Non-Destructive Damage Detection and Retrofitting Techniques on a Historical Masonry Tower

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ABSTRACT

The aim of the paper is to detect the damage of the bell tower of the Church "Santa Maria della Natività", Noci (Bari, Italy) that in 2012 was hit by a lightning and to propose a retrofitting solution. The solution must be designed to improve the constructive regularity, the strength and ductility of the structure, especially on the more resistant structural elements or on the mechanisms of collapse so as to transform them from brittle to ductile ones. The tower is 35 m tall and it is structurally connected to the Church for about one third of its height; the remaining part of the tower is totally disconnected from the structure of the Church. The results of the experimental tests have been analyzed in order to estimate the modal parameters of the tower and to calibrate a 3D finite element model of the tower in order to design *ad hoc* improvement interventions.

INTRODUCTION

The detailed model of a historical building is very important for a correct seismic vulnerability evaluation and for designing possible structural interventions. On the other hand, due to the historical character of the building, all the necessary information for the definition of the aforementioned model are usually unavailable, and the possibility of conducting classical tests is limited to the non-destructive ones [1-4].

The historical buildings analysis by means of an accurate model is a topic of great interest especially considering the possible applicability of this model to the seismic vulnerability evaluation and to the design of eventual structural renovation [5-6]. Different kinds of non-destructive tests are proposed in the codes and in the research field, such as radar tomography, microwave remote sensing, endoscopy inspection, laser scanner. They are widely spread as they may be compatible with the ordinary service of the structure. In principle, the collected knowledge of global and local geometry, the construction details and the mechanical characterization of the materials should be synthesized in a Finite Element (FE) model, so that the results of numerical simulations should provide the decision makers with an exhaustive picture of the structural condition.

Also, the dynamical tests have acquired a great importance in recent studies. These tests are based on the consideration that the modal parameters of a structure are strictly dependent on its geometrical and mechanical characteristics. The analysis of the vibrations of a structure allows the evaluation of these parameters, which are then used to update some uncertain structural parameters of the FE model in order to get a numerical model able to give an exhaustive picture of the real structure [7-11].

The final aim is to evaluate the vulnerability of historical existing buildings, and eventually to design repair intervention solutions and retrofitting to seismic actions. Retrofitting interventions on old masonry structures are quite common as these ones, like the tower considered in this study, which have been built to resist only to vertical actions. Studies consider different kinds of analyses [12], the mechanical characterization of the structural materials [13] and intervention techniques [14-15] to improve the behavior of masonry structures.

In this paper a series of in-situ tests have been performed in order to detect the damage of the bell tower of the Church "Santa Maria della Natività", Noci (Bari, Italy) that in 2012 was hit by a lightning (Figure 1). The aim is to propose a retrofitting solution to improve the constructive regularity, the strength and ductility of the structure, especially on the more resistant structural elements or on the mechanisms of collapse, so as to transform them from brittle to ductile ones.

Figure 1 Church of "Santa Maria della Natività", Noci (Bari, Italy).

The bell tower was built, in substitution of the existing one, between the 1758 and 1761, by architect Magarelli from Monopoli (Bari). It is 35 m tall and is structurally connected to the Church for about one third of its height; the remaining part of the tower is totally disconnected from the structure of the Church.

The tower is realized with two types of masonry: outside blocks of limestone with regular geometry joined by mortar with a thickness of less than 5 mm; inside, the masonry walls consist of limestone ashlar with a rather regular geometry and mortar of good quality, although with variable characteristics.

The supporting structures are developed through six levels of masonry walls for a total height of 30 m. From a plan view the structure is organized with four baffles, two parallel to the main axis of the church and two in the orthogonal direction; the wall sections are variable in thickness and performance according to the vertical development of the tower. The interior masonry walls are plastered up to the first level, while the remaining interior walls and all the external ones are made with face-view limestone blocks.

The in-situ tests carried out on the tower consist in:

- endoscopic inspection;
- 2D and 3D radar tomography:
- local acquisition of structural vibrations.

