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Procedia Structural Integrity 44 (2023) 782–789

Structural Integrity Procedia

www.elsevier.com/locate/procedia

XIX ANIDIS Conference, Seismic Engineering in Italy

Dynamic identification of a strategic building of the sixties with a mixed structure

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Abstract

The present paper shows and discusses the results of the identification procedure applied to the building of the Municipality of Castellaneta, Taranto (Puglia, Italy). The case study has been chosen for its structural complexity; indeed, the building has been built in two successive phases, block A built between 1955 and 1957, and block B, for completion, presumably built between the years 1960 and 1961.
Block A constitutes the main building and represents the original nucleus. The structure was subsequently subject to structural interventions to replace some load-bearing walls with steel beams. The building constituting block B, joined to the original body of the building, has a structure in load-bearing masonry, while the floors, more recently built, are also in brick-concrete, but with pre-cast prestressed joists.
The dynamic response of the building was assessed through OMA (Operational Modal Analysis) identification technique which allows you to determine the properties of the structure in real operating conditions, without the need to interrupt the activities inside the structure itself. The structural excitement, therefore, is due only to the environmental condition. A 3D model was defined in PRO_SAP; the results obtained from the finite element model were subsequently compared with those obtained from the experimental model.

Keywords: Dynamic identification; OMA, FE model; seismic behavior; mixed masonry-RC structures.

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10.1016/j.prostr.2023.01.102

1. Introduction

In recent years, Operational Modal Analysis (OMA) has become one of the most utilized techniques to check the safety of buildings and to allow an economic and effective planning of maintenance interventions, especially when, during their service life, they are utilized for several uses different from those initially foreseen.

The technique leads to the identification of the unknown modal parameters of a building from output-only acquisitions (Peeters et al. 2001; Reynders, 2012; Ranieri et al. 2014; Bru et al. 2015). This procedure allows us to accurately evaluate in situ the actual behavior of structures (Diaferio et al. 2019; Foti et al. 2019) in standard environmental conditions, without any applied force; for this reason, it is also used for buildings with monumental and historical value (Diaferio et al. 2016, 2021, 2022; Foti et al. 2012, 2015). In fact, some recent studies (Diaferio et al. 2007, 2017; Sivori et al. 2018) demonstrate that the knowledge of the modal parameters of the structure is essential for the evaluation of the seismic vulnerability and the effects of horizontal forces on structural and non-structural parts during earthquakes.

In the literature, several experimental tests on structures have been studied and carried out to improve the level of knowledge; the final aim is to validate an FE model capable to best reproduce the behavior of a real building.

In detail, this study involved a first phase of monitoring by acquiring data with the installation of accelerometers in *ad hoc* chosen points of the structure. The first results obtained from the experimental campaign are discussed and utilized to perform OMA by means of ARTeMIS software (Structural Vibration Solutions, 2019). Frequencies, modal shapes and damping ratios are utilized to update a numerical model of the building, in accordance with the archival documents and geometrical survey performed on the structure.

2. The case study

The building under study is the seat of the Municipality of Castellaneta, a town in the province of Taranto, in Puglia. The geographical coordinates of the municipality are 40°37'41"52 N 16°56'15"72 E, with an average altitude of the urban center equal to 245 m above the sea level (a.s.l.) and a maximum altitude equal to 411 m a.s.l.

The building host the territorial government offices (Fig. 1.a) and it is considered a strategic building, pursuant to the Ministerial Decree of 21/10/2003, issued by the Department of Civil Protection.

Figure 1. (a) Frontal view of the building. (b) Building blocks.

The structural complex being analyzed appears to have been built in two successive phases and is composed of two units (Figure 1.b): block A built between 1955 and 1957, and block B, for completion, presumably built between 1960 and 1961. From the recovered documentation and from the surveys carried out, the structural complex appears to be 9.80 m high, a footprint of about 860 m^2, with a rectangular shape and characterized by the presence of an internal courtyard.

Block A constitutes the main building and represents the original nucleus. It consists of two levels and is essentially in load-bearing masonry, with the addition of elements in reinforced concrete. The structure was subsequently subject to structural interventions which resulted in the removal of some load-bearing walls which were replaced with steel beams (rooms on the ground floor, where the bank is located and the room on the first floor, used as a council chamber). The floors are in brick-concrete made up of bricks and with reinforced concrete beams cast on site. The foundations presumably consist of curbs and plinths in reinforced concrete.

