Modal parameters identification on environmental tests of an ancient tower and validation of its FE model

M. Diaferio, D. Foti, and N. I. Giannoccaro

Abstract— An accurate knowledge of the dynamical parameters of structures is definitely useful for seismic assessment and for the 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 (Corfù, Greece), a masonry tower, which shows a high damaged scenario and, consequently, a high vulnerability to dynamic and seismic forces. The paper presents the experimental investigations and the operational modal analysis results, useful for defining the finite element (FE) model of the tower. The monitoring system consists of several elements properly connected. The positioning of the instrumentation has been conditioned by many operative problems due to the limited accessibility of the structure, not only to the main access but also to reach the top. This difficulty and the presence of a higher level connected only with very slender columns in correspondence of the arched-windows have affected the finite element model that cannot act as a non-linear structure for the local modes at this level. However, it has been possible to identify with a certain confidence the first six frequencies of the tower and their corresponding mode shapes.

Keywords— Non-destructive tests, operational modal analysis, non-linear FE model, dynamic analysis, model updating.

I. Introduction

IN the preservation of architectural heritages it is important a careful study of the structure and the dynamic characteristics in order to describe its actual behavior [1-3].

The necessity of identifying unknown geometrical data and material properties is due to the usual impossibility of conducting classical tests for their evaluation; so the numerical models may be validated only by means of non-destructive techniques. In literature, various methods have been proposed for estimating the mechanical properties of the structural materials also in the presence of retrofitting interventions by means of innovative materials [4-9]. However, these procedures allow to evaluate only local properties and the extension to the global structure is too much burdensome for practical applications. Other approaches make use of the experimental evaluation of the modal parameters of the structure. The modal parameters can be compared with those obtained from a preliminary model so that the unknown materials and the geometrical parameters may be estimated for obtaining an accurate finite element (FE) model. These techniques have the advantage of providing information on the overall structure with a reasonable experimental effort [10-32].

In this paper the bell tower of Announziata Church has been investigated. It is a masonry tower, which shows a high damaged scenario (Fig. 1). It is located in the center of the town of Corfù in Greece. The Announziata Church was built in 1394 and was part of a Roman monastery. Inside the church [1] there were the tombs of the 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; then it 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 has survived to the present day with heavy cracking. The bell tower is a stone tower with an almost square plan section. A double arched-window supported by a stone column in the center is located at each of the four façades (Fig. 1). It should be emphasized the problems connected to the modeling of this part of the tower due to difficulties of instrumenting it as better explained in the following sections. In addition, the slenderness of the columns of the arched-windows cannot be modelled as linear elements and this peculiarity must be taken into account in the analysis of the results when compared to the preliminary model.

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Fig. 1. Announziata bell tower.

II. Geometrical Description

The bell tower is a stone tower with an almost square transversal section, with a side of about 3.5 m. The tower is about 20 m tall and has a belfry on the top. A double arch supported by a stone column in the center is present on each of the four sides.

The structure of the tower consists of masonry walls; three floors made by solid bricks are supported by vaults. At the top of the tower there is the bell cell with four bells (Fig.2). The actual state of the tower may be defined critical, with many damages and cracking. It is possible to observe deep vertical and diagonal cracking, deterioration of mortar on the stonewalls and development of creepers in the masonry (Fig.3).

Fig. 2. The four bells of the tower.

The possibility of defining a detailed Finite Element (FE) model of the tower taking into account the experimental modal identification data may be considered very important for getting information about the structural health of the tower. To this aim, the present paper shows all the details and results of the experimental modal identification procedure able to complete the preliminary analysis presented in [32] and also an updating procedure that will match the FE model with the experimental data nevertheless the peculiarities due to the presence of the arched-windows at a high level of the tower.

Fig.3. The damaged state of Announziata bell tower.

III. Experimental Setup and Identification Procedure

Announziata tower was instrumented with 24 high sensitivity seismic accelerometers ICP PCB 393B31 placed on 12 positions (labelled with numbers 1-12 on the transversal sections in Fig.4) on three different levels and oriented according to the orthogonal directions x and y. Eight accelerometers were placed on the four corners of the first floor (positions 1-4), eight were placed on the four corners of the second floor (numbers 5-8), and eight were placed on the basis of the four columns (numbers 9-12).

