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Nonlinear Analysis and Strengthening Design of an Italian Masonry Monumental Building

F. Angotti, L. Aprile, M. Orlando, B. Ortolani and A. Vignoli
University of Florence, Department of Civil Engineering, Florence, Italy

ABSTRACT: The analysis of monumental buildings is very complex because of their irregular geometry, the variability of the properties of traditional materials, the different building techniques. As a matter of fact due to their intrinsic complexity, monumental buildings are by definition unique buildings, and they cannot be reduced to any standard structural scheme: this makes difficult to evaluate their seismic capacity. In this paper the masonry structure of an Italian monumental building, the "Teatro Comunale dei Ricomposti" in Anghiari is analysed using the computer code DIANA in order to assess its structural behaviour and its seismic vulnerability. On the basis of the numerical analyses, the unsafe regions are identified and their strengthening is designed using steel lattice structures together with other retrofitting techniques.

1 INTRODUCTION

The main object of the present work is the structural analysis of a masonry monumental building, the "Teatro Comunale dei Ricomposti" in Anghiari (see Fig. 1), a beautiful medieval Tuscan village. The objectives of the structural analysis are the following:

- evaluation of the building's safety against vertical loads according to the Italian Recommendations;
- evaluation of the building's safety against horizontal seismic loads according to the Italian Seismic Recommendations;
- proposal of strenghthening works.

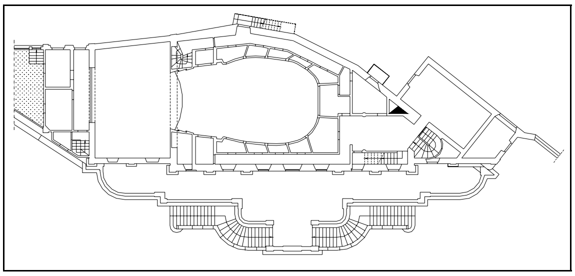
(a) (b)

Figure 1 : "Teatro Comunale dei Ricomposti" in Anghiari. (a) External view. (b) Internal view.

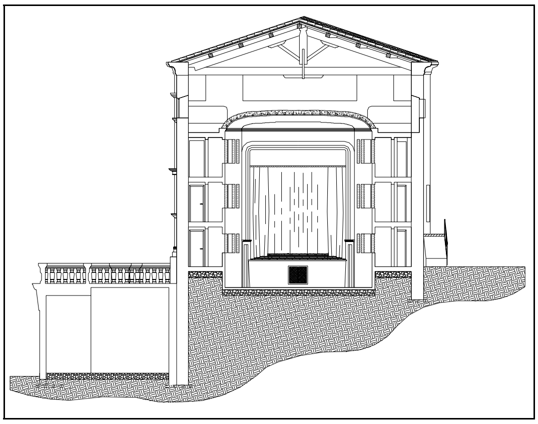
Existing buildings are different from new ones for two reasons. Firstly the project of existing buildings is based on the techniques known at the time of their construction, so that they can be defective both from a theoretical and practical point of view. Moreover these defects are usually difficult to be noticed. Secondly existing buildings can be characterized by a general structural “chaos” due to past repairing works, which have been often made in a hurry or as temporary works and have become permanent later.

The knowledge of the structure of an existing masonry building is the basis to properly study and then to efficiently restore it (Siviero et al., 1997). That is why the Italian Seismic Recommendations (O.P.C.M. no. 3274, 2003) establish the analysis method and the safety coefficient on the basis of the knowledge level (LC1 basic knowledge, LC2 adequate knowledge, LC3 accurate knowledge). The level of knowledge depends on the completeness and on the reliability of the acquired information concerning the geometry, the structural details and the material properties (see Fig. 2).

In order to acquire the here above mentioned information, the study of the examined building has to start from a preliminary analysis. Both the architectural and structural surveys have been made; these are useful to trace the outline of the building’s “functioning” and its actual load “extent”.



(a)



(b)

Figure 2: Structural surveys of the Theatre. (a) Plan. (b) Section.

2 MAIN CHARACTERISTICS OF THE BUILDING

The “Teatro Comunale dei Ricomposti”, built in 1790, has gone through several subsequent annexes and restorations. The consequence is a complex shape characterized by the superposition of four blocks.

