*Protection of Historical Buildings, PROHITECH 09 – Mazzolani (ed)*
*© 2009 Taylor & Francis Group, London, ISBN 978-0-415-55803-7*

# Strengthening of masonry using metal reinforcement: A parametric numerical investigation

M.E. Stavroulaki
*Applied Mechanics Laboratory, Sciences Department, Technical University of Crete, Greece*

G. Bartoli & M. Betti
*Department of Civil and Environmental Engineering, University of Florence, Italy*

G.E. Stavrolakis
*Department of Production Engineering and Management, Technical University of Crete, Greece*

ABSTRACT: Metal rods and prestressed cables can be used for the strengthening of masonry structures and monuments. These techniques are partially reversible and introduce minimal changes in the characteristics and the structural system. Therefore they are practical and economical alternatives to more traditional strengthening methods. The research to date has mainly focused on component behavior and only few results about the effectiveness of post-tensioning retrofits on overall structure response has been published. In this paper a number of numerical models and extensive parametric investigation was done in order to study the effectiveness of the previously mentioned technique, by analyzing the response of a masonry tower as a case study. Both static and dynamic analyses considering base excitation loads were performed. The effectiveness of the prestressing depends on the location of the cables, the material and the geometry of the structure. Prestressing must be accompanied with local reinforcement in order to reduce excessive deformations and damages.

## 1 INTRODUCTION

Today, the finite element method is widely adopted for sophisticated simulations of the structural behaviour. Significant information can be obtained from advanced numerical analysis, that helps to understanding existing damages on a structure and to optimizing the design of all the strengthenings. A clear understanding of the structural behaviour and reliable strengthening, based on sophisticated tools of structural analysis, can therefore reduce the extent of the remedial measures in the restoration of ancient structures (Leftheris et al. 2006).

Post-tensioning of masonry structures is one of the effective ways for the reinforcement of old buildings, both of monumental (churches, town halls. . .) and common type (family houses. . .). Post-tensioning (when the prestressing tendons tightly enlace the existing building) is suitable for facing several kinds of failures like: horizontal displacement of supports (leadings to possible cracks and overturning of the external walls), presence of extensive cracks, partial collapse or buckling of walls, failures in the corners and crossings. Static support and strengthening of badly damaged constructions by means of steel post-tensioning tendons is a fast, economically profitable and powerful alternative. Numerical analyses should help to describe and understood the behaviour of masonry and help the project engineers to design the post-tensioned masonry (Čajka 2008).

Historical masonry buildings exhibiting a prevailing vertical character, such as towers and bell-towers, represent a structural typology with several common aspects: they are slender tall structures, which mainly have to support their own weight. These characteristics, together with all damages induced by several different factors during the years, make them particularly vulnerable with respect to (even small) base movements, such as those provoked by small seismic actions or base settlements; the crack pattern which is inevitably present on these structures appears more or less in a known typical way.

In the present paper the strengthening of a masonry tower by using metal reinforcement so that the stresses under static and dynamic loads will not exceed the admissible tensile and compressive strength of the masonry, was studied. The aim of the research is to select the application points and the magnitude of the pretension forces of the cables in order to optimize the response of the structure under dynamic loads. As a case study a historic masonry tower was analyzed by finite element analysis and different cases of cables locations were examined. Their influence was indicated by the reduction of plastic strains and tensile stresses. During this study the appearance of local damages due to the prestressing was also shown and possible measures against it were discussed as well.

## 2 RESTORATION OF BUILDINGS WITH PRESTRESS CABLES

In order to stabilize an existing deformed structure and to stop its further unhinging development, prestress cables are used. Prestress reinforcements do not cancel the cause of deformation, but instead they overcome the destructive tensile stresses by applying initial stresses. The method permits the monitoring of its effectiveness (e.g. prestress intensity) during its use and provides the means to adjust it. Nevertheless, relatively high technical expertise is necessary for the installation and follow up adjustments.

The selection of the force applied in the prestress cables should be made carefully in order to be compatible with the permissible masonry compressive stresses, applied only in anchorages where the wall can withstand the high local stresses. As a rule, local protective measures are required at the anchorage regions. The introduction of prestress cables that tie the building affects the state of stress locally as well as the state of stress of the whole building. This reinforcement method increases the strength of the building without any substantial increase in both the stiffness and the mass of the structure (Betti & Vignoli 2008).