From the first tests it has been possible to notice a stratigraphy of the structural walls that everywhere detects the presence of masonry layers, interspersed with small cavities made from a little altered material.

Finally, a finite element model of the building has been built taking into account all the information gathered by the non-destructive techniques. The results of the experimental tests, in fact, have been analyzed in order to estimate the modal parameters of the tower, to calibrate a 3D finite element model and to design *ad hoc* improvement interventions.

DESCRIPTION OF THE STRUCTURES

By a careful visual exam and by means of a structural diagnostic investigations campaign, it has been possible to outline the conservation status of the structures. From the point of view purely related to the structural design criteria, the walls can be classified into two typologies: outside blocks of limestone of regular geometry with mortar joints of thickness less than 5 mm, while, inside, the wall apparatus consists of blocks of limestone with a fairly regular geometry and mortar of good quality, although with varying characteristics.

From the analysis of related endoscopic inspections, integrated with 2D and 3D radar tomographies, it has been possible to reconstruct the masonry stratigraphy; it is everywhere composed by a layered masonry, interspersed with cavities of a reduced thickness formed by little altered material.

The pictures in Figs. 2-4 show the problems evidenced above.

Figure 2 Internal disconnection of the keystone of an opening in the belfry.

STRUCTURAL MODEL – ANALYSIS OF THE COLLAPSE KINEMATISMS

Following the Italian Code on existing masonry buildings it is necessary to check the presence or not of partial collapses due to seismic events, usually for loss of equilibrium of portions of masonry. The control of these mechanisms, according to the methods described below, has meaning if a certain monolithic nature of the masonry wall is guaranteed, such as to prevent punctual collapses for disintegration of the masonry. These controls may be carried out using the limit equilibrium analysis, according to the kinematic approach, based on the choice of the collapse mechanism and the horizontal force evaluation, which activates such mechanism.

The application of the verification method assumed the analysis of the local mechanisms significant for the construction, identified considering the wide crack pattern that affects the masonry of the bell tower of the Church.

Figure 3 Disconnection a) at the basement level of the spire collapsed because of lightning; b) at the level of the keystone of an opening of the belfry.

Moreover, thanks to the knowledge of the building process conducted in depth, it was possible to ascertain the poor quality of such connections, the presence of wall layers and, in contrast, a good masonry texture (Figure 4).

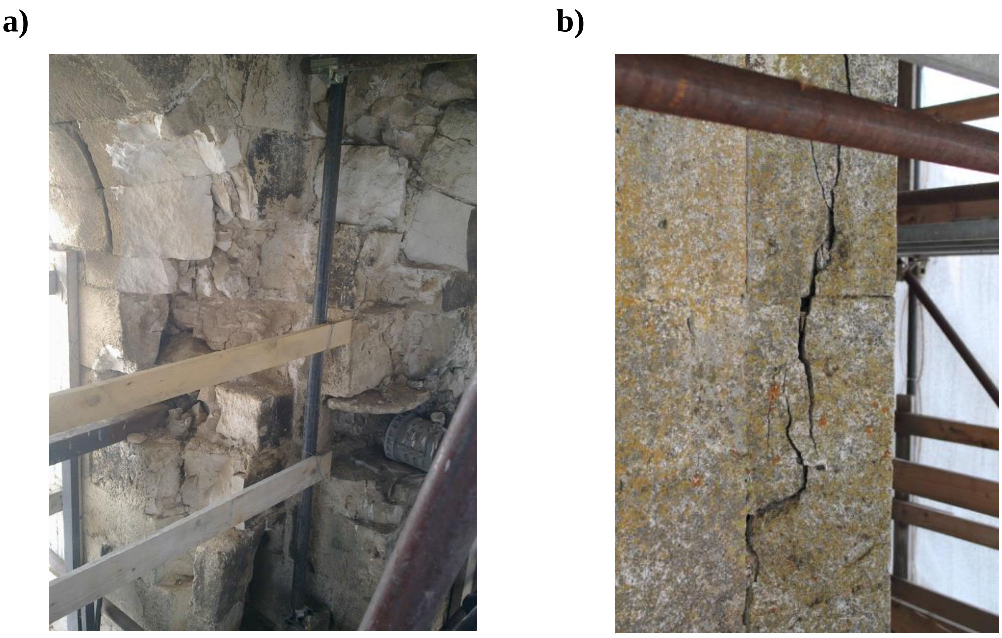

Figure 4 Detail a) masonry layers; b) crack pattern at the first level.