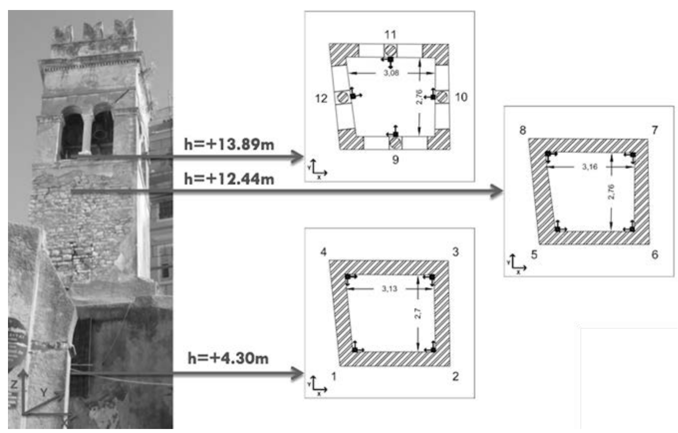


Fig. 4. Announziata bell tower and layout of the experimental setup.

Appropriate rectangular blocks were designed and realized 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 realized on the perpendicular faces of the block (Fig.5).

Fig.5.Two monoaxial accelerometers positioned at the base of a column of an arched-window of Announziata bell tower.

It was not possible to achieve the belfry of the campanile to place the accelerometers on the top of the tower; also, it must be considered the compromised situation of the building, let now fall into decay. In addition, it was very difficult to reach the first floor, because there was no stair connecting it to the ground floor. A provisional one was utilized to get inside the tower and positioning the accelerometers (Fig.6).

The bell tower stands in the city centre, in a crowded area especially during the daytime. There is a main entrance road open to the car passage and a rotatory very close to the tower where cars and trucks can select the preferential way (Fig.7). The continuous passage of bikes, cars, motorcars and pedestrians characterized all the environmental tests.

Fig.6. Entrance to Announziata bell tower.

Preliminary tests were carried out on the 12th of October 2012; in the following day several consecutive tests were conducted. 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. About ten consecutive acquisitions were carried out and, in each acquisition, all the relevant events (passage of cars and motorbike, other possible disturbs), were noted. A heavy rain, unfortunately, occurred in the night between the 12th and 13th of October, and infiltration of water was verified on the instrumentation that was placed at the 2nd floor and at the basis of the columns.

Fig. 7. Position of the Announziata bell tower.

IV. IDENTIFICATION RESULTS

The analysis of the experimental results was performed soon after the tests. A specific software [33] was used for the extraction of the modal parameters from ambient vibration data. Fig. 8 shows the model defined by means of this software referred to the *xyz* system.

A preliminary analysis was conducted on the time series recorded by the accelerometers for evaluating the effects of the urban traffic and the functionality of the accelerometers considering the difficult environmental conditions.

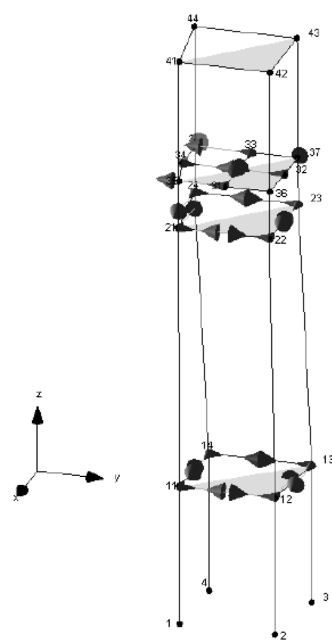


Fig. 8 Reference system and model for the identification.

The preliminary analysis allowed to individuate that accelerometers in positions 23 (both directions *x* and *y*), 24 (direction *x*) and 32 (direction *y*) were not performing properly (see the model in Fig.8). In addition, the preliminary analysis allowed to clearly highlight the effects of external disturbances.

