PAPER REF: 3988

NON-DESTRUCTIVE CHARACTERIZATION AND DYNAMIC IDENTIFICATION OF AN HISTORICAL BELL TOWER

Angelo Salvatore Carone[1], Dora Foti[2], Nicola Ivan Giannoccaro[1(*)], Riccardo Nobile[1]

[1]Department of Engineering for Innovation , University of Salento, Lecce, Italy

[2]Department of Department of Civil Engineering and Architecture, Technical University of Bari, Bari,Italy

[(*)]*Email:* ivan.giannoccaro@unisalento.it

ABSTRACT

An accurate knowledge about the dynamics of structures is definitely useful for seismic assessment and design of risk mitigation interventions. In this paper, the opportunities provided by dynamic identification techniques for the non-destructive evaluation of heritage structures are discussed with focus on the bell tower of Announziata, a masonry tower, which shows a high damaged scenario and, consequently, a high vulnerability to dynamic and seismic forces. A Finite Element (FE) numerical model has been built for research into the structural behaviour, deformation and stress distribution of the tower under static and dynamic loading. The model has been updated considering the modal parameters obtained by experimental tests carried out on the tower. The experimental measurements were difficult because of the compromised state of the building. The dynamical identification and the model updating required several assumptions of the material behaviour and properties that are not accurately available.

Keywords: non-destructive tests, operational modal analysis, model updating, dynamic analysis, FEM analysis

INTRODUCTION

In the preservation of architectural heritages it is important a careful study of the structure and the dynamic behavior using identification techniques and numerical models. The necessity of identifying unknown geometrical data and material properties is due to the usual impossibility of conducting classical tests for estimating the materials characteristics of the elements of the structures; so the numerical models may be validated only using non-destructive techniques. One of these techniques is based on the experimental evaluation of the modal parameters of the structure in environmental conditions. The modal parameters may be compared with those of the model and the unknown materials and geometrical parameters may be estimated for obtaining an accurate FE model. Some recent examples of this technique may be found in Ivorra et al. 2006, Gentile et al. 2007, Lepidi et al. 2009, Foti et al. 2011 and 2012, Trombetti et al. 2011.

In this paper, the case of study is the bell tower of Announziata, a masonry tower (Fig. 1a), which shows a high damaged scenario. The bell tower of the church is located in the town of Corfù, Greece. The church was built in 1394 and it was one part of a Roman monastery, nowadays the only existing one. The interior of the church, (Kouris, 2009) contained the tombs of generals who died in the sea battle of Naupactus (1571). The church of Announziata was bombed during the second world war and it suffered from cracking and the church collapsed in March, 1952, when long term phenomena and earthquakes contributed to increase the damage. So, from 1952 , only the campanile of the medieval monastery complex has survived to the present day with heavy cracking. It is possible to observe (Fig. 1b) deep vertical and diagonal cracking, deterioration of mortar on the stone walls and development of creepers in the masonry.

Fig.1: a) Announziata bell tower b) cracking on the lateral side (west side)

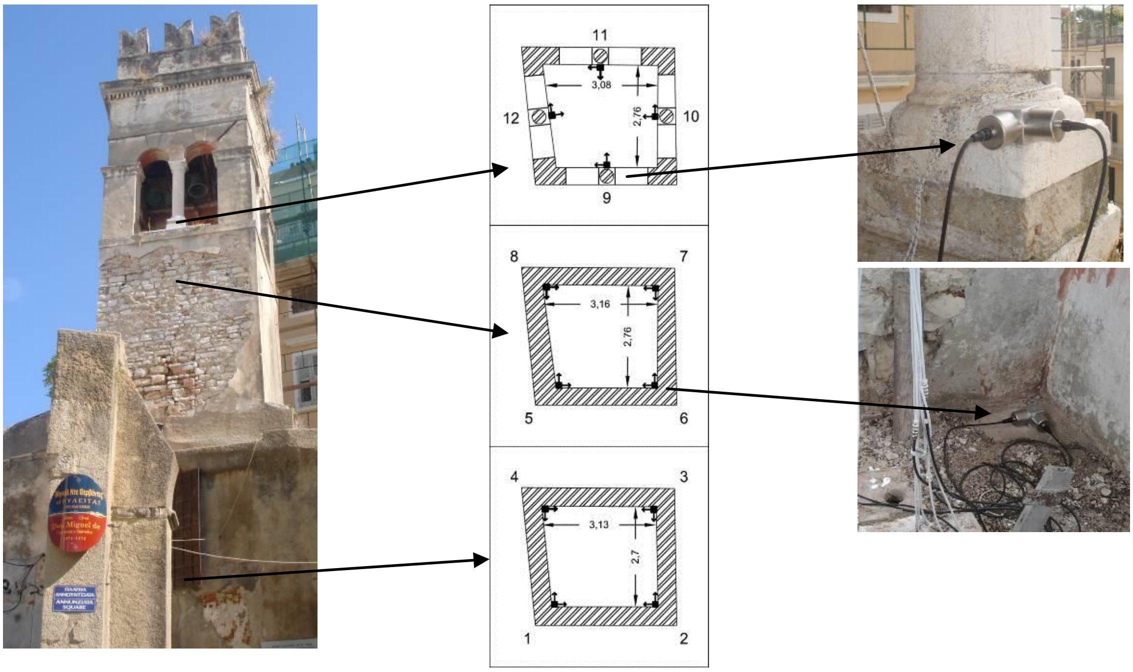
The bell tower is a stone tower with an almost square ground plan, with a side of about 3.5 m. The tower is about 20 m height and it shows a belfry at the top. A double arch (Fig. 1a) supported by a stone column in the center is present at each of the four sides. The structure of the tower is made with masonry walls and the three floor systems made by solid bricks are supported by vaults. At the top of the tower there is the bell cell with four bells. The actual state of the tower may be defined critical, with many damages and cracking.

A numerical model using beam and shell elements has been built to evaluate the structural behaviour of the tower. The initial value of the material properties and the mass distribution has been assumed on the basis of literature data and has been changed according to the experimental dynamical identification as will be fully described in this paper.