Starting from the knowledge of the geometry, the safety of the building has been evaluated against vertical loads, using the Limit State Method according to Italian Recommendations (D.M. 1987). In order to evaluate the stress pattern, the Italian Recommendations adopt a simplified calculation procedure. The building is conceived as a three-dimensional structure, made of vertical resistant elements (walls), connected to one another and to the foundations. The walls are assimilated to simple rollers for the floors. Vertical loads are applied considering their actual eccentricities together with conventional ones. The building does not show any particular problem due to vertical loads.

3 FINITE ELEMENT MODELLING

The expression “structural analysis” means the operations through which the real behaviour of a building can be represented by means of a theoretical model. This model has to take into account: the load-bearing elements, the restraints which connect the building and the ground, the material properties, the load conditions. Due to all the previously mentioned reasons, the definition of such theoretical model for a masonry is an extremely complex matter; for the present study a finite element (F.E.) model has been chosen (Aprile and Ortolani, 2005).

A F.E. element model has the benefit of implicitly underlying the connection between the different elements of the building, although the local responses are less reliable than mean ones. In particular, due to the complexity of the structure, the masonry has been treated as an homogeneous material without distinction between blocks (or bricks) and joints, such that joints and cracks are smeared out. This approach is usually adopted in global analysis of large-scale masonry structures, where a model which takes into account the combined action of bricks and joints as though under a magnifying glass would be inappropriate (Orlando et al., 2003, Carpinteri et al., 2005).

The structural analysis has been made using DIANA computer code, Release 8.1 (2002). The 3D model has been created using Autocad 2000 and then imported in the Preprocess iDIANA environment. The following elements have been used (see Fig. 3): shell elements for masonry walls; beam elements for girders and columns and truss elements for tie-beams.

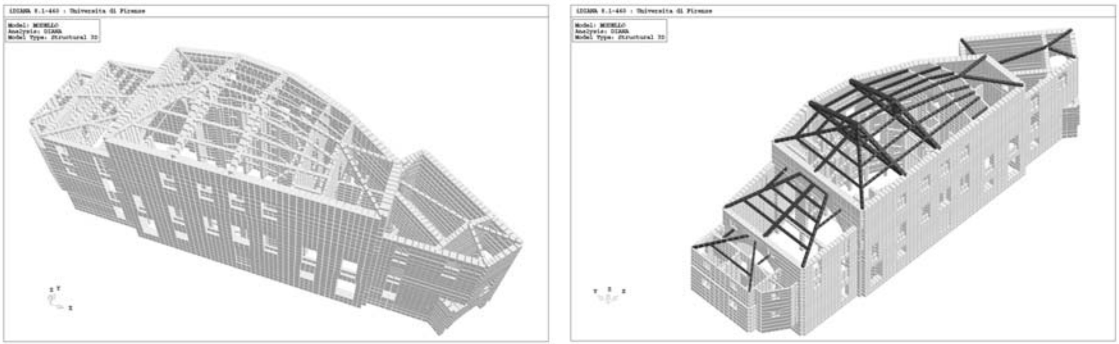


Figure 3 : Views of the F.E. Model.

The required mechanical characteristics for linear analyses are Young modulus E, Poisson coefficient ν and mass density ρ (see Table 1). Their values have been assumed equal to those suggested by Italian Recommendations for masonry walls with similar characteristics. It is important to outline that in the building three different kinds of masonry have been detected: 1 - three wythes masonry: outer wythes of marble blocks and inner fill with poor mechanical properties, 2 - stones masonry, 3 - full brick masonry. Walls have variable thicknesses, from a minimum of 12 cm for inner walls to a maximum of 60 cm for external ones.

Table 1 : Mechanical properties of the masonry.