Figs. 9-11 show the time histories of three opportunely selected tests (named *a*, *b*, *c*) for accelerometers in positions 21 and 22 of Fig.8 (both the directions *x* and *y*). It is evident from Figs. 5-7 the extreme sensibility of the experimental setup to external events: for test *a* (Fig.9), there was a heavy and continuous traffic after 260-280 seconds and some spurious events after about 440, 540 and 560 seconds from the starting time. For test *b* (Fig.10) there are not evident effects on the time histories, while on test *c* (Fig.11) heavy traffic effects are evident at the beginning of the registration (80-120s) and at the end of the registration (400-480 s) for the accelerometers along *x* axis (left column of Fig.11), while some spurious events are evident after around 510 s. The time-histories of the selected tests seem to be very different each other depending on the external disturbances that influence the accelerometers data.

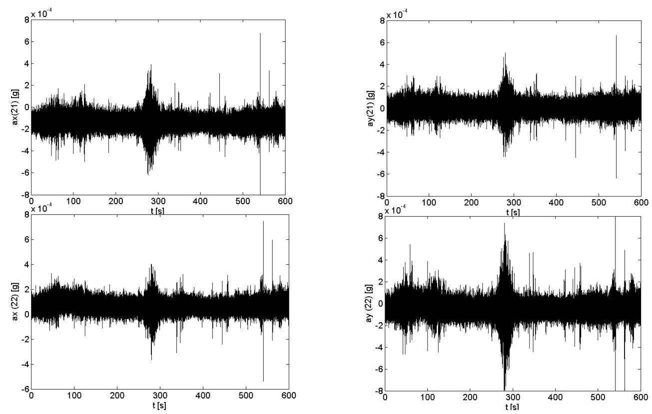


Fig.9. Acquisition signals at points 21 and 22 for test *a*.

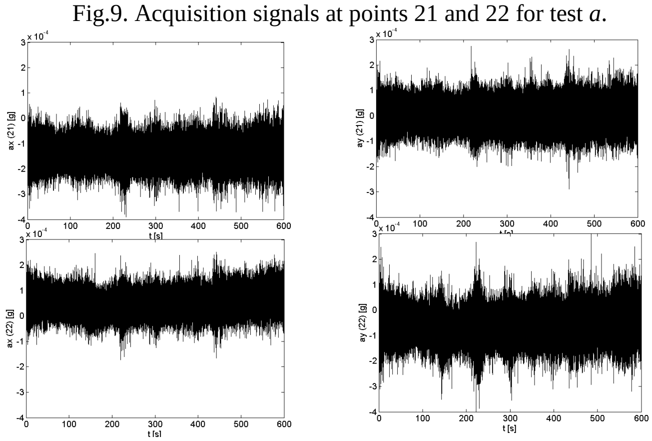


Fig.10. Acquisition signals at points 21 and 22 for test *b*.

It is important to emphasize that the data results are closely connected to external events detected during the acquisitions, as demonstrated by the preliminary analysis conducted on the time signals of the accelerometers [32]. For this reason, a wide experimental analysis was carried out repeating the acquisition for 14 times in two consecutive days.

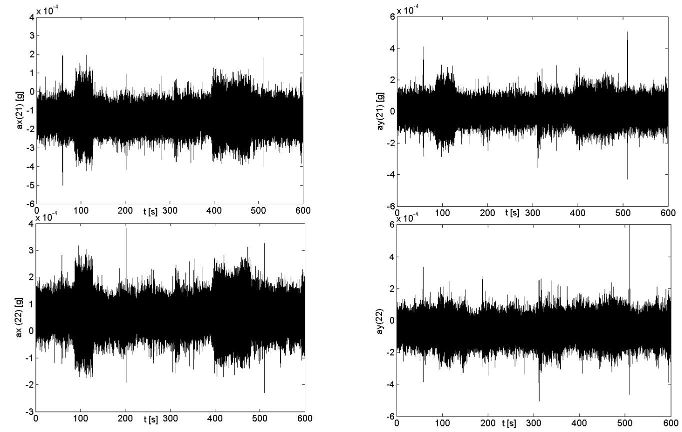


Fig.11. Acquisition signals at points 21 and 22 for test *c*.