EXPERIMENTAL SETUP AND IDENTIFICATION PROCEDURE

The tower was instrumented with 24 high sensitivity seismic accelerometers ICP PCB 393B31 placed on 12 positions (labelled with numbers 1-12 on the plans in Fig. 2) on 3 different levels; 8 accelerometers were placed on the four corners of the first floor (position 1-4 in Fig. 2b), 8 placed on the four corners of the second floor (number 5-8 in Fig. 2b), the last 8 placed on the basis of the four columns (numbers 9-12 in Fig. 2b). Appropriate rectangular blocks (see Fig. 2c) were designed and realised in order to ensure the orthogonality of the couple of accelerometers placed on the same position. The accelerometers were inserted with screws on the threads realised on the perpendicular faces of the blocks (Fig. 2c).

It was not possible to achieve the belfry of the campanile to place accelerometers on the top of the tower; also there is to underline the situation very compromised of the building, let fall into decay in the last years. It was very difficult, also, to achieve the first floor that was accessible only using an external temporary wood.



a) bell tower b) positions of the accelerometers c) details of accelerometers positioning

Fig.2 Announciata bell tower and layout of the experimental setup.

The bell tower is positioned in the city centre, in a crowded area especially during day time. There is a main entrance road (Fig. 3a) open to the car passage and a rotatory (Fig. 3b) very close to the tower where the cars and the trucks can select the preferential way. The continuous passage of bikes, cars, motorcars and pedestrians has characterised all the environmental tests.

a) entrance road b) rotatory

Fig.3 Position of the Announziata bell tower.

Preliminary tests were carried out on 12th October 2012; in the following day several consecutive tests were conducted during the daytime. The data acquisition was carried out by recordings of 10 minutes with a frequency of 1024 Hz, which has been subsequently decimated by a factor equal to 4 to have a frequency of 256 Hz. Around 10 consecutive acquisitions were carried out and, in each acquisition, all the relevant events (passage of cars, motorbike, other possible disturbs), were noted. An heavy rain, unfortunately, occurred in the night between 12th and 13th October, and infiltration of water was verified on the instrumentation that was placed on the second floor and on the basis of the columns.

IDENTIFICATION RESULTS

The analysis of the experimental results was subsequently performed. A specific software (ARTeMIS) was used for the extraction of the modal parameters from ambient vibration data. The model crated in Artemis is shown in Figure 4 with the corresponding reference system *xyz*. A preliminary analysis was conducted on the time series of the accelerometers for evaluating the effects of the urban traffic and the functionality of the accelerometers considering the difficult environmental conditions. The preliminary analysis permitted to individuate as not properly running the accelerometers placed in the positions 23 (both directions x and y), 24 (direction x) and 32 (direction y) following the numeration introduced in the model of Fig. 4. Moreover the preliminary analysis permitted to clearly highlight the effects of external disturbances. In Figs 5-7 the time histories of three opportunely selected tests (named a, b, c) are shown in relation to the accelerometers placed in positions 21 and 22 of Fig. 4 (both the directions x and y). It is evident from Figs. 5-7 the extreme sensibility of the experimental setup to external events: for the test a (Fig. 5), there was heavy and continuous traffic after 260-280 seconds and some spurious event after around 440, 540 and 560 seconds from the starting. For the test b (Fig. 6) there are not evident effects on the time histories, while on the test c (Fig. 7) heavy traffic effects are evident at the beginning of the registration (80-120 seconds) and at the end of the registration (400-480 seconds) for the accelerometers along x axis (left column of Fig. 7), while some spurious events are evident after around 510 seconds.

