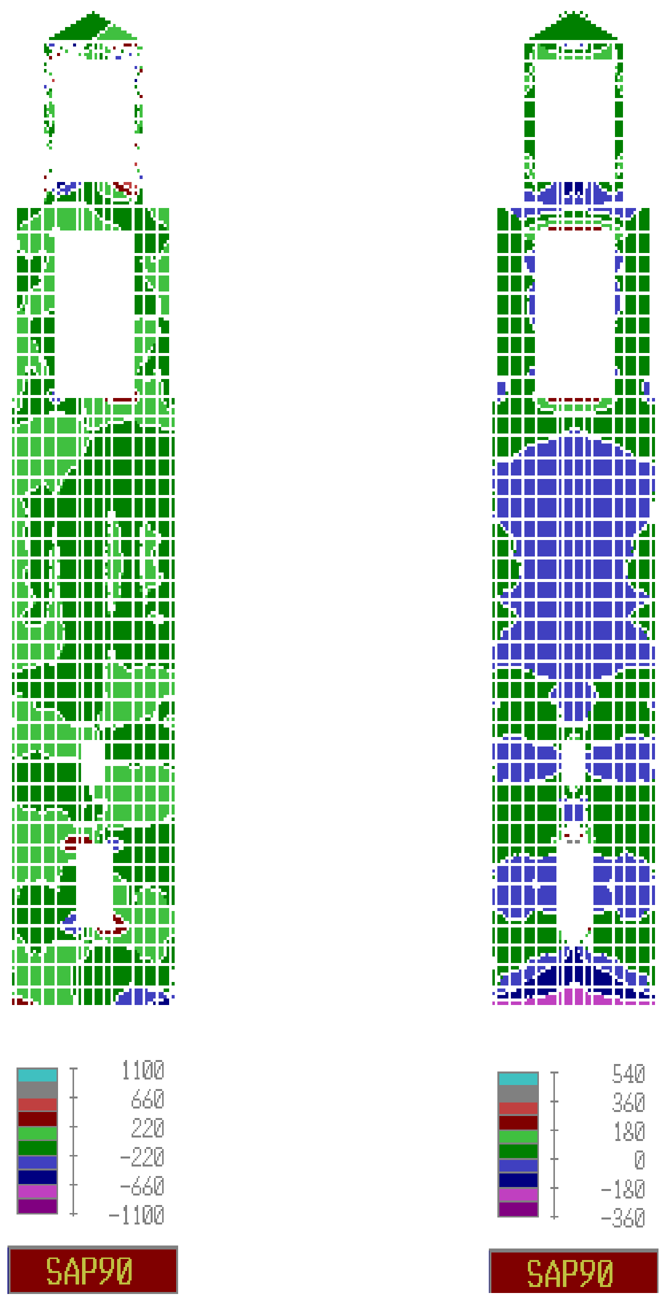
For the sequence of cases I, II, III, IV of curves, it is observed that the existence of reinforced concrete jacket significantly increases the design strength of a plain cross-section. External jacket is much more efficient than the internal one. Nevertheless, external jacketing is very often forbidden due to architectural/archaeological restrictions.

On figure 4, for the case of external jacket, the comparative analysis results among plain, lightly reinforced and heavily reinforced section are presented. It is to be mentioned that the use of reinforced concrete jacket even lightly reinforced, increases drastically the design strength of the section. Such mild interventions are highly desired, as they allow simultaneous satisfaction of all requirements, beyond the structural ones.

For the FEM analysis, the discretisation of the structure has been developed using 3 and 4 node elements, combining membrane and plate bending behaviour. Program SAP90 has been used.

The whole analysis has been elaborated twice. In the first case the horizontal loading was distributed along the height of the tower according simplified requirements of the codes, in an inverse triangular shape. In the second case the Response Spectrum Analysis Method (RSAM) has been used, in accordance with the EC 8 design spectrum (soil B). In both cases a failure analysis has been also performed for the estimation of the damage distribution on the structure.

On figures 5a, 5b, a typical example of stress distribution is given for the two cases of the loading considered. Namely, distributions of horizontal normal stresses on the facade wall of the tower are presented, for combination of dead, live and earthquake loads. It is to be noted that for the inverse triangular loading, stress level is higher, compared to the RSAM results. This



(a) (b)

Figure 5: Analysis results for the existing status of the masonry tower. a) Triangular distribution of loading along its height and b) Response spectrum analysis method.