The frequencies of the building and the modal shapes time-histories were identified for all the tests with a simple operational modal analysis (OMA). In particular, for each analysis [33] two different OMA methods were used: 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 [34-35]. The estimated frequencies for all the tests are named using the number of the tests' day (first number 1 or 2 considering the first or second day) and the number of acquisition (second number 1, 2…5 for the first day, 1, 2…9 for the second day). Table 1 shows the frequencies evaluated by means of SSI method. The identified frequencies are consistent for all the tests and for both methods. SSI method allows evaluating all the frequencies at each acquisition, while EFDD method in some cases is not able to identify high order frequencies.

Table 1. Identified frequencies [Hz] with SSI method for all the tests.

Fr	1-1	1-2	1-3	1-4	1-5	2-1	2-2	2-3	2-4	2-5	2-6	2-7	2-8	2-9
1	2.61	2.61	2.61	2.60	2.61	2.63	2.62	2.62	2.63	2.62	2.62	2.63	2.65	2.63
2	2.83	2.83	2.82	2.81	2.82	2.82	2.82	2.84	2.83	2.82	2.82	2.84	2.84	2.83
3	5.50	5.47	5.48	-	5.46	5.52	5.44	5.51	5.51	5.50	5.51	5.52	5.55	5.55
4	7.04	7.05	7.05	7.05	7.05	7.05	7.04	7.02	7.03	7.07	7.03	7.05	7.07	7.06
5	8.04	8.01	8.03	7.99	8.021	8.01	8.01	8.01	8.04	8.05	8.03	8.05	8.0	8.03
6	11.28	11.3	11.27	11.14	11.19	11.32	11.25	11.28	11.3	11.37	11.31	11.29	11.32	11.29

Table 2. Statistical analysis of the identified frequencies considering all the 14 performed tests.

Frequency number	Mean value [Hz] (SSI)	Standard deviation (SSI)	Mean value [Hz] (EFDD)	Standard deviation (EFDD)
1	2.625	0.0125	2.616	0.0157
2	2.832	0.0078	2.828	0.0071
3	5.505	0.0325	5.526	0.0220
4	7.052	0.0152	7.067	0.0526
5	8.027	0.0191	8.051	0.0497
6	11.280	0.0571	11.323	0.1137

In Table 2, a statistical analysis of the identification results is shown. It can be observed that the mean values are almost the same for both methods and the standard deviation is very low ensuring a good repeatability and consistence of the identified frequencies nevertheless the different casual events. The frequencies identified by means of the mean value may be considered a stable experimental benchmark that characterizes the dynamical behavior of the tower.

Figs. 12 and 13 show the identification diagrams using SSI and EFDD methods, respectively, for test 2-4. The identified frequencies were analyzed for detecting the mode shapes. It was observed that in all the tests of Table 1, the six identified mode shapes perfectly correspond to the ones expected for the tower. In detail, the 1st and 2nd frequencies were identified as the first couple of flexional modes on the *y* and *x* axes, respectively, the 3rd frequency was the torsional mode, the 4th and 5th frequencies were the second couple of flexional modes on the *y* and *x* axes, respectively, and the 6th frequency was the second torsional mode. Fig.14 shows graphically the experimental identified mode shapes for test 2-4 by mean of SSI method.

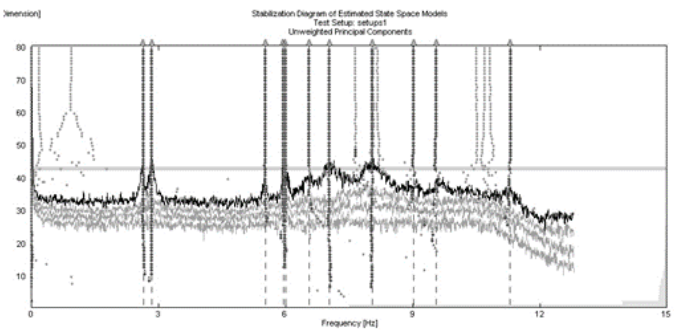


Fig.12. Identification by using SSI for test 2-4.