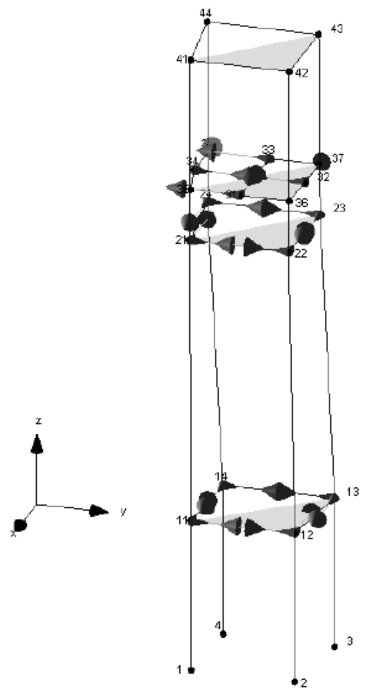


Fig.4 Reference system and model for the identification

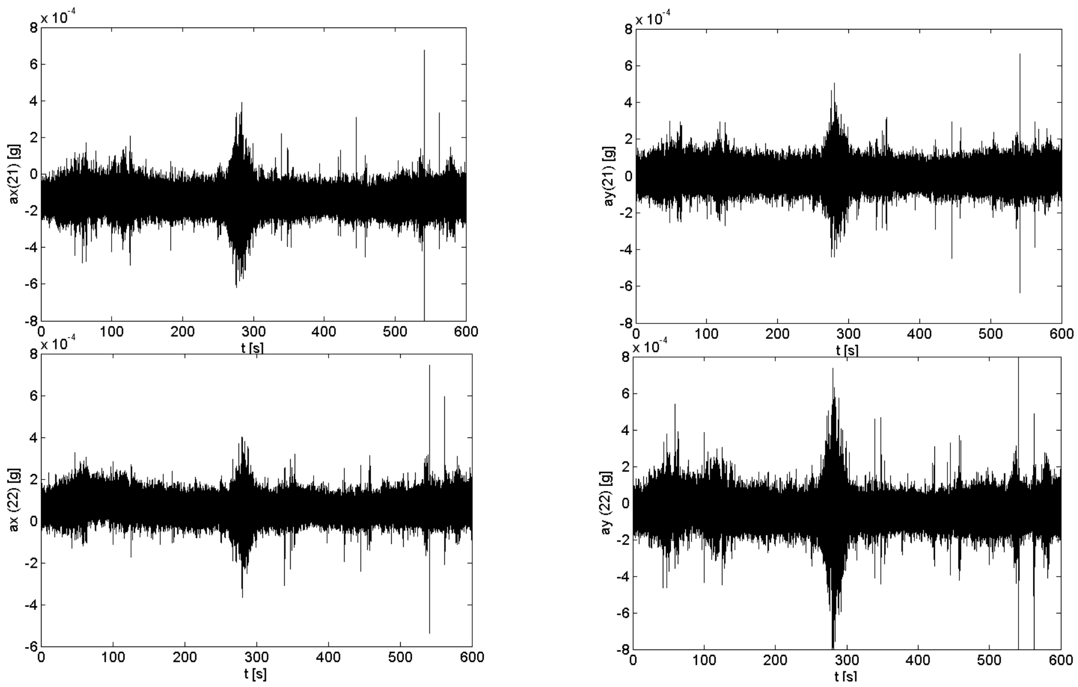


Fig.5 acquisition signals for points 21 and 22 for the test named a

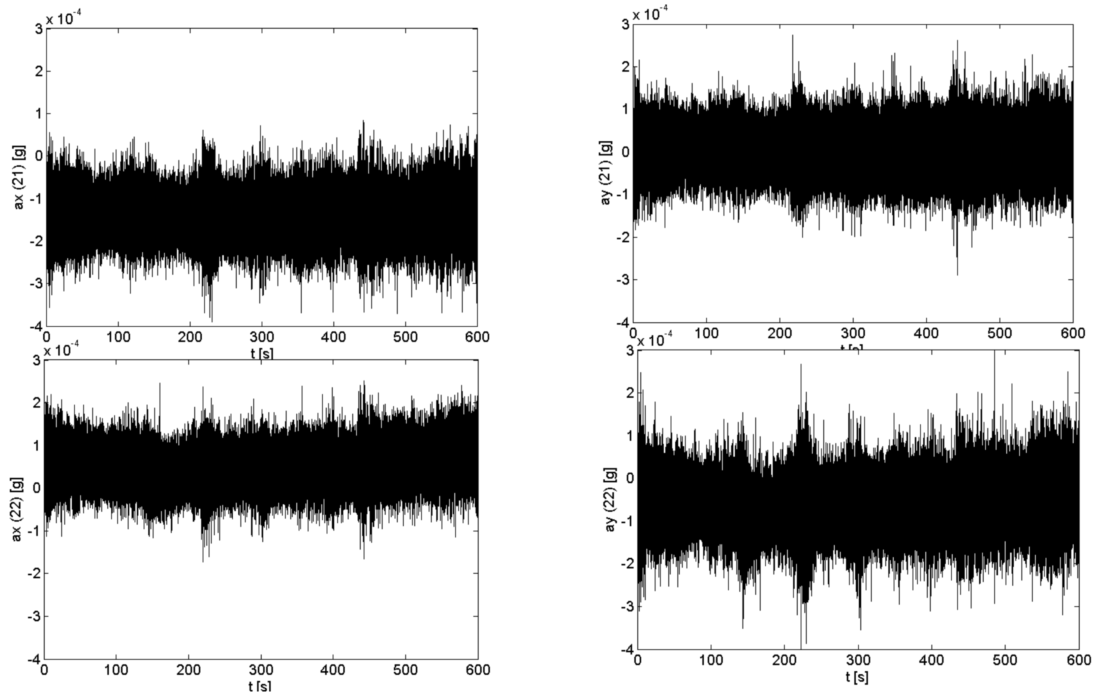


Fig.6 acquisition signals for points 21 and 22 for the test named b

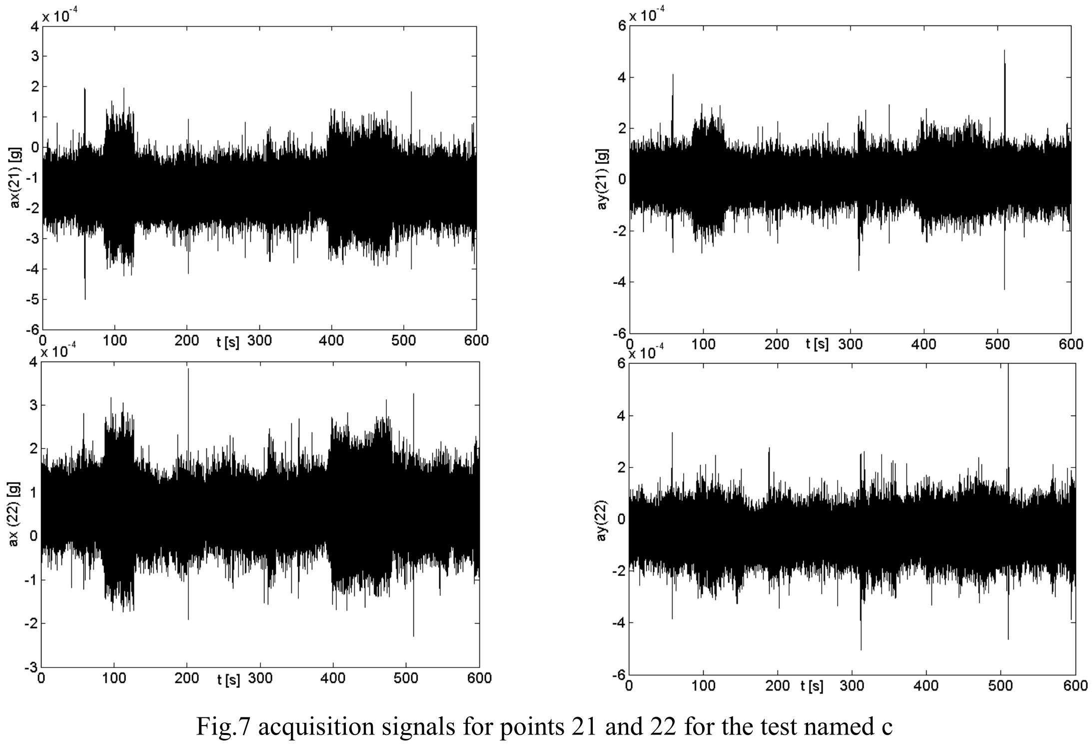


Fig.7 acquisition signals for points 21 and 22 for the test named c

The time histories of the selected tests seem to be very different each other for the external disturbances that influence the accelerometers data.
Anyway, a statistical based analysis of the tests using the classical methods of the operational modal analysis (OMA) has permitted to identify the frequencies of the building and the modal shapes with an extreme repeatability for all the tests.
Two different OMA methods were used for each analysis (Artemis, 2012): the Enhanced Frequency Domain Decomposition (EFDD) in the frequency domain and the Stochastic Subspace Identification (SSI) using Unweigthed Principal Components (UPC) in the time

domain. The estimated frequencies with the two methods for the tests a, b, c are reported in Table 1. The extreme repeatability of the first six estimated frequency is evident from Table 1. The corresponding estimated damping are reported in Table 2 and the EFDD method graphics for tests b and c are reported in Fig. 8 and 9.