By tensioning the cables, a compressive stress field is induced in the wall that in turn reduces the tensile stresses. In this way destructive effects, such as the appearance of tensile cracks in the theoretically no-tension masonry structure, can be avoided and methods of linear elastic structural analysis can be employed for the strength and seismic evaluation of the reinforced structure. For masonry facades, prestressing cables are usually placed horizontally at the height of the floors or the roof, vertically at the edges of the building and at places where crossing of walls exist or in an x-shape formation at the aprons.

The technique of externally placed tendons is widely used for the reinforcement of existing structures, because of several advantages: minimum disturbance of the structure and its users, negligible change of mass, stiffness and dynamic characteristics of the existing structure, minimum loss of prestressing forces due to friction, and the use of high-quality materials with known properties (Figueiras et al. 1994, Leftheris et al. 1993, Leftheris et al. 2006).

## 3 CASE STUDY

### 3.1 *Geometry*

The above discussed methodology of analysis is explained with reference to a specific case study: the medieval "*Torre Grossa*" (see Fig. 1) (Bartoli et al. 2006). This building is a tall masonry tower, dated as back as thirteenth century and is located in Piazza Duono (Square of the Cathedral). It is the tallest and most mighty of the towers preserved in the town of San Gimignano (Italy). The cross section is a square one measuring 9.5 × 9.5 meters, with an overall height of about 60 m. The walls are of variable thickness, between 2.6 m and 1.6 m. The sustaining walls are infilled ones, with the external face made by stone masonry (20–30 cm thickness), and the internal layer constituted by brick masonry (25 cm thickness), with mortar layers nearly a centimeter thick. The internal filling is composed of heterogeneous material (remainders of brick tied by a poor mortar). Up to the height of 20 m the tower is incorporated in a previous dated building, "*Palazzo Comunale*" (Town Hall). Floors have been constructed through masonry vaults, while in the upper part of the tower there is a concrete floor, connected to the bottom part of the tower by a steel stair.

Figure 1. The Torre Grossa in San Gimignano, Italy.

### 3.2 *Material model*

In case of an earthquake, the structure will be subjected to a series of cyclic horizontal actions, which will often cause high additional bending and shear stresses in structural walls, possibly exceeding the range of the elastic behaviour. The nonlinearity of the material can be accounted for by using nonlinear stress-strain relationships or constitutive equations. Thus, for the nonlinear analysis of the examined models, in addition to the elastic material constants (Young's modulus and Poisson's ratio), the yield stress and yield function must be determined. The inelastic (plastic) material behaviour is described by a stress-strain curve made up from two branches, the first one which corresponding to the elastic region of the material and the second one to the plastic region. The magnitude of the yield stress is generally obtained from an uniaxial test but since the stresses in a structure are usually multiaxial, a yield condition must be used for measurement of yielding of the multiaxial state of stress. The yield condition depends on all stress components, on shear components only, or on hydrostatic stresses.

From previous researches on this tower (Bartoli & Mennucci 2000, Bartoli et al. 2000) it is known that the type of masonry used in the tower (made up by three different layers: the external stone masonry, the in-fill material and the internal brick masonry) is often referred as a "multiple leaf" one: the overall mechanical behaviour of it is quite complex and it is not easily described by very simple models. Nonlinear behaviour of masonry was observed during flat jack tests on the wall external surface of the Torre Grossa, in San Gimignano. In order to obtain actual values of stress by means of such tests, an interpretation procedure was defined, demonstrating that results obtained with the hypothesis of linear elastic behaviour of masonry leads to an overestimation of about 20% with respect to the actual stress values (confirming other results from numerical simulations obtained by F.E. models). Starting from some experimental results, a numerical model was set up, after a preliminary research work on the determination of the characteristics of an "equivalent" stone to be used in the analysis (Bartoli & Spinelli 2003).

In the present study the general purpose finite element program MARC, was used in which several elastoplastic models can be used (MARC 2002). In particular the Mohr-Coulomb Parabolic criterion was used, which is a first two-parametric yield surface for the maximum compression and tension. The model is the first one that takes shearing into account. It should be noted that the criterion considers the maximum difference between the major and the minor principal stresses only, and does not consider the intermediate principal stress in the strength criterion. The Mohr-Coulomb strength criterion can be represented graphically, by Mohr's circle. Most of the classical engineering materials, including rock materials, somehow follow this rule in at least a portion of their shear failure envelope.