Masonry	f_k N/mm^2	E N/mm^2	ν	ρ kN/m^3
Three wythes masonry	1.5	1500	0,25	18.00
Mixed stone masonry	2.0	2000	0,25	22.00
Brick masonry	3.0	3000	0,25	18.00

$$\begin{aligned} E &= 1000 \cdot f_k \\ G &= 0.40 \cdot E = 400 \cdot f_k \end{aligned} \;\Rightarrow\; \mathrm{G} = \frac{\mathrm{E}}{2 \cdot (1+\nu)} \;\Rightarrow\; \nu = 0.25 \qquad (1)$$

Only the main beams of the slabs have been modelled; the dead weight of the secondary beams has been added to the floor surface loads. Finally, fixed restraints have been applied to the nodes at the base of the building

4 LINEAR ANALYSIS

The static linear analysis of the building under vertical loads has been performed in order to verify the correctness of the choices made in the creation of the F.E. model of the building. The check has consisted in comparing the medium stresses obtained numerically with those evaluated by hand calculations.

Taking into account the irregularity of the building, the evaluation of its safety against horizontal seismic loads has been made through a dynamic analysis, according to the Italian Seismic Recommendations. In order to check the safety according to the "Limit State Method" on the basis of a linear analysis, the strength of each structural element has to be checked against the in-plane eccentric load, the shear action and the out-of-plane eccentric load (Augenti, 2004). The checks have been performed at the base of walls and in the sections adjacent to openings; the stresses in each section have been integrated automatically by the F.E. code. Table 2 lists the percentage of sections where the strength checks are satisfied.

Table 2: Results of the linear analysis.

Check	Percentage of verified sections	Percentage of not verified sections	Sections subject to tensile normal force
In-plane eccentric load	72 %	15 %	13 %
Shear	15 %	72 %	13 %
Out-of-plane eccentric load	73 %	14 %	13 %

The linear analysis highlighted:

- a lack of resistance of the building against horizontal seismic loads;
- the shear resistance is the least accomplished;
- among all load combinations, the worst are those where horizontal loads in the transversal direction prevail and/or wind-brace elements are missing.

5 NONLINEAR ANALYSIS

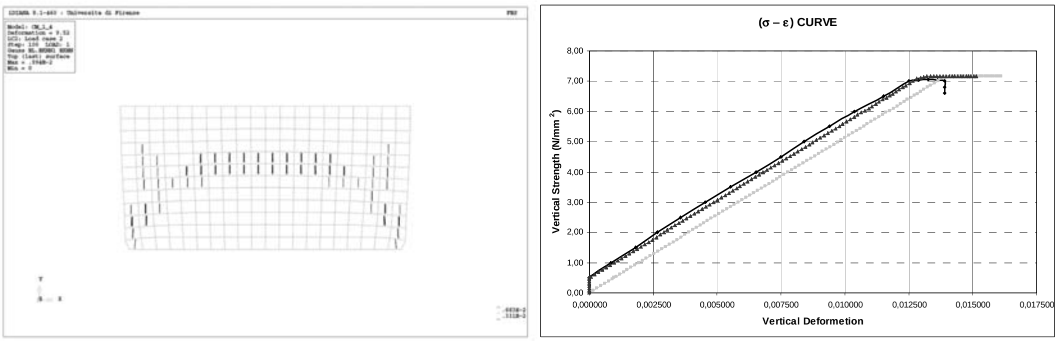
The strengthening design of the building based upon results of the linear analysis would lead to massive works, so that a most reliable evaluation of the actual carrying capacity has been performed in the nonlinear field. Assuming the lack of reliable information concerning the mechanical characteristics of the building masonry, significant indications about the most suitable mechanical model and the values of the input parameters have been taken from the numerical simulation of some literature experimental tests on masonry panels: one normal compression test and three shear –compression tests in situ.

Among all mechanical models available in DIANA, the isotropic plasticity model (Drucker – Prager) and multi–directional fixed smeared crack model have been chosen. The following parameters have been assumed: internal friction angle $\phi = 10°$ (suggested in DIANA's manual for the biaxial compression state in reinforced concrete); dilatancy angle $\psi = \phi = 10°$ (normality law); cohesion according to eq. (2), where f_c is the uniaxial compressive strength; constant tension cut-off criterion for tension-compression regions with uniaxial tensile strength according to eq. (3), where f_{vk0} is the shear strength at zero compression; no tension softening (i.e. brittle cracking); shear retention factor $\beta = 0.25$. For f_c and f_{vk0} common values from literature have been adopted:

$$c = \frac{1-\sin\phi}{2\cdot\cos\phi}\cdot f_c\,, \tag{2}$$

$f_t = 1.50 \cdot f_{vk0}$ (for parabolic distribution of shear stresses over the transversal section). (3)