Table 1 Identified frequencies with the two methods, tests a,b,c.

Frequency [Hz]	**EFDD test a**	**SSI test a**	**EFDD test b**	**SSI test b**	**EFDD test c**	**SSI test c**
1°	2.62	2.621	2.639	2.639	2.624	2.639
2°	2.834	2.829	2.842	2.842	2.829	2.831
3°	5.495	5.517	5.526	5.526	5.539	5.554
4°	7.061	7.036	7.053	7.053	7.029	7.061
5°	8.035	8.034	8.058	8.058	8.106	8.036
6°	-	11.31	11.29	11.29	11.23	11.29

Table 2 Identified damping values with the two methods, tests a,b,c.

Damping ratio [%]	**EFDD test a**	**SSI test a**	**EFDD test b**	**SSI test b**	**EFDD test c**	**SSI test c**
1°	0.926	0.868	1.327	1.39	1.365	1.343
2°	0.646	1.249	0.999	1.165	0.407	0.8767
3°	0.093	1.054	0.285	0.894	0.166	1.908
4°	1.598	1.621	1.695	2.105	1.052	2.889
5°	0.999	2.61	1.12	2.278	0.252	2.281
6°	0.926	1.947	0.189	1.843	0.041	2.667

The results shown in Table 1 were opportunely carried out considering all the experimental tests and demonstrating that, nevertheless the different casual events, the identified frequencies were consistent in all the cases.

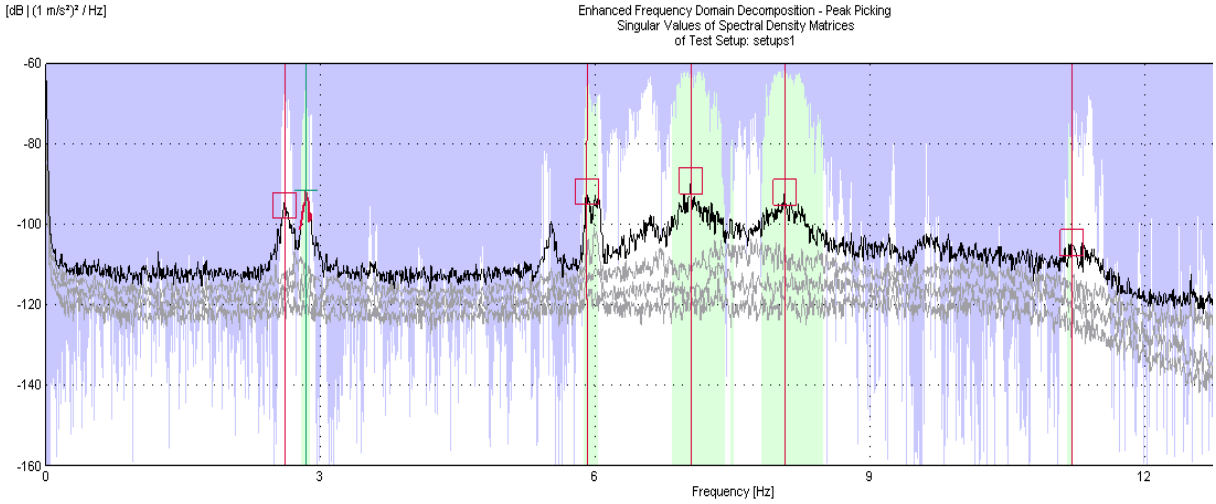


Fig.8 Identification by using EFDD, test b.

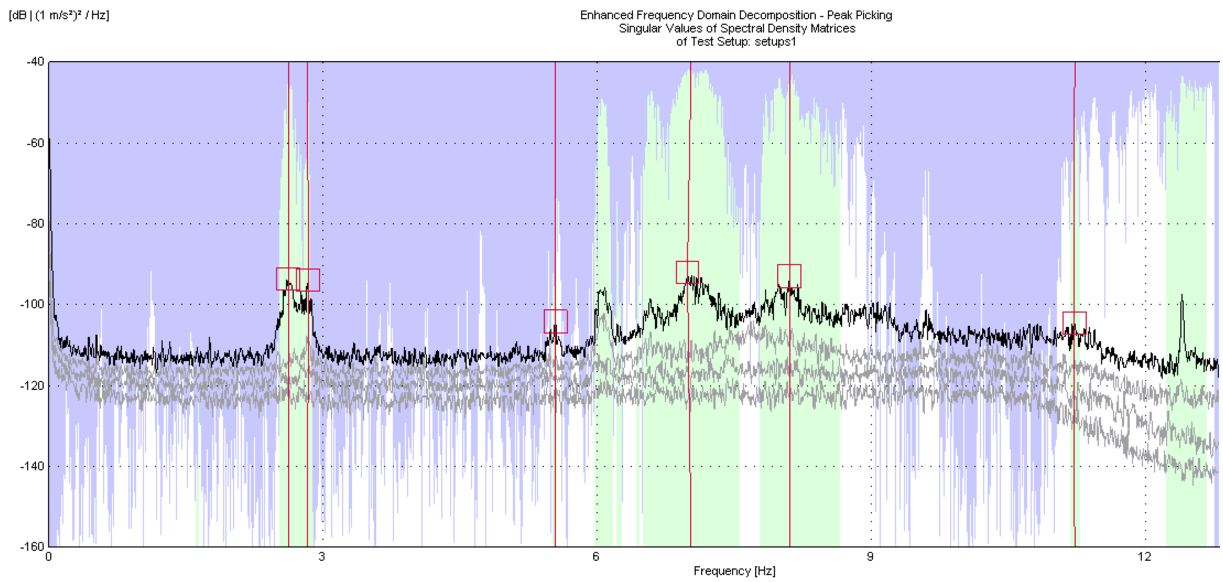
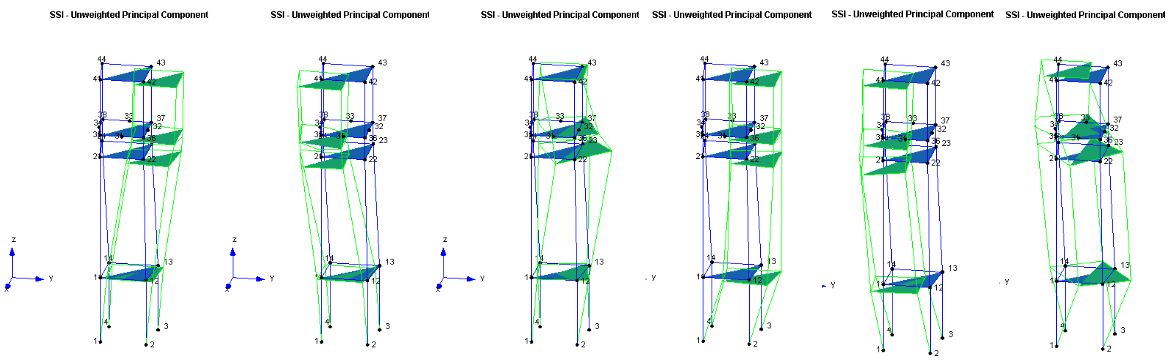