The material data and the materials models parameters are given in Tables 1 and 2. The material has been considered as homogeneous and isotropic, the numerical values have been chosen on the basis of compression tests performed on specimens and from literature data (Bartoli et al. 2006).

Table 1. Mechanical properties of materials.

| Material | Modulus of Elasticity [GPa] | Poisson ratio | Own weight [Kg/m$^3$] |
|---|---|---|---|
| External stone masonry | 11.0 | 0.15 | 2400 |
| Internal filling | 1.6 | 0.15 | 2000 |
| Internal brick masonry | 3.0 | 0.15 | 1800 |
| Steel | 209.7 | 0.30 | 7850 |
| Concrete | 2.74 | 0.20 | 2400 |

Table 2. Parameters of yield function of materials model.

| Material | Tensile strength [Pa] | Compression strength [Pa] |
|---|---|---|
| External stone masonry | 1.1 $10^6$ | 2.86 $10^5$ |
| Internal filling | 1.6 $10^5$ | 4.16 $10^4$ |
| Internal brick masonry | 3.0 $10^5$ | 7.80 $10^4$ |

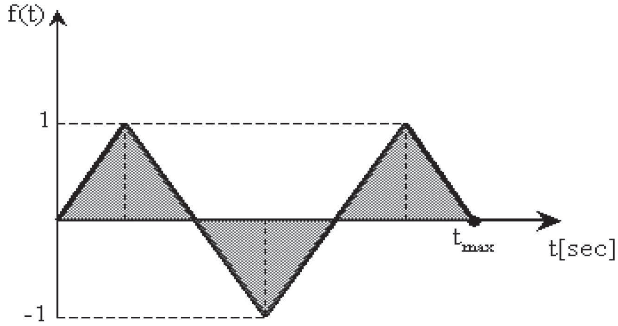


Figure 2. Displacement history of base excitation.

## 3.3 *Loads of the models*

In order to investigate the response of the structure under base excitation, time varied displacements are considered. The loading history is a sinusoidal function A horizontal sinusoidal displacement $f(t) = \sin(2\pi t/T)$ (Fig. 2), at the base of the wall and in the diagonal direction were applied, in addition to the weight of the structure. Maximum value of displacement is equal to 0.01 m in the two horizontal directions x and y.

## 3.4 *Finite element models*

The finite element method was used on a three-dimensional, solid model of the tower modeling. Solid hexahedral finite elements have been used for the masonry and truss elements for the steel cables with three degrees of freedom per node. At the beginning, the following three models, with different material models were examined:

- Model A: tower without any reinforcement (Fig. 3);
- Model B: tower with the first group of prestress cables located around the masonry (Fig. 4);

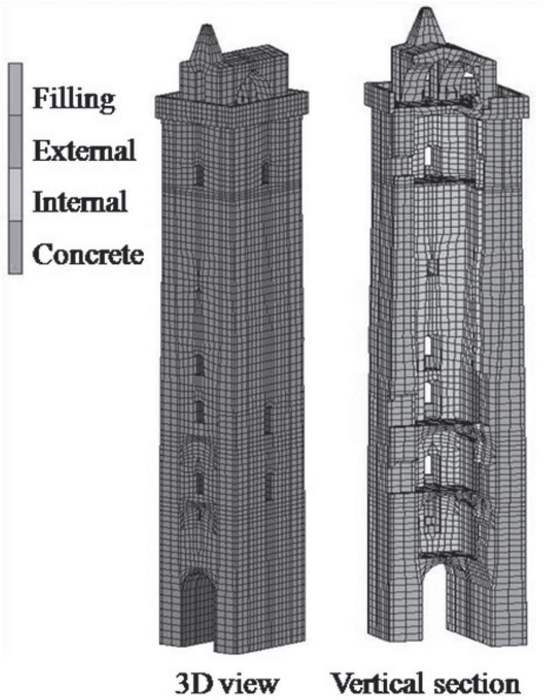


Figure 3. Finite element model of the tower.