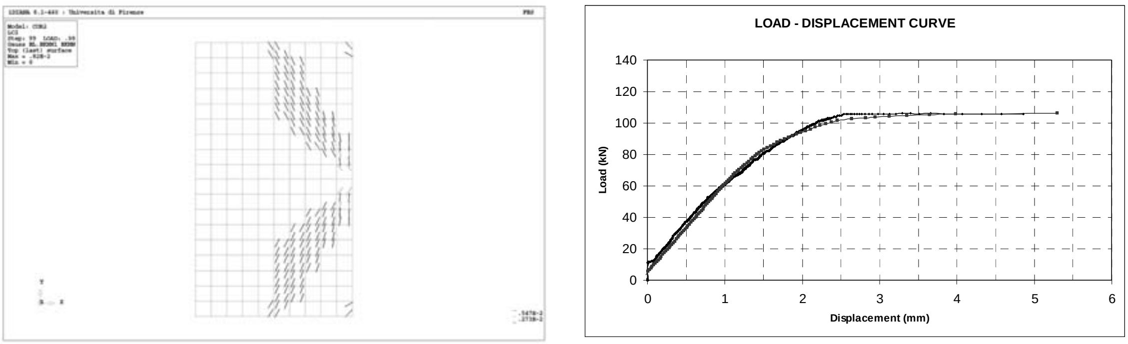
The normal compression test is a destructive test for the determination of the characteristic compressive strength f_k of a masonry. The panel has been modelled with shell elements, fixed restraints have been applied to the nodes at the base. The load has been imposed through a fixed displacement. The crack pattern and the σ - ε curve are shown in Fig. 4.



(a) (b)

Figure 4 : (a) Crack pattern. (b) $\sigma - \varepsilon$ curve.

In the shear-compression test the panel is subject to a constant normal load, while a shear load is applied monotonically up to collapse. The panel has been modelled with shell elements, fixed restraints have been applied to the nodes at the base. One test has been made on a mixed stone and brick masonry with block of different dimensions and with prevailing middle and big size blocks, and almost regular brickwork. The other two have been made on a three wythes masonry: outer wythes of marble blocks and inner fill with poor mechanical properties. The crack pattern and the load – displacement diagram of the first test are shown in Fig. 5.



(a) (b)

Figure 5 : First shear-compression test: (a) Crack pattern. (b) Load – displacement curve.

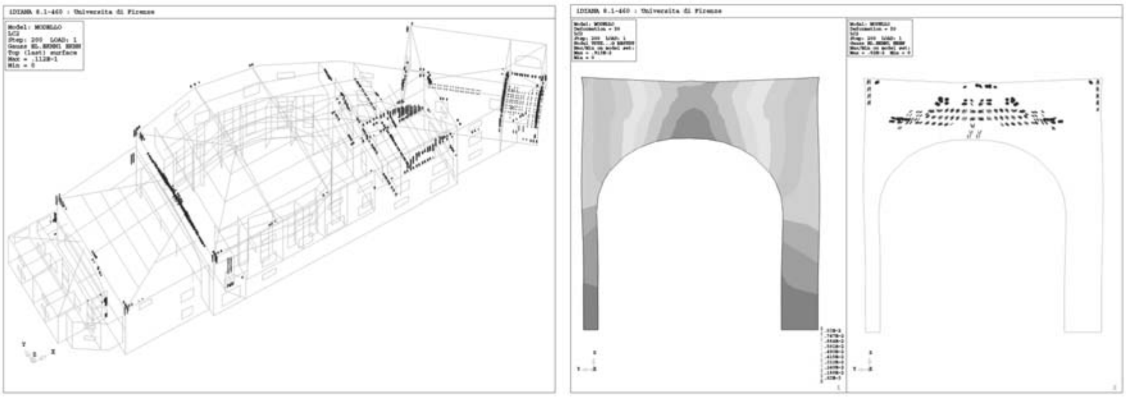
The nonlinear static analysis of the building under vertical (see Fig. 6) and seismic loads has allowed:

1. to assess the percentage of seismic load that the structure is able to carry in the two main directions;
2. to identify the weakest portions of the structure and its possible collapse mechanisms on the basis of the obtained crack patterns;
3. to design strengthening works to prevent the recognized collapse mechanisms.