Fig.9 Identification by using EFDD, test c.

In detail the first and second frequency were identified as the first couple of flexional modes directed respectively on the *x* and *y* axis, the third frequency was the torsional mode, the fourth and fifth were the second couple of flexional modes directed on the *x* and *y* axis respectively, and the sixth is the second torsional mode.
A graphic of the experimental identified mode shapes is shown in Fig. 10 for the test a, considering the SSI method.



1st mode 2nd mode 3rd mode 4th mode 5th mode 6th mode

Fig.10 Identification of the mode shapes, SSI method, test a.

FINITE ELEMENTS MODEL

The finite element (FE) model of the Announziata tower has been realized considering the plant as perfectly squared (length 3.5 m). The structure is realized with brickwork walls and the three floor systems are made by full bricks supported by vaults that are based on the walls parallel to the N-S axis of the building.
The materials characteristics (characteristic strength in compression f_{wd}, module of elasticity E, weight per unit of volume γ_s and mass per unit of volume m_s) have been deduced by (Kouris, 2009) where the properties have been defined on the basis of Eurocode-6. In (Kouris,

2009) the pattern of cracking evident in vertical stone has taken into account considering a reduction of the module of elasticity of the masonry with cracking E and the initial $E_{initial}$ according with Eq. (1).

$$E = \frac{2}{3} \cdot E_{initial} \tag{1}$$

The considered material properties for the vertical masonry of the structure (material 1), the three solid brick floors of the campanile (material 2), and the stone column (material 3) are reported in Table 3.

Table 3 Material properties by Eurocode-6 before updating

Material	f_{wd} [MPa]	E[Mpa]	γ_s [kN/m^3]	m_s[kg/m^3]
1	0.87	1740	22	2243
2	0.66	1318	18	1835
3	n.a.	2600	18	1835

Finally for the vaults filling material it has been assumed γ_s equal to 10 kN/m^3. The three dimensional geometrical model of the building has been firstly realized in CAD environment (Fig. 11a), then converted in a FE software (SAP2000, Fig. 11b) opportunely inserting the material properties.

The model has two typologies of elements: the 'frame' and the 'shell'. The frame, prismatic linear elements, have been used for structural components such as the stone columns supporting the bell tower openings and the bells supporting framework (Fig. 12a) and they assume the corresponding materials properties.

The shell elements have been used for modeling the masonry walls, such as the vertical walls; for the vaults caps (Fig. 12b) specific shell elements with 4 nodes have been used for combining the membrane behavior with that of a flexible plate.

An adequate mesh was created in such a way to model the real behavior of the structural elements. The preliminary mesh was composed by 362 shell elements and 10 frames for a total of 404 nodes. In order to increase the realibility of the numerical model, the mesh was refined (Fig.12c) dividing opportunely the starting elements (the thickened model has 10541 elements and 10668 nodes).