The kinematic approach also allows to determine the trend of the horizontal force that the structure is progressively able to withstand as the mechanism changes. This curve is expressed through a multiplier ***a***, ratio between the horizontal forces applied and the corresponding weights of the masses, represented in function of the displacement d_k of a reference point of the system; the curve must be determined up to the inability to withstand horizontal forces (a = 0). This curve can be transformed into the curve of an equivalent one-degree-of-freedom system, in which the ultimate displacement capacity of the local mechanism may be defined, to compare with the displacement demand requested by the seismic action. As a consequence, it was decided to evaluate the kinematic mechanisms of simple tilting of the walls of the tower. This analysis is by far the worst rating for the response both of the load-bearing wall structure and the reinforcement interventions to be executed. A choice was made, therefore, according to safety reasons, given the dangerous situation of the structure.

Once the local mechanism considered significant for the building is identified, the method can be divided into the following steps:

- Transformation of a part of the construction in a labile system (kinematic chain), through the identification of rigid bodies -defined by probable fracture plans due to the poor strength of the masonry- able to rotate or slide between them (mechanism of damage and collapse);
- Evaluation of the horizontal loads multiplier a_0 which is able to activate the mechanism (damage limit state);
- Evaluation of the evolution of the horizontal loads multiplier a as the displacement d_k of a control point increases up to the cancellation of the horizontal seismic force; this point is usually chosen close to the center of gravity of the masses;
- Transformation of the curve so obtained in a capacity curve, that is in spectral acceleration a^* and displacement d^*, with the evaluation of the ultimate displacement for collapse mechanism (ultimate limit state);
- Security checks, through the control of the displacements and/or strengths compatibility requested to the structure.

One of the key parameters to be assessed for the modeling of the mechanisms is the critical angle of the masonry crack; this parameter determines the shape of the wall portions involved in the kinematism according to forces acting in their higher resistance plane (amplitude of the gap wedges). The procedure for determining this angle was conducted graphically by evaluating the most narrow wedge among all obtainable wedges without breaking stones or blocks but only passing through the joints, and the one that minimizes the mutual interlocking between the blocks.

ASSESSMENT OF THE LEVEL OF KNOWLEDGE AND CONFIDENCE FACTOR

During the realization of the structural model, a high level of knowledge was reached on the basis of the diagnostic investigations (LC3 knowledge level according to DM 2008 [18]), resulting in a confidence factor FC = 1.00 of the many parameters involved in the model (geometry, construction details and materials). To this end, given the type of structure made in masonry, all inspections and investigations carried out in-situ are considered "extended", due to the complexity of the structure and the implementation of mechanical tests for what concerns the material properties. The following investigations have been performed:

1. endoscopic investigations to assess the boundary stratigraphy, together with high frequency prospecting and radar tomography in order to reconstruct the structural geometry of the tower;
2. thermographic inspections and in-depth visual analysis to detect the presence of anomalies in the wall;
3. tests with flat jacks, removal of a masonry ashlar for compression strength laboratory tests and rebound hammer tests on stone blocks of the wall to define the mechanical characteristics of the wall itself.

Specifically, for what concerns the mechanical parameters, the resistance value of the masonry is obtained from the following tests:

- Flat Jacks: $\sigma_r = 220$ N/cm^2
- Characteristic strength (EC6 [18]) using the strength of the stone ashlar evaluated in the laboratory and that of the mortar on-site: $\sigma_r = 495$ N/cm^2.