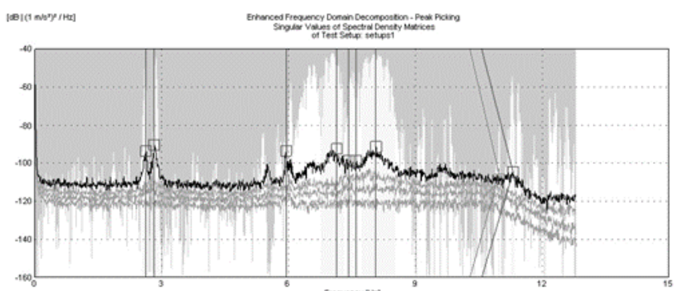
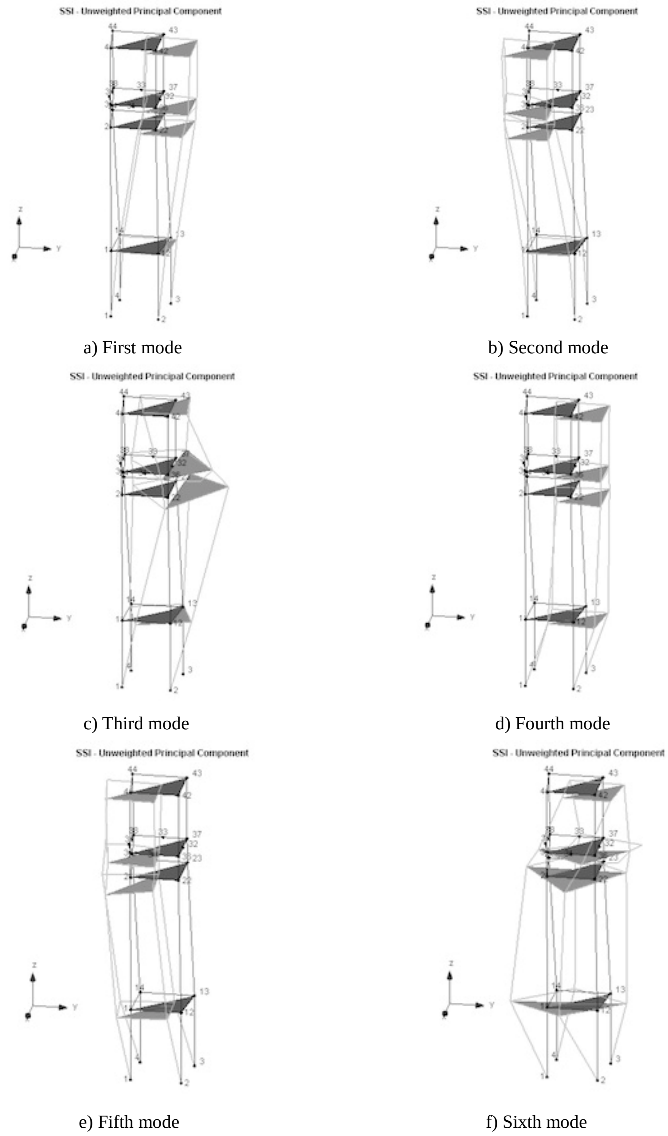


Fig.13. Identification by using EFDD for test 2-4.



a) First mode b) Second mode

c) Third mode d) Fourth mode

e) Fifth mode f) Sixth mode

Fig.14. Mode shapes identification by using SSI method for test 2-4.

V. Finite Element Model

A preliminary finite element (FE) model of Announziata tower has been developed considering a perfectly square plan section. The structure is made of brickwork walls and the three floor systems are made by full bricks supported by vaults that, in turn, are supported by the walls parallel to the N-S axis of the building.

The characteristics of the materials (characteristic strength in compression f_{wd}, elastic modulus E, weight per unit volume γ_s and mass per unit of volume m_s) have been deduced from [1] where the properties have been defined on the basis of Eurocode 6 [36]. In [1] the pattern of the vertical cracking has been taken into account considering a reduction of the elastic modulus of the masonry. In detail, it has been adopted the following elastic modulus for the cracked masonry:

$$E = \frac{2}{3} E_{initial} \qquad (1)$$

where $E_{initial}$ is the elastic modulus of the undamaged masonry. The considered material properties for the vertical walls of the tower (material 1), for the three solid brick floors of the campanile (material 2), and for the stone column (material 3) are reported in Table 3. Finally for the filling material of the vaults it has been assumed γ_s =10 kN/m^3.