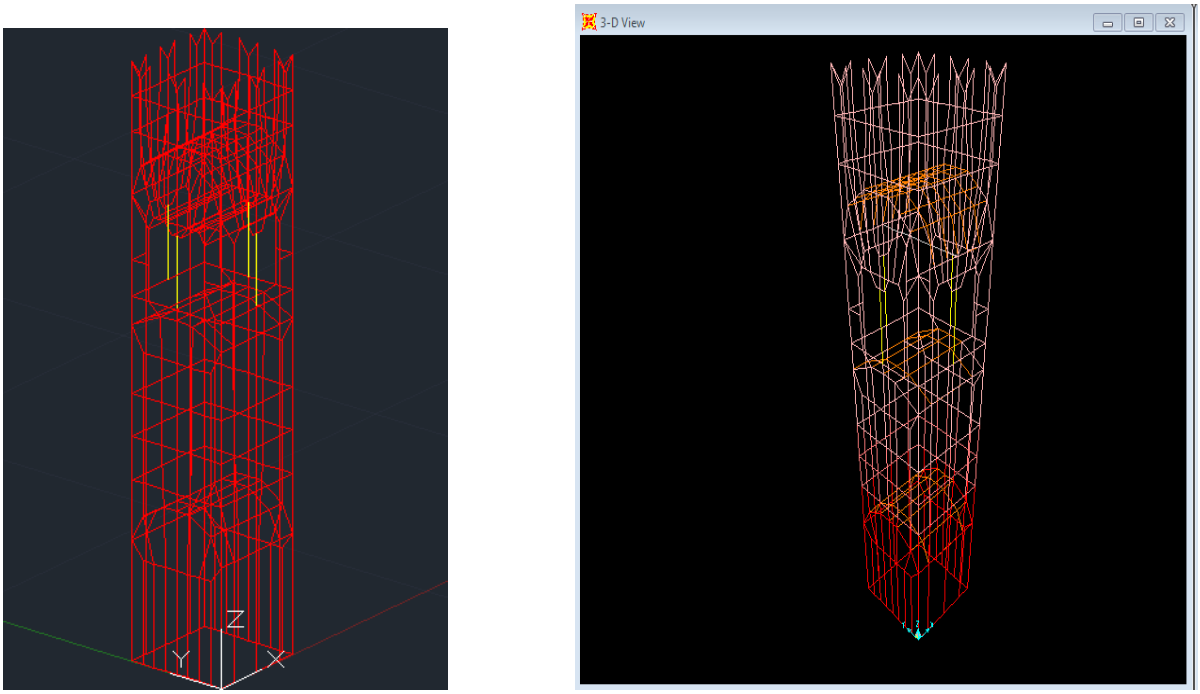


Fig.11: a) preliminary CAD model b) FE model

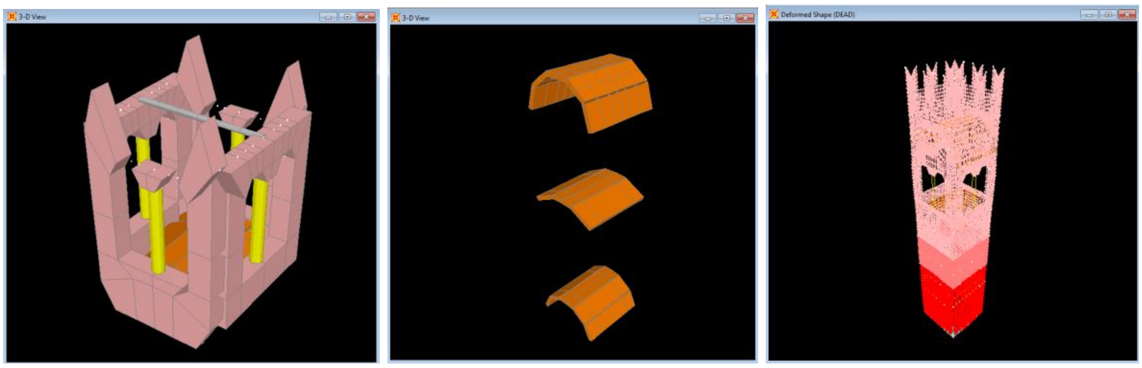


Fig.12: a) detail of the tower columns b)details of the valuts caps c)thickened FE model

The vaults filling material, that does not have a structural function, has been considered in the model as a load acting on the caps, while the bells have been modeled as added masses.

The weight P of the three bells, having different diameter D, has been estimated by Eq.(2) (Ivorra et al, 2002).

$$P = 580.8 \cdot D^{2.7839} \tag{2}$$

Considering no inclination signs on the tower, a joint constraint has been assumed on the basis. The adjacent buildings effects have also been considered in the model, introducing some joint constraints on the lateral walls connected to the adjacent units.
The dynamical analysis of the FE model gives the results shown in Table 4, referred to the first 7 frequencies; for each identified mode, Table 4 reported also the excited percentage

mass in the two principal direction *x* (U_x) and *y* (U_y) and all rotations around z axis (R_z) in such a way as to identify the typology of each mode.

Table 4 Modal frequencies and participating mass ratios of the FE model

Mode number	Frequency [Hz]	U_x	U_y	R_z
1	2.215	0.0003	0.37	0.13
2	2.393	0.33	0.0003	0.09
3	5.367	$6\cdot10^{(-8)}$	0.0012	0.15
4	8.901	$10^{(-6)}$	0.24	0.1
5	11.07	0.13	$1.1\cdot10^{(-9)}$	0.04
6	12.46	$4.8\cdot10^{(-7)}$	$6.5\cdot10^{(-5)}$	$1.7\cdot10^{(-5)}$
7	14.48	$1.9\cdot10^{(-6)}$	0.011	0.011

The participating mass ratios in Table 4 and the animation of the mode shapes clearly indicate that the first and the fourth mode are flexional on y direction, the second and the fifth are flexional on x direction, the third and the seventh are mainly torsional, while the sixth is a mode referred to a local movement on the z axis.
The model frequencies calculated in this work are closer to the identified ones with respect to the ones reported in (Kouris, 2009, even if the same material properties have been used. This is due to the major complexity and accuracy of the FE model (Fig. 12c) and to the attention to the architectural details. Anyway an updating procedure was also considered for improving the quality of the model and its adherence to the identified frequencies.
At this aim some passages were necessary to manage the SAP model with a specialized updating tool (DDS, 2012). It was necessary to convert the model in Ansys format (.cdb file) for avoiding importing errors. Moreover, after the conversion, the lateral joints constraints introduced to take into account the adjacent walls have been substituted by uniaxial springs (Fig. 13), whose stiffness (equal for all the elements) has been considered as an unknown parameter in the following updating procedure.

MODEL UPDATING

The parameters selected for the updating procedure are, besides the stiffness k of the springs, the Young's modulus E_i and the densities ρ_i of the three materials of Table 3. Considering that the different masonry thickness at different tower heights, respectively of 1, 0.55 and 0.65 m, could have an influence on the material properties, the material 1 in Table 3 has been divided in three groups (named 1,2,3) . The other two materials of Table 3 assumes the names 4 and 5 respectively. Figure 14 reported the material that have been used for the different zones of the tower. Finally, 11 parameters were considered for the model updating.