Having reached a high level of knowledge of the existing building and with an average characteristic strength equal to 357 N/cm^2 the average values taken from DM 2008 - Table C8A.2.1. have been considered (Table 1).

After the reinforcing interventions the masonry parameters - compression and shear strengths, longitudinal and shear moduli of elasticity of the existing masonry of stone with a good texture - were multiplied by the correction factor 1.65 (= 1.1 joint reinforcing * 1.5 injections of mixtures and binders) (see Table 1):

Table 1 Characteristics of masonry before and after the restoring interventions.

Pre-Interventıon	**Post-Interventıon**
$f_m = 320$ N/cm^2	$f_m = 530$ N/cm^2
$\tau_0 = 6.5$ N/cm^2	$\tau_0 = 10.7$ N/cm^2
E = 1740 N/mm^2	E = 2870 N/mm^2
G = 580 N/mm^2	G = 960 N/mm^2

ANALYSIS OF STRUCTURAL REGULARITY

For the purposes of the modeling settings it is required the preliminary analysis "of the regularity in plan and height of the building", through the check of eight points indicated by the code; the first four relate to the regularity check of the structure in plan-view, the other four in elevation. They are described below:

- Configuration in plan compact and roughly symmetrical with respect to two orthogonal directions, in relation to the distribution of masses and stiffnesses;
- Ratio between the sides of a rectangle in which the building is inscribed (max 4);
- Maximum value of indentations and projections expressed as a percentage of the total size of the building in the direction of the indentation or projection (max 25);
- Infinitely rigid floors;
- Minimum vertical extension of a resistant element of the building (frames or walls) expressed as a percentage of the building height (min 100%);
- Maximum variation, from one floor to the following one, of the mass and stiffness expressed as a percentage of the mass and stiffness of the adjacent floor that shows the highest values (max 20%);
- Maximum narrowings of the section of the building, in proportion to the size corresponding to the first floor, and to the one corresponding to the immediately underlying floor (30% max - min 10%);
- Presence of structural elements particularly vulnerable or that can adversely affect the response of the structure.

It is possible to immediately define a regularity in height of the tower.

LINEAR-STATIC ANALYSIS

The linear static analysis consists in the application of static forces equivalent to inertial forces induced by the seismic action. It can be applied to all buildings that meet the following specific requirements:

- The period of the principal mode of vibration T1 < 2.5 TC
- The building is regular in height.

In the present case the tower is regular in height; T1 = C1*H3/4 = C1*34 *3/4 = 0.70 s. Therefore T1=0.70s ≤ 1.05s = 2.5TC. This means that the linear static analysis can be applied.

The analysis of the structure was carried out by "Spectra NTC v.1.03" software of C.S.LL.PP [16].

To analyze the tensile-deformation behavior of the tower, a finite element modeling was built. A macro-elements discretization was utilized by means of the calculus software SismiCad [19]. The reason why to utilize this methodology is mainly because the constitutive law has been referred to the whole structural element, evaluated with the results obtained by a double flat-jacks test and radar surveys. The modeling consisted of 835 mesh-type shell elements with 3 or 4 nodes using the method of sparse matrices for the resolution.

The maximum stresses of the shells due to dead loads are shown in Figure 5.

a) b) c)