The three dimensional FE model of the tower has been defined by means of the SAP2000 [37]. The model has two typologies of elements, 'frame' and 'shell'. Frame elements are prismatic linear ones used to model structural components such as the stone columns supporting the arched-window of the bell tower and the bells' supporting framework (Fig. 15a). Shell elements have been used for modeling the masonry walls, such as the vertical walls; for the vaults (Fig. 15b) specific shell elements with 4 nodes have been used for combining the membrane behavior with that of a flexible plate.

Table 3. Material properties by Eurocode-6

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

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 frame elements for a total of 404 nodes. In order to increase the reliability of the numerical model, the mesh was refined (Fig.15c) dividing opportunely the elements (the thickened model has 10541 elements and 10668 nodes).

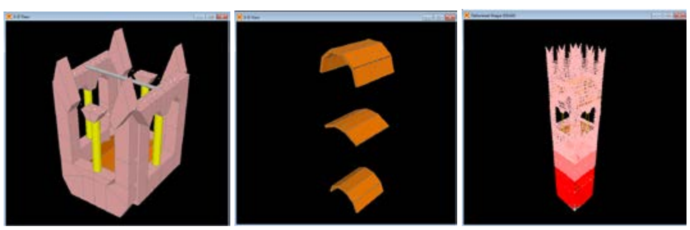


Fig.15 a) detail of the columns b) details of the vaults c) FE model.

The filling material of the vaults, 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 obtained from Eq.(2) [38]:

$$P = 580.8\, D^{2.7839} \qquad (2)$$

The tower has been modelled as fixed at the base because no rocking sign of the foundation of the tower has been detected. 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 starting dynamical analysis of the FE model gives the results shown in Table 4, referred to the first seven frequencies of the model.

For each identified mode, Table 4 also reports the excited percentage mass in the two principal direction x (U_x) and y (U_y) and the 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	6·10(-8)	0.0012	0.15
4	8.901	10(-6)	0.24	0.1
5	11.07	0.13	1.1·10(-9)	0.04
6	12.46	4.8·10(-7)	6.5·10(-5)	1.7·10(-5)
7	14.48	1.9·10(-6)	0.011	0.011

The participating mass ratios in Table 4 and the animation of the mode shapes clearly indicate that the 1st and 4th modes of the model are flexional in y direction, the 2nd and 5th modes are flexional in x direction, the 3rd and 7th modes are mainly torsional, while the 6th is a mode referred to a local movement on z axis.

The model frequencies calculated in this study are closer to the identified ones with respect to the ones reported in [1], even if the same material properties have been utilized. This is due to the major complexity and accuracy of the FE model and the careful observation of the architectural details (Fig. 11c). However, an updating procedure has also been considered for improving the quality of the model and its adherence to the identified frequencies.

In the forthcoming part of the research the model has been updated in order to match the identification results [39], the main objective of this study being the characterization of the behavior of the cracked masonry and the interaction between the tower and the surrounding buildings.

VI. Model Updating

The parameters selected for the updating procedure are the stiffness k of the springs, the Young's modulus E_i and the densities ρ_i of the three materials reported in Table 3. Considering that the different masonry thickness at different tower heights could have an influence on the material properties, Material 1 in Table 3 has been divided in three groups (named 1, 2, 3). As described in Fig. 16, Material 1 referred to the bottom up to a height of about 4.2 m, Material 2 from 4.2 m to about 8 m, Material 3 from 8 m to the upper part of the building; the structural walls corresponding to the three materials have a thickness of 1 m, 0.65 m and 0.55 m, respectively. The other two materials of Table 3 assume the names Material 4 and Material 5, respectively. Fig.16 shows the materials that have been used for the different zones of the tower. Finally, eleven parameters were considered for the model updating.