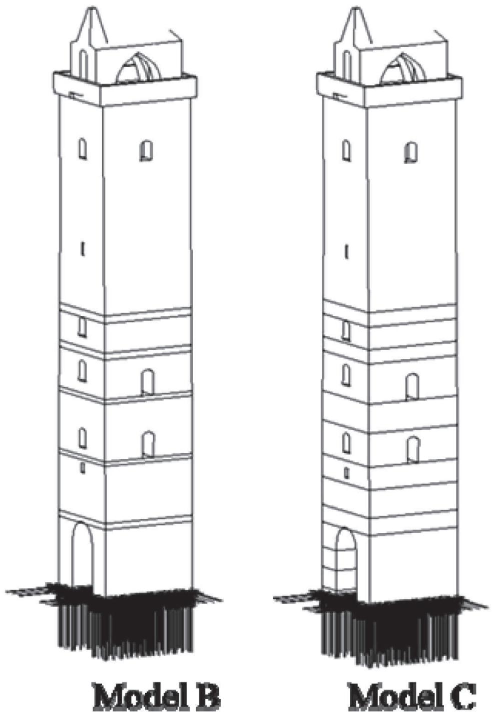


Figure 4. Location of prestress cables for Models B and C.

- Model C: Ttower with the second group of prestress cables located around the masonry (Fig. 4).

After the analysis, the need of applying additional local strengthening, especially at the lower first part of the model appeared. Therefore, two additional models were analyzed. They include new enhanced strength values of the internal filling material (which can be obtained by means of properly designed injections):

- Model D: tower with the second group of pre-stress cables (model C) but considering internal filling material at the lower part with almost double strength (like the internal brick masonry);
- Model E: tower with the second group of prestress cables (model C) but considering internal filling material at the lower part with high strength (like the internal brick masonry).

Table 3. Volume ($m^3$) of masonry with plastic strains.

| Material | Model A | Model B | Model C | Model D |
|---|---|---|---|---|
| External stone masonry | 956.12 | 952.83 | 948.06 | 947.45 |
| Internal filling | 1813.40 | 1814.7 | 1816.1 | 1817.7 |
| Internal brick masonry | 372.03 | 369.35 | 368.07 | 366.52 |
| Total | 3141.55 | 3136.88 | 3132.23 | 3131.67 |

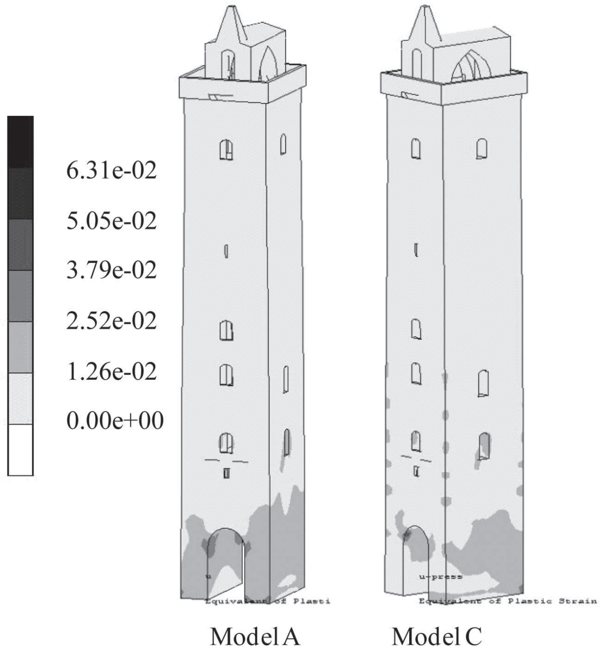


Figure 5. Contour plot of equivalent plastic strains.

## 4 RESULTS

When dealing with the elastoplastic material models, the estimation of the region with plastic strain is an indication of failure and crack development. In Table 3 the volume of the masonry with remaining plastic strain at the final step of the analysis, is given. The placement of prestress cables at the perimeter of the tower leads to reduction of the plastic strains in particular at the internal and external layers of the material. The same is shown in Figure 5 where the contour plots of the equivalent plastic strains for the models A and C are presented. Better response is presented in model C where the location of the cables are more normally distributed along the height of the tower.