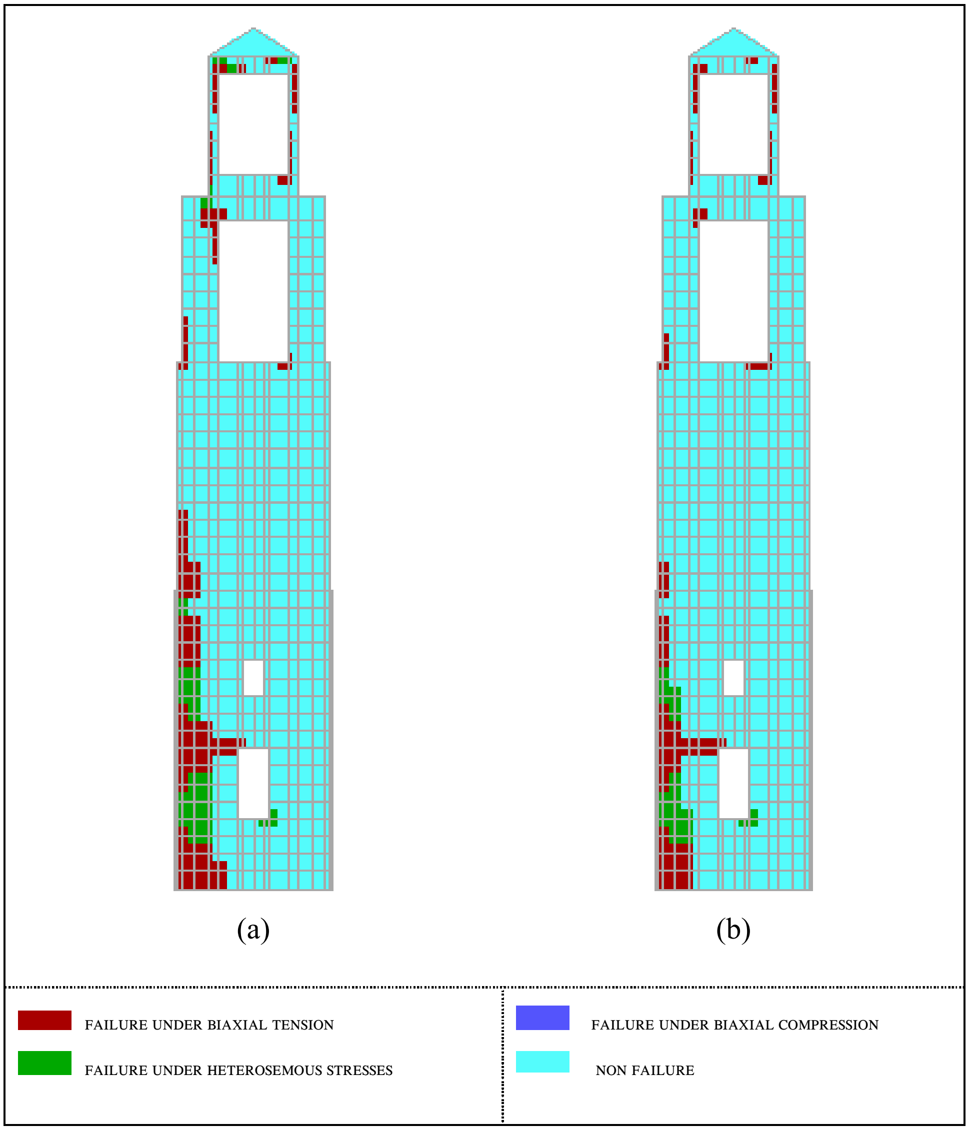


Figure 6: Failure results of the bell tower of Agia Paraskevi in Metsovo city, in Greece. a) Triangular distribution of loading along its height and b) Response spectrum analysis method.

difference is due to the fact that the second solution is closer to the real response of the structure, while the first is less accurate and much more conservative. Consequently, response spectrum analysis method is more suitable for this type of masonry tower structures, with a significant ratio of height to length.

Similar remarks are valid for the total horizontal displacements at the top of the structure: for the first case of loading it has been calculated equal to 4.30 cm, while for the second case equal to 0.62 cm.

Failure analysis results, for the same facade wall, are shown on figure 6, for the two aforementioned cases of loading. It is to be noted that for both cases, the main source of damage is biaxial tension. Aforementioned remarks are still valid for this couple of figures: case b, although less conservative, should be considered as more reliable.

5 Conclusions

In this paper a methodology for the analysis and redesign of tower masonry structures is presented. Namely, two analysis methods as well as their specific features for the type of structures considered are proposed and compared: a) Limit State analysis and b) Finite element analysis.

Limit State Analysis can be mainly used for the redesign of an existing masonry tower structure to be strengthened using modern materials (e.g. reinforced concrete jacket). For the investigation of the existing status of the masonry structure, before any intervention, the method can apply on condition that overturning stability is fulfilled.

On the other hand the finite element method combined with the failure analysis can be used both for the analysis of the present status of the structure and its redesign. This methodology is much more integrated and precise giving full knowledge of the response of the whole structure. So the designer has the advantage to decide for the necessary strengthening measures to be taken in a more safe and precise way enabling him to reduce both the extent and the degree of interventions.

It has been observed that the existence of reinforced concrete jacket, even lightly reinforced, significantly increases the design strength of a plain cross-section, and that external jacket is much more efficient than the internal one. Also it can be stated that the response spectrum analysis method is more suitable for this type of masonry tower structures, with a geometry of significant ratio of height to length.

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