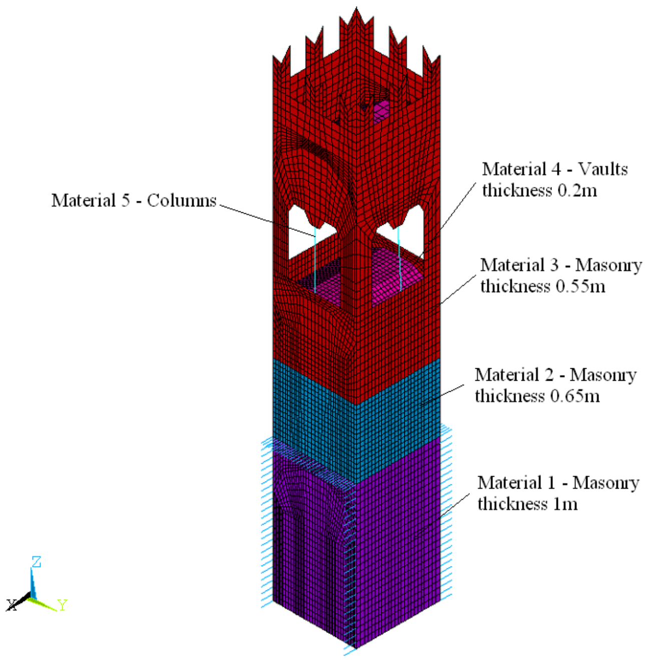


Fig.16. Numerical model used for the structural analysis: mesh and identification of material used.

The method used for the normal modes evaluation in the 3D finite element model is the Lanczos subspace method [39], while the parameters estimation was carried out iteratively minimizing the differences between the theoretical and experimental natural frequencies. The strategy used for updating the eleven parameters previously indicated is the well-known Inverse Eigen-Sensitivity [40].

In detail:

$$R_e = R_a + S \cdot (P_u - P_o) \tag{3}$$

or in a short form:

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

where in (3):

R_e is the vector of the reference system responses (experimental data);

R_a is the vector of the predicted system responses for a given state P_o of the parameter values;

P_u is the vector of the updated parameter values;

S is the sensitivity matrix;

$\Delta R = R_e - R_a$;

$\Delta P = P_u - P_o$.

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 [39] will minimize iteratively the residue defined as:

$$residue = S \cdot [\Delta P_{n+1} - \Delta R] \tag{5}$$

with ΔP_{n+1} calculated as follows (6):

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

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

The parameters values before and after the updating procedure are shown in Tables 5 (for the densities), 6 (for the Young modules) and 7 (stiffness), with also indicated the parameter variation due to 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 walls of the structure change considerably respect to the values assumed on the basis of Eurocode 6. The new estimated parameters in future researches will probably give useful information about the real damage state of the tower.

Table 5. Parameters before and after the updating procedure.

Element groups	ρ [kg/m^3] initial	ρ [kg/m^3] final	Variation [%]
1	2243	6730.1	200
2	2243	869.1	-61.2
3	2243	224.3	-90
4	1835	224	-87.8
5	1835	1729.4	-5.7

Table 6. Parameters before and after the updating procedure.

Element groups	E[MPa] initial	E[MPa] final	Variation [%]
1	1740	1639.1	-6.5
2	1740	645.5	-62.9
3	1740	5220	200
4	1318	1237	-6.1
5	2600	2414	-7.1

Table 7. Parameters before and after the updating procedure.

Stiffness initial [N/m]	Stiffness final [N/m]	Variation [%]
10^6	$2.224 \cdot 10^6$	122

The comparison of the experimental data with the updated FE model is shown in Table 8. In the Table 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 natural frequencies of the model are very close to the experimental ones and the correlation (MAC) between mode shapes shows a good agreement especially for the bending mode shapes. Considering the impossibility of measuring the tower all over its height, the authors evaluated the MAC coefficients considering only the lower part of the tower (until the 2nd floor) corresponding to nodes [1-4;11-14;21-24] of Fig. 4. This choice is also justified from the particular structure of the analyzed tower that has a lower part (the instrumented one) connected to the upper part only through a few slender and thin columns, letting foresee the possibility of local modes regarding the lower and the upper parts completely independent from each other. The results collected in Table 8 clearly show that the validated model, after the updating procedure, has a dynamical behavior really close to the experimental data; the torsional mode (3rd mode) has a MAC lower than in the other modes, but this is connected to the many operative problems previously underlined.

The updating has also changed the 6th frequency of the model that was not anymore the local mode in the *z* direction (before the updating procedure) but a torsional mode similar to the one identified experimentally (Table 8).

Table 8. Comparison between theoretical and experimental frequencies after the updating.