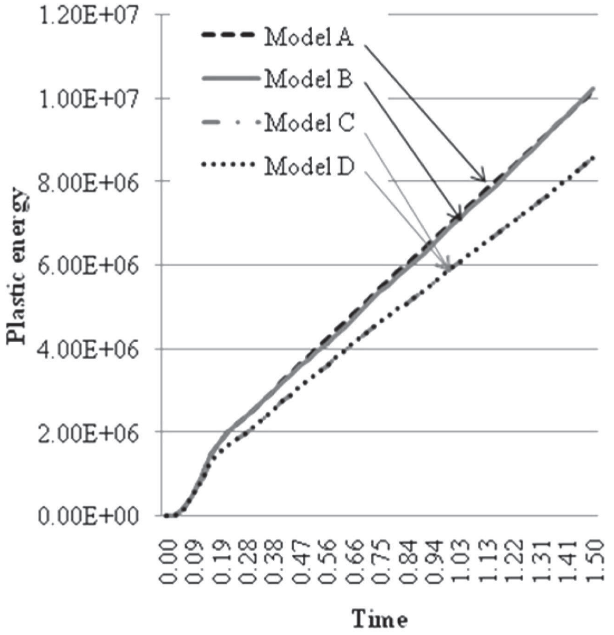


Figure 6. History of plastic energy.

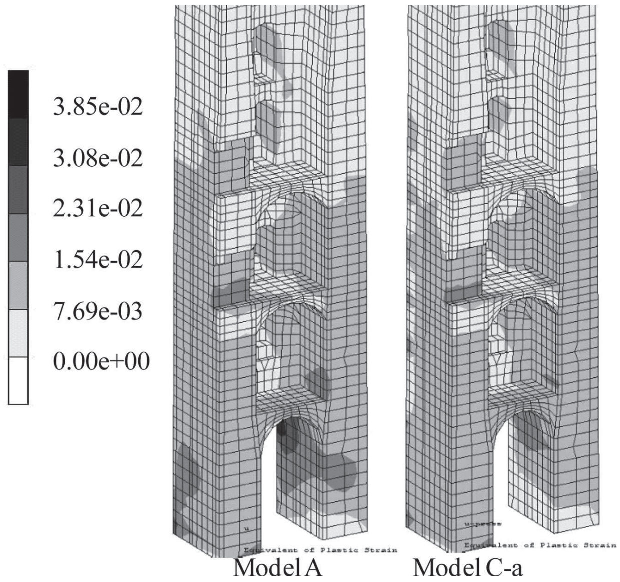


Figure 7. Contour plot of the equivalent plastic strains for a vertical section.

The reduction of the tensile stresses due to prestressing is not so high since topical increase of the compression stresses appears as well, and the development of tension stresses at other locations. For these topical problems, additional strengthening techniques can be applied in order to increase the strength and to give better protection to the structure, like the application of injections in order to increase the strength of the internal filling material. In the present analysis two more models were analysed considering higher strength of the filling material.

The influence of the prestress cables on the overall mechanical behaviour is presented by the history of the plastic energy of the examined models (Fig. 6). It must be noticed that the location of the cables is important since a non normal distribution (like in Model B) does not enhance the mechanical behaviour.

From the results obtained from the analysis, additional local strength problems were identified at the lower part of the tower due to the arch at the base and the adjacent building which leads to a non symmetric structural behaviour. Stress concentrations are developed around the openings and to the connections of the arches with the walls. The reinforcement of the filling material could relief the lower part of the structure. Another possibility is to increase the strength of the material at these areas. A modification to the Model C has been done by considering better material around the openings and at the inner face of the arch at the base floor of the tower (Model C-a). The results show a small reduction of the plastic strains as it is shown in Figure 7.

Another interesting comment can be done on the displacement results presented in Figure 8. For the initial placement of the prestressing cables, the displacements of the structure are higher. A better positioning of the prestressing cables, reinforcement of the material near the basement and possible local reinforcement (last model Ca) result to reduction of the displacements.