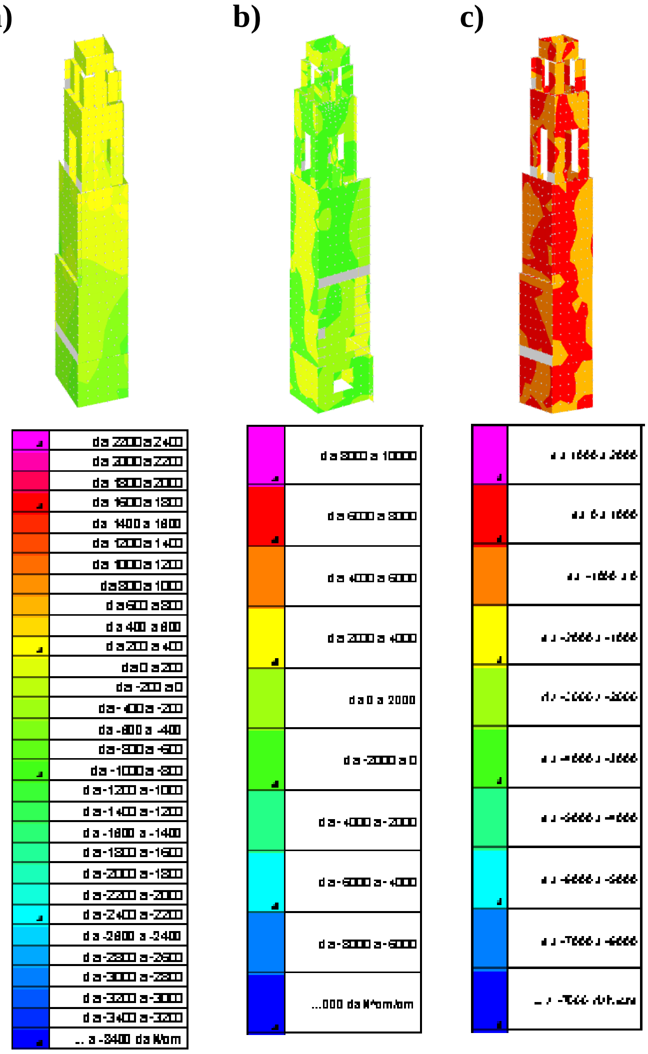

Figure 5 Maximum stresses of the shells due to dead loads for a) normal force; b) bending moment; c) shear force.

CONCLUSIONS AND RESTORING INTERVENTIONS

The restoring intervention carried out on the tower consisted in a set of works aimed to reinforce the structural elements. They are classified as interventions for improvement of an existing building according to DM 14.01.2008 (chap. 8.4.2) [17]. It must be avoided all the works of demolition-replacement and demolition-reconstruction, and make "... interventions that integrate with the existing structure without transforming it radically." In fact the intervention on an existing building must improve the structure constructive regularity, strength and ductility. The intervention is therefore applied more extensively, for example, on the most resistant structural elements or on the failure mechanisms so as to transform them from brittle to ductile.

Following the analysis of the failure mechanisms and the security checks, the structure requires a diversified approach of the interventions. The seismic behavior of such structures depends on factors such as the slenderness of the structure, the degree of clamping, the connections with lower structures (like in the present case with the Church), the presence of very vulnerable belfries. In the tower considered in the present research it is possible to classify the interventions in two main parts:

• Lower area, consisting of the base of the bell tower and the first level;

• Higher area, consisting of the second and third levels and the collapsed spire.

In the higher part it could be a good solution to restore the wooden horizontal structural elements originally present and now inexistent. Such elements will confer horizontal stiffness and, by means of the structures that connect the wooden beams (i.e. with a system of tie-rods) it will help to impose the stresses necessary to cover the overturning moments that trigger collapse kinematic mechanisms.

In the lower area it will be necessary to proceed to an extreme solidarization of the masonry walls which, in some points, are also made up of three separate panels. In addition, a system of rods and/or reinforced perforations can absorb seismic and pushing forces, which act on the masonry vaults. The latter also house the stairs that run from the base of the bell tower up to the belfry. It is important to emphasize the proper use in laying the rods system in accordance with the "Guidelines for the assessment and mitigation of the seismic risk of cultural heritage" [19]; in section 6.3.2: "the tie rods will have to be arranged side by side to the main wall. In cases where it is essential to drill the wall in the longitudinal direction, you will have to give way to chains inserted into the sock and not injected, in order to make the intervention reversible" (Figure 6).

Finally, along all the tower it is necessary interventions of grouting and sealing of all the uneven mortar joints, the lesions of the ashlars and all the clampings between the partition walls.

On the basis of the analyses and investigations carried out the following interventions are finally proposed. For each one the area of use is evidenced.