Frequency number	Theoretical frequency [Hz]	Theoretical frequency [Hz]	Percentage error [%]	MAC [%]
1	2.6541	2.6332	0.79	76.4
2	2.7884	2.8309	-1.50	69.9
3	5.5677	5.5721	1.12	40.8
4	6.8895	7.0265	-1.95	71.8
5	8.1261	8.0241	1.27	74.9
6	11.826	11.272	4.92	71.9

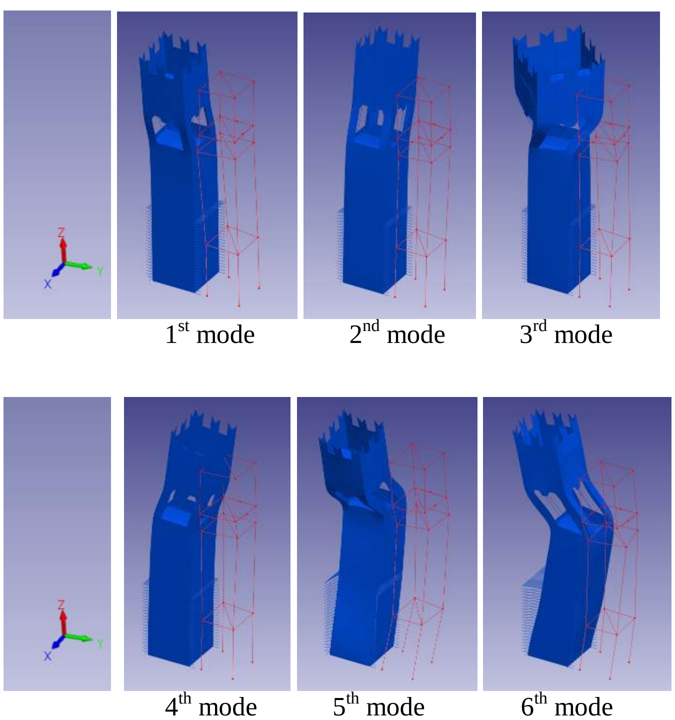


Fig. 17 Comparison between the first six theoretical (blue) and experimental (red) mode shapes.

Moreover, in Fig. 17 the comparison between the first six experimental and theoretical mode shapes is shown: the overlapping is evident and the correlation is demonstrated for all the modes. An offset has been added to the two models in Fig. 17, in order to graphically distinguish them. It has to be considered that the upper part of the tower was not instrumented and, therefore, the experimental reconstructed modes cannot be perfectly defined but have been traced with a linear interpolation 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.

VII. UNITS

Use either SI (MKS) or CGS as primary units. (SI units are strongly encouraged.) English units may be used as secondary units (in parentheses). **This applies to papers in data storage.** For example, write “15 Gb/cm^2 (100 Gb/in^2).” An exception is when English units are used as identifiers in trade, such as “3½ in disk drive.” Avoid combining SI and CGS units, such as current in amperes and magnetic field in oersteds. This often leads to confusion because equations do not balance dimensionally. If you must use mixed units, clearly state the units for each quantity in an equation.

The SI unit for magnetic field strength H is A/m. However, if you wish to use units of T, either refer to magnetic flux density B or magnetic field strength symbolized as $\mu_0 H$. Use the center dot to separate compound units, e.g., “A·m^2.”

VIII. CONCLUSION

The dynamic identification of Announziata bell tower presented in this study is characterized by many difficulties especially for the high damaged state of the tower, the impossibility to achieve the higher floors of the structure and the presence of heavy external disturbances in environmental conditions. The tower is a historical heritage of Corfu; so it is very important to apply a non-destructive strategy to identify its dynamic behavior.

The analyzed building presented many difficulties related to the real damaged state, the difficulties of achieving the higher floors of the tower and the presence of heavy external disturbances in environmental conditions. Anyway, the modal identification performed with two different statistical approaches in different domains has allowed to evaluate with a certain confidence the frequencies of the structure and its mode shapes. The statistical analysis performed on the identified frequencies shows the extreme repeatability and consistency of the first six estimated frequencies with respect to all the tests and the two adopted methods. This demonstrates that, nevertheless the different casual events, the identified frequencies were persistent in all the cases.

The analysis carried out in this paper shows also that the material data assumed for the calculus highly influence the results especially in presence of an important damaged 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 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.

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