The horizontal seismic loads have been represented through equivalent static forces, calculated on the basis of the elastic response spectrum prescribed by the Italian Recommendations.

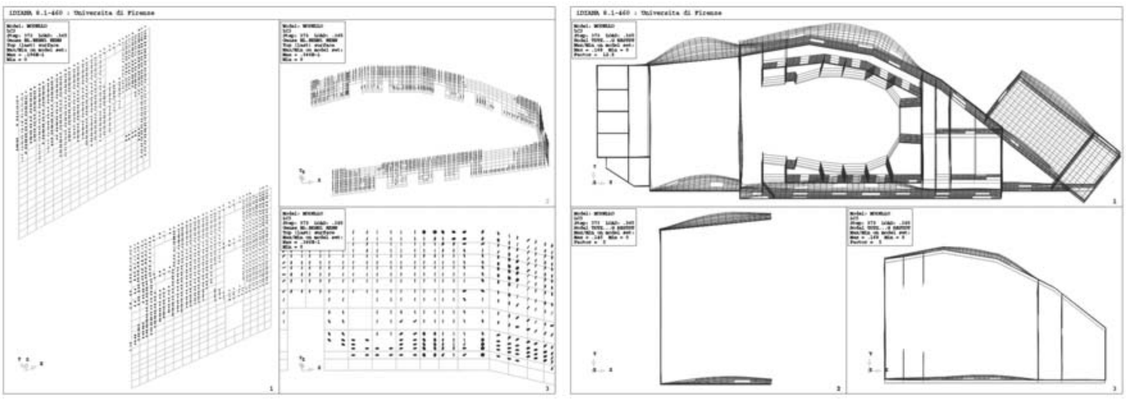
The percentage of seismic load that the structure is able to carry along the longitudinal direction is the 83,0 % of the load prescribed by the Italian Recommendations, while along the transversal direction it is only the 36,5 %. The carrying capacity of the building could have been overestimated due to the hypothesis of perfect connections among orthogonal walls used in the

F.E. model (and very difficult to eliminate in complex models). This hypothesis has brought to an “arch” collapse mechanism instead of an “overturning” mechanism of the upper portion of one of the longitudinal walls (see Fig. 7).



(a) (b)

Figure 6 : Vertical loads: (a) Cracking path. (b) Vertical stresses and cracking pattern on the proscenium arch.



(a) (b)

Figure 7 : Horizontal seismic loads in the transversal direction;
(a) Cracking pattern. (b) Deformed shape.

6 PROPOSAL OF STRENGHTENING WORKS

On the basis of the results of numerical analyses of the building, the weakest areas of the masonry walls have been identified (see Fig. 8) and strengthening works have been designed in order to increase the safety level of the building, in the logic of the seismic improvement. In fact it is impossible to modify the structure in order to satisfy the requirements of the Italian Seismic Recommendations without doing major restoring works, which would anyway alter its nature. Therefore a few minor as well as reversible strengthening works have been proposed, in order to enhance the box behaviour of the building (Del Piero, 1983).

The results of the nonlinear analysis (deformed shape and crack patterns) are confirmed by the crack patterns observed in situ. The local check of the walls against their out-of-plane overturning highlighted that this collapse mechanism for the upper walls is characterized by the lowest safety factor. Compared to the nonlinear analysis, this difference is due to the fact that in the F.E. model a perfect connection among orthogonal walls has been assumed to reduce both the complexity of the model and the computational time, although weak connections could be modelled using double non-connected nodes or interface elements.

The strengthening design has been developed in order to guarantee a box behaviour of the structure and to eliminate the thrust of the roof. To this aim the connection among masonry walls and roof beams has been improved, through the insertion of a steel beam over the top of the walls. In the scenic tower a horizontal lattice truss meant to replace the existing gallery (required to reach the trellis, device used to move the scenography) is created in order to avoid the overturning of the walls (see Fig. 10).