• Regeneration and consolidation of walls by injecting mixtures of binders made of hydraulic lime, aimed at consolidating the core and make the masonry homogeneous; through this intervention a continuity and a strength increment will be given to the wall apparatus.

• Fixing, pointing and sealing of the stone masonry joints by means of thixotropic mortars, which allow to impart to the masonry an improvement of strength to shear stress.

• In the points where significant quantities cracks were highlighted a replacement intervention to “undo and sew” of the irrecoverable stone elements will be applied, with removal of the loose parts and insertion of new blocks in the masonry.

• On the cantonal connection between the dividing walls, in correspondence of the vault pressure (lower zone) and at the base of the spire, at the base of the third and second level (higher zone) it is expected to insert the wooden bars connected by threaded steel tie rods with external anchor; the utilized mixture will be made of premixed mortar based on hydraulic lime.

This series of interventions gave the structure a better alignment from the point of view of the stiffness since the behavior can be considered monolithic. In this way the behavior at the base of the kinematic study of the previously analyzed structural problem is better approximated.

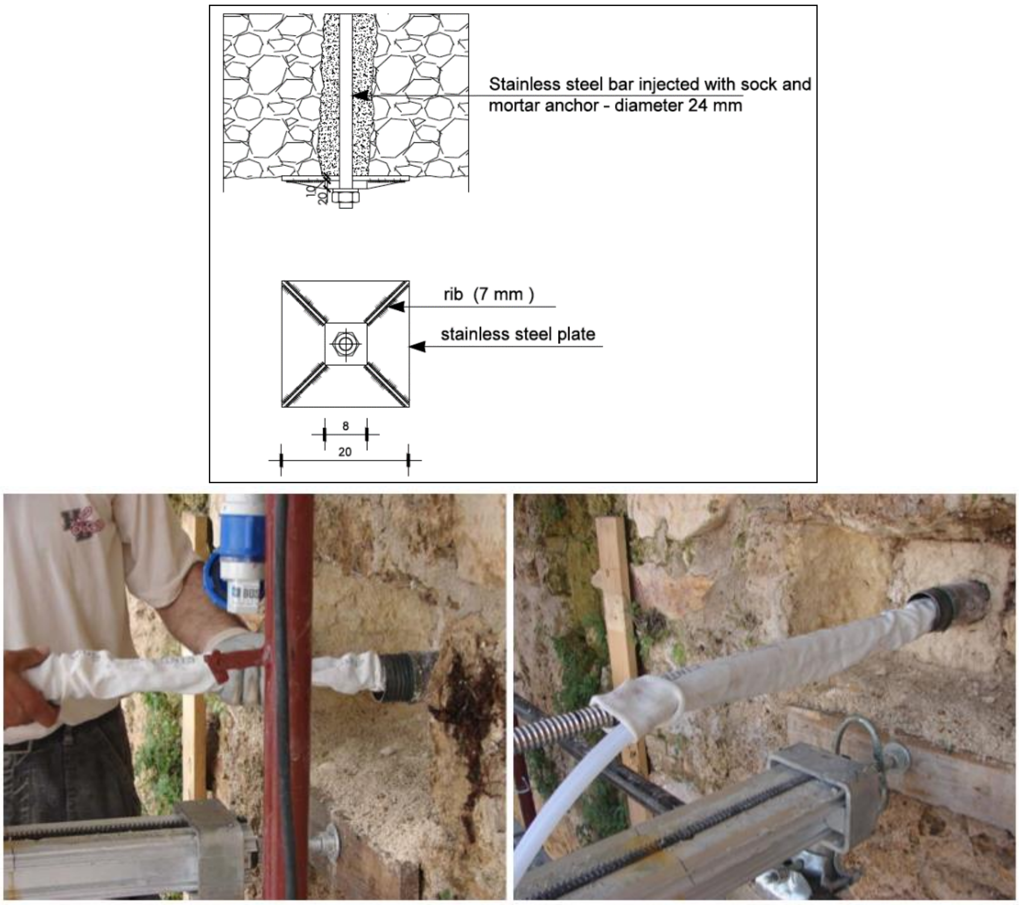

Figure 6 Installation of a stainless steel bar injected with sock and reinforcing anchoring steel plate.