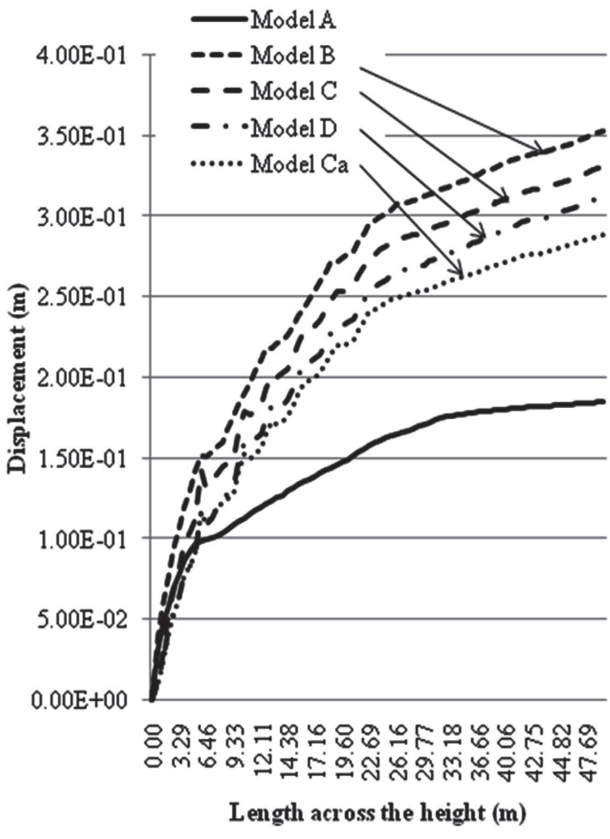


Figure 8. Displacement along the height.

From the few numerical results presented here, it is clear that a nonlinear finite element model can be used for the evaluation of the efficiency of several reinforcement schemes.

## 5 CONCLUSIONS

Prestressing reinforcement, accompanied with local enhancement of material properties and classical reinforcement can be useful for masonry structures. The shape of each structure requires an individual design of the reinforcement. Based on the experience and a trial-and-error procedure, supported by nonlinear finite element calculations, a case study has been presented in this paper. The model has been suitably parametrized, so that in the near future a structural optimization algorithm will be employed for the automatic calculation of optimal placement and dimensioning of the prestressing cables.

## REFERENCES


Bartoli G., Mennucci A. 2000. Progetto S. Gimignano, la Torre Grossa: indagini conoscitive e diagnostica. *Atti del primo convegno su la diagnostica per la salvaguardia dei Beni Culturali, S. Gimignano, Teatro Comunale, 6–7 Dicembre 1999 (in italian)*.

Bartoli G., Chiostrini S., Innocenti S. 2000. Problems related to the analysis of experimental data from flat-jack tests. *Convegno CICOP "FIRENZE2000", 5th International Congress on Restoration of Architectural Heritage, Settembre 2000.*

Bartoli G., Spinelli P. 2003. The 'Torre Grossa' in San Gimignano: Experimental and numerical analysis. *Proceedings of the First International Congress on Construction History, ed. S. Huerta, Madrid: I. Juan de Herrera, SEdHC, ETSAM, A. E. Benvenuto, COAM, F. Dragados, Madrid*.

Bartoli G., Betti M., Spinelli P., Tordini B. 2006, An 'innovative' procedure for assessing the seismic capacity of historical tall buildings: the "Torre Gross" masonry tower. *Structural Analysis of Historical Constructions, P.B. Lourenco, P. Roca, C. Modena, S. Agrawal (Eds.), New Delhi.*

Betti M., Vignoli A. 2008. Modelling and analysis of a Romanesque church under earthquake loading: assessment of seismic resistance. *Engineering Structures* 30(2): 352–367.

Čajka R., Kaloèová L. 2008, Progressive Approach to the Analysis of Post-Tensioned Masonry Structures using FEM. *EngOpt 2008. International Conference on Engineering Optimization, Rio de Janeiro, Brazil.*

Figueiras J.A., Povoas R.H.C.F. 1994. Modelling of prestressin non-linear analysis of concrete structures. *Computers & Structures* 53:173–187.

Leftheris B., Tzanaki E. & Stavroulaki M.E. 1993. Dynamic criteria for reinforcement of old buildings. *In Proc. of STREMA 93, 'Structural Studies, Repairs and Maintenance of Historical buildings III'. Editors: C.A. Brebbia & R.J.B. Frewer. UK.*

Leftheris B., Stavroulaki M.E., Sapounaki A.C., Stavroulakis G.E. 2006. *Computational methods for heritage structures*, WIT Press, Southampton, U.K.

MSC engineering group, (2002), Marc-Mentat, 2000 Manuals, Theory and User Information.