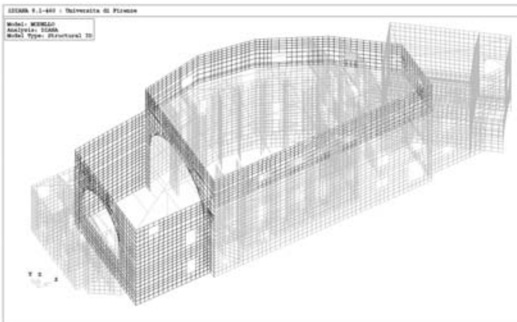
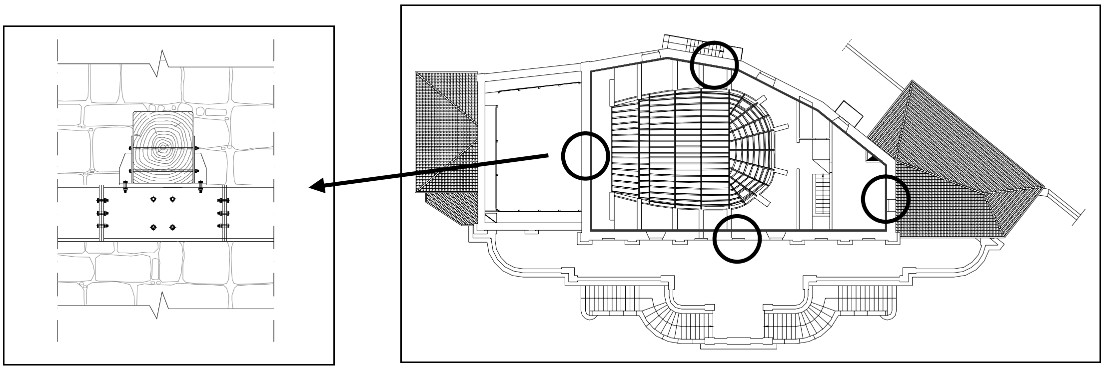


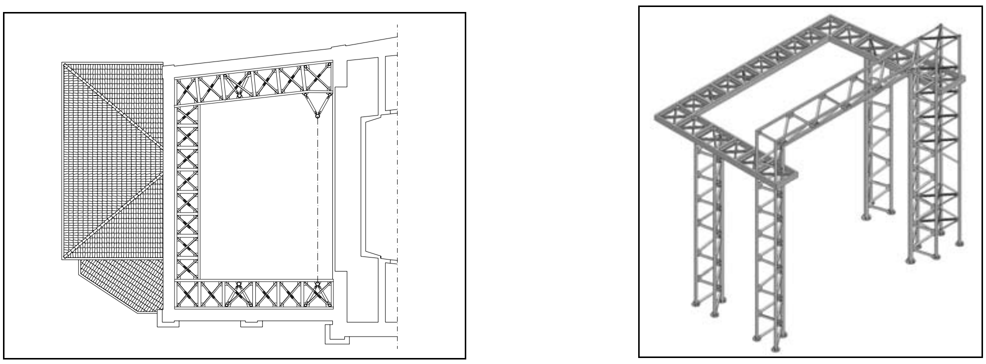
Figure 8 : Weakest areas of the masonry walls.

A 3D frame is placed in order to help the proscenium arch to resist the horizontal seismic actions. Moreover, two lattice-columns are placed to avoid the buckling of the compressed chord of the horizontal lattice truss and the walls bending.



(a) (b)

Figure 9 : Strenghtening work at the top of the walls using a steel beam.
(a) Detail. (b) Plan.



(a) (b)

Figure 10 : Strenghtening work in the scenic tower using a steel lattice structure.
(a) Plan. (b) Steel lattice structures 3D view.

7 CONCLUSIONS

The studied building does not show any peculiar problem due to vertical loads, whereas it clearly shows a general lack of resistance as far as the global seismic behaviour is concerned.
Within the hypotheses adopted in the numerical model, valid indications have been obtained about the weak areas of the building and about possible collapse mechanisms that can be prevented with adequate strengthening works.
The main limit of the finite element model is the fact that does not take into account the actual degree of connection among orthogonal walls.
In the weak areas of the building a few strengthening works suitable to avoid the overturning of the walls as well as to favour their collaboration against the seismic action have been planned.
Mostly important are the results obtained from the nonlinear analysis, which is useful, although complex, each and every time ancient buildings must be restored making sure to improve their behaviour without doing massive works.

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