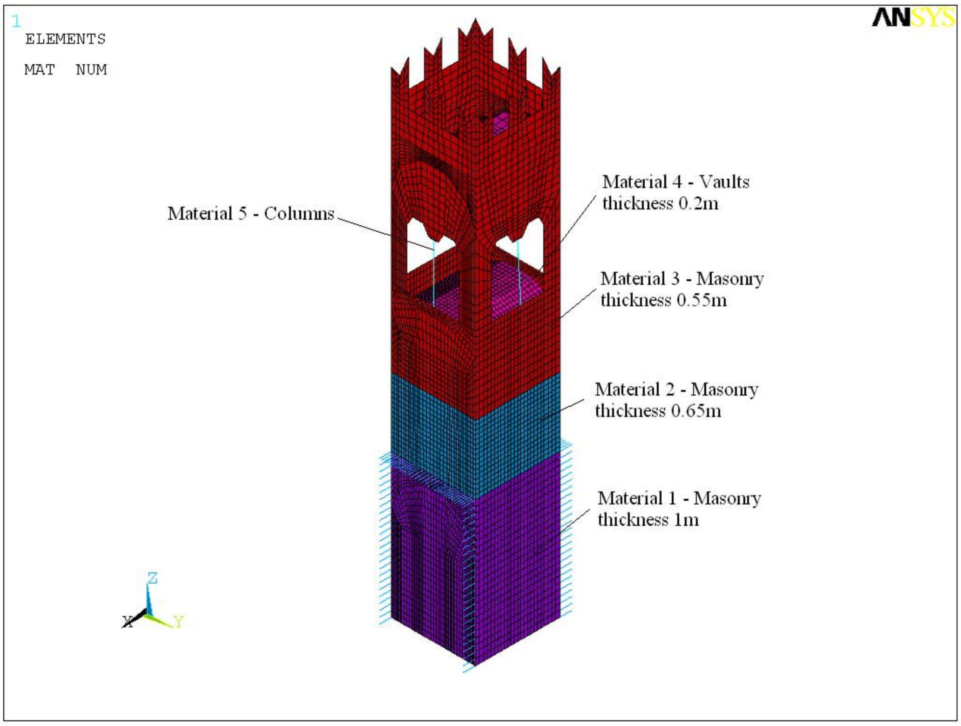


Fig. 14 Numerical model used for structural analysis: mesh and identification of material used

The method used for the normal modes evaluation in the 3D finite element model is the Lanczos subspace method (DDS,2012), while the parameters estimation was carried out iteratively minimizing the differences between theoretical and experimental natural frequencies. The strategy used for updating the 11 parameters previously indicated is the well-known Inverse Eigen-Sensitivity (Friswell,1995) In detail:

$$\mathbf{R}_e = \mathbf{R}_a + \mathbf{S} \cdot (\mathbf{P}_u - \mathbf{P}_0) \tag{3}$$

or in a short form:

$$\Delta\mathbf{R} = \mathbf{S} \cdot \Delta\mathbf{P} \tag{4}$$

where in (3):

$\mathbf{R}_e$ is the vector of the reference system responses (experimental data);

$\mathbf{R}_a$ is the vector of the predicted system responses for a given state $\mathbf{P}_0$ of the parameter values;

$\mathbf{P}_u$ is the vector of the updated parameter values;

$\mathbf{S}$ is the sensitivity matrix;

$\Delta\mathbf{R} = \mathbf{R}_e - \mathbf{R}_a$

$\Delta\mathbf{P} = \mathbf{P}_u - \mathbf{P}_0$.

Equation (3) is underdetermined and can be solved using a pseudo-inverse (least squares) method. In this case a weighted least squares or Bayesian technique has been used. The applied least squares solution (DDS,2012) will minimize iteratively the residue defined as:

$$residue = \mathbf{S} \cdot \left[\Delta \mathbf{P}_{n+1} - \Delta \mathbf{R} \right] \tag{5}$$

with $\Delta \mathbf{P}_{n+1}$ calculated as follows:

$$\Delta \mathbf{P}_{n+1} = [\mathbf{S}^T \cdot (\mathbf{S}^T \cdot \mathbf{S})^{-1}] \cdot \Delta \mathbf{R} \tag{6}$$

The previous procedure was applied to achieve the fixed convergence in a limited number of iterations, limiting the variation range of all the 11 parameters considered in comparison with the starting ones.

The parameters values before and after the updating procedure are shown in Table 5 with also indicated the parameter changes for the updating. A maximum range of variation [-90% 200%] was imposed to all the parameters.

It is interesting to observe that the properties of the vertical masonry of the structure change considerably respect to the values assumed on the basis of Eurocode-6; the new estimated parameters probably will give in the future researches useful information about the real damage state of the tower.

Table 5 Parameters before and after the updating procedure

Element groups	ρ [kg/m³] initial	ρ [kg/m³] final	Variation [%]	E[MPa] initial	E[MPa] final	Variation [%]
1	2243	6730.1	200	1740	1639.1	-6.5
2	2243	869.1	-61.2	1740	645.5	-62.9
3	2243	224.3	-90	1740	5220	200
4	1835	224	-87.8	1318	1237	-6.1
5	1835	1729.4	-5.7	2600	2414	-7.1
Springs	Stiffness initial [N/m]	Stiffness final [N/m]	Variation [%]			
	$1 \cdot 10^{(6)}$	$2.224 \cdot 10^{(6)}$	122			