REFERENCES

[1] Binda, L., Lenzi, G., Saisi, A. (1998) NDE of masonry structures: use of radar tests for the characterization of stone masonries. *NDT & E Int*, **31**, 411–419.

[13] Binda, L., Zanzi, L., Lualdi, M., Condoleo, P. (2005) The use of georadar to assess damage to a masonry Bell Tower in Cremona, Italy. *NDT & E Int*, **38**, 171–179.

[14] Diaferio, M., Foti, D., Giannoccaro, N.I. (2014) Non-destructive characterization and identification of the modal parameters of an old masonry tower, *EESMS 2014 - 2014 IEEE Workshop on Environmental, Energy and Structural Monitoring Systems*, Naples, Italy, 17-18 September.

[15] Diaferio, M., Foti, D., Gentile, C., Giannoccaro, N.I., Saisi, A. (2015) Dynamic testing of a historical slender building using accelerometers and radar. *6th Int Operational Modal Analysis Conference*, Gijón, Spain, paper ID 153, pp.129-130, ISBN: 978-84-

617-3880-9.

[16] Ivorra, S., Brotons, V., Foti, D., Diaferio, M. (2015) A Preliminary Approach of Dynamic Identification of Slender Buildings by Neuronal Networks. Int J of Non-Linear Mechanics, DOI: 10.1016/j.ijnonlinmec.2015.11.009.

[17] Diaferio, M., Foti, D., Giannoccaro, N.I. (2015) Identification of the modal properties of a building of the Greek heritage, Key Eng Mat, 628, 150-159.

[18] Diaferio, M., Foti D. and Sepe V. (2007) Dynamic Identification of the Tower of the Provincial Administration Building, Bari, Italy, Proc. of the Eleventh International Conference on Civil, Structural and Environmental Engineering Computing, Malta.

[19] Foti, D., Diaferio, M., Mongelli, M., Giannoccaro, N.I., Andersen, P. (2011) Operational Modal Analysis of a Historical Tower in Bari. Proc. of the Society for Experimental Mechanics Series, "IMAC XXIX", Jacksonville, Florida, USA, p. 335-342.

[20] Foti, D., Diaferio, M., Giannoccaro, N.I., Mongelli M. (2012) Ambient vibration testing, dynamic identification and model updating of a historic tower. NDT & E Int, 47, 88-95.

[21] Ivorra, S., Pallares, F.J. (2006) Dynamic investigations on a masonry bell tower. Eng Struct 28(5),660–667.

[22] Lepidi, M., Gattulli, V., Foti, D. (2009) Swinging-bell resonances and their cancellation identified by dynamical testing in a modern bell tower. Eng Struct, 31(7), 1486–1500.

[23] Foti, D. (2015) A New Experimental Approach to the Pushover Analysis of Masonry Buildings. Comp and Struct, 147, 165-171.

[24] Diaferio, M., Foti, D., Gentile, C., Giannoccaro, N.I. (2016) Modal parameters identification on environmental tests of an ancient tower and validation of its FE model", Int J of Mechanics, vol. 10, pp. 80-89.

[25] Modena, C. Valluzzi, M.R. Tongini Folli, R. Binda, L. (2002) Design choices and intervention techniques for repairing and strengthening of the Monza cathedral bell-tower. Constr and Building Mat, 16(7), 385–395.

[26] Foti, D. (2013) On the numerical and experimental strengthening assessment of tufa masonry with FRP. Mechanics of Advanced Mat and Struct, 20(02) 163-175.

[27] D.M. Infrastrutture 14.1.2008 and Circolare Esplicativa n.617, 02.02.2009 (in Italian).

[28] Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures.

[29] "SismiCad", Concrete s.r.l., Padova, 2006.

[30] "Circolare del Ministero dei Beni e le Attività Culturali", 2 dicembre 2010, n. 26.