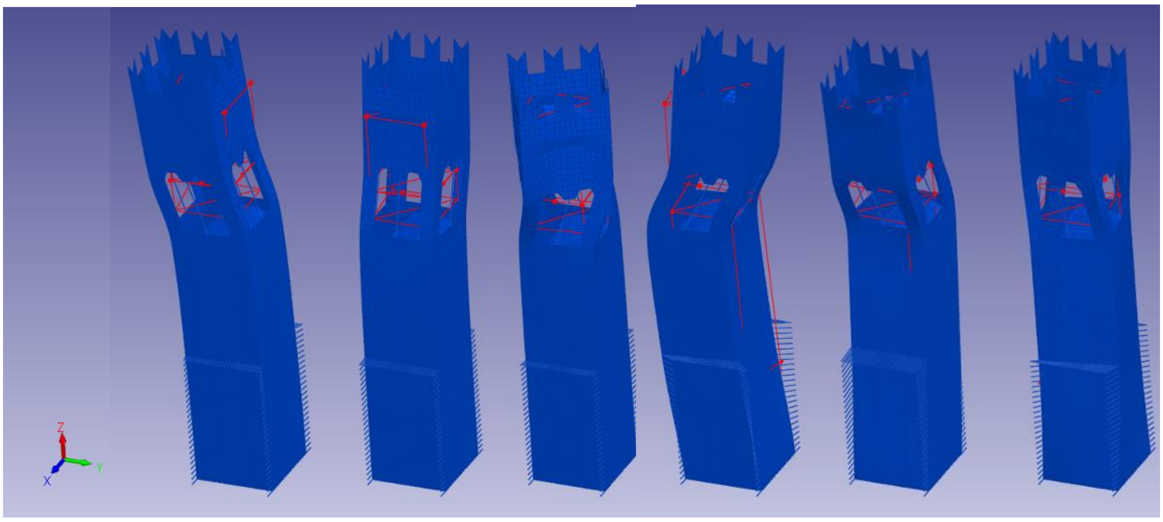
The comparison of the experimental data with the updated FE model is shown in Table 6 in which the first six estimated experimental frequencies, the first six frequencies of the updated model, the percentage error and the MAC coefficients between theoretical and experimental frequencies are shown. It is possible to note that the model natural frequencies are very close to the experimental ones and the correlation (MAC) between mode shapes shows a good agreement especially for the bending mode shapes.

The updating has also changed the model sixth frequency that was not anymore the local mode in the z direction but a torsional mode similar to the one identified experimentally (Table 6).

Table 6 Comparison between theoretical and experimental frequencies after the updating.

Frequency number	Theoretical frequency [Hz]]	Experimental frequency [Hz]	Percentage error [%]	MAC[%]
1	2.6541	2.6332	0.79	56.5
2	2.7884	2.8309	-1.50	39.3
3	5.5677	5.5721	1.12	6.5
4	6.8895	7.0265	-1.95	10.4
5	8.1261	8.0241	1.27	26.8
6	11.826	11.272	4.92	26.6

Moreover, in Figure 15 the comparison between the first six experimental and theoretical mode shapes are shown: the overlapping is evident and the correlation is demonstrated for all the modes. There is to consider that the upper part of the tower was not instrumented and so the experimental reconstructed mode can not be perfectly defined but have been traced with a linear interpoltaion respect to the instrumented nodes. Anyway the result is satisfactory and it encourages the possibility of using the validated model for simulations regarding the tower vulnerability.



1st mode 2nd mode 3rd mode 4th mode 5th mode 6th mode

Fig. 15 Comparison between the first six theoretical (blue) and experimental (red) mode shapes

CONCLUSION

The analysis carried out in this paper shows that the influence of material data assumed for the calculation is relevant especially in presence of an important damage state of the building. Therefore, a considerable effort is required in defining a reliable model that could efficiently describe the identified experimental dynamical properties. The main goal of this research thematic is the definition of the applicability limit of the non-destructive strategies.

The building analyzed in the present study presented many difficulties related to the quite squat profile, to the real damaged state, to the difficulties of achieving the higher floors of the tower and to the present of heavy external disturbances in environmental conditions. Anyway the modal identification approached with two different statistical approaches in different domains has permitted to evaluate with a certain security the structure frequencies.

The final results carried out after the updating procedure may be considered very good, and the validated model shows the same dynamical characteristics of the experimental tests. The material data values estimated in this way will constitute an important reference for the evaluation of the state of the building and for the planning of eventual renovations works.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the funding by European Union and by National Funds of Greece and Italy, European Territorial Cooperation Programme "Greece-Italy 2007-2013", under grants of the project Structural Monitoring of ARTistic and historical BUIlding Testimonies (S.M.ART. BUIL.T.).

REFERENCES

ARTeMIS Extractor Pro software. Issued by Structural Vibration Solutions ApS. NOVI Science Park, Niels Jernes Vej 10, DK 9220 Aalborg East, Denmark; 2012.

Diaferio M., Foti D., Sepe V. 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, 2007; Malta.

Dynamic Design Solution NV (DDS). 'Fem Tools Model Updating Theoretical Manual, vers.3.6'. Dynamic Design Solution NV (DDS) Interleuvenlaan 64 – 3001 – Leuven – Belgium; 2012.

Foti D., Diaferio M., Mongelli M., Giannoccaro N.I., Andersen P. Operational Modal Analysis of a Historical Tower in Bari, Proceedings of the Society for Experimental Mechanics Series, "IMAC XXIX", Jacksonville, Florida, USA, 2011; 7, p. 335-342.

Foti D., Diaferio M., Giannoccaro N.I. & Mongelli M. Ambient vibration testing, dynamic identification and model updating of a historic tower, NDT & E International, 2012; 47, p. 88-95.

Friswell MI, Mottershead JE. Finite element model updating in structural dynamics. The Netherlands, Dordrecht, Kluver Academic; 1995.

Gentile C, Saisi A. Ambient vibration testing of historic masonry towers for structural identification and damage assessment. Constr Build Mater 2007; 21(6), p. 1311–21.

Gentile C, Saisi A. Ambient vibration testing of historic masonry towers for structural identification and damage assessment. Constr Build Mater 2007; 21(6), p. 1311–21.

Kouris S.S., Karaveziroglou-Weber M. Dynamic investigation and strengthening on mansonry bell tower. Protection of Historical Buildings, PROHITECH 2009, p. 941-947.

Ivorra S, Llop y Bayo S. Determinacion de algunas magnitudes fisicas caracteristicas de una campana. XIV Congreso de Conservación y Restauración de bienes culturales. Valladolid-2002, p. 1-12.

Ivorra S, Pallare´ s FJ. Dynamic investigations on a masonry bell tower. Eng Struct 2006; 28(5), p. 660–7.

Lepidi M, Gattulli V, Foti D. Swinging-bell resonances and their cancellation identified by dynamical testing in a modern bell tower. Eng Struc, Elsevier 2009; 31 (7):p. 1486–1500.

Trombetti T., Silvestri S., Gasparini G., Palermo M., Dallavalle G. Monitoring the structural health of the “Due Torri” in Bologna. Proceedings of the XX Congresso dell'Associazione Italiana di Meccanica Teorica e Applicata Bologna AIMETA, 12-15 settembre 2011, Bologna (Italy).