The building constituting block B, joined to the original body of the building, consists of three levels which are located at staggered heights with respect to the body of Building A. The structure is in load-bearing masonry, while the floors, of more recent construction, are also in brick-concrete, but with pre-cast prestressed joists. The foundations presumably consist of reinforced concrete curbs.

Inside the atrium there is an elevator shaft built in recent times. It is made entirely of reinforced concrete and allows the ground floor to be connected vertically with the first floor and block B, located at a height of approximately 1.05 m higher than block A. This structure is composed of reinforced concrete walls, arranged in a "C" shape, 25 cm thick. The foundations, on the other hand, are made up of reinforced concrete curbs.

Also, inside the atrium, there is a small technical room intended for a thermal power plant, with a structure completely independent from the rest of the building: it is made with bearing walls in tuff with a thickness of about 30 cm and a brick-concrete floor of the type with prestressed joists. The foundations presumably consist of reinforced concrete curbs.

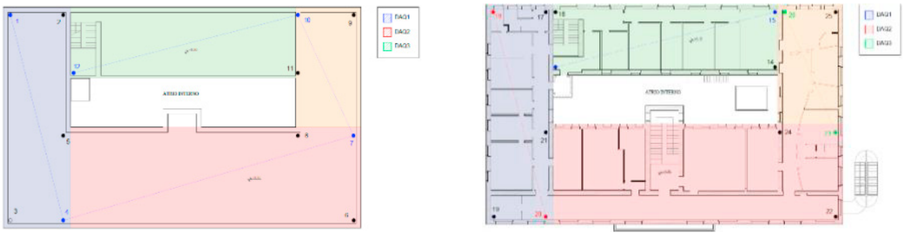
One of the most important tools for analyzing the state of health of a building is structural monitoring, i.e. the set of all the operations necessary to collect, manually or automatically, data relating to appropriate structural parameters, which are subsequently processed to obtain a picture of the behavior of the structure. The dynamic response of the building in question was assessed through OMA (Operational Modal Analysis) identification technique which allows you to determine the properties of the structure in real operating conditions, without the need to interrupt the activities inside the structure itself.

The structural excitement, therefore, is due only to the environmental conditions being measured, namely: wind, traffic noise, passage of people and any other micro-noise detectable by special accelerometers positioned on the structure. The monitoring campaign carried out on the Municipality of Castellaneta was conducted using 22 uniaxial piezoelectric accelerometers and the appropriate data acquisition system. The accelerometers used (PCB Piezotronics mod.393B31) are high sensitivity sensors (10 V/g) IPC (Integrated Circuit Piezoelectric), with a frequency range from 0.1 to 200 Hz, specifically designed to allow the detection of vibrations at very low level and low frequency. They were mounted, through a threaded pin, on a cubic-shaped metal element so that they can be arranged perfectly orthogonal to each other, allowing each of them to measure the acceleration value on the reference axis along which it was placed. The cubic support element (Figure 2c) was then fixed to the structure at the points where the measurement was intended to be carried out. The data acquisition system is made up of several NI-9230 modules, i.e. three-channel control units, 12.8 kS/s/channel, capable of simultaneously measuring the signals coming from the different sensors and inserted inside three CompactDAQ USB (Chassis National Instrument mod. Cdaq-9178). The data acquired by the control units were sent, via wireless connection, to the data acquisition software, LabView of National Instrument.

Figure 2. a) CompactDAQ USB with 8 slots; b) NI-9230 module c) Accelerometer blocks

As regards the positioning of the accelerometers, according to OPCM n.3907/2010, to allow a correct identification of the modal parameters, the measurements must be performed on all above-ground decks of the structure, recording the vibrations in the two main directions of the building (x and y). In the specific case in question, 11 measurement points were identified, and the arrangement of the accelerometers remained unchanged for all decks, except for significant variations in the plan along the height.

In particular, five measurement points were identified on the roof, and then 10 accelerometers were positioned, connected to the cDAQ1 (Fig. 2a); on the first floor of block B, on the other hand, two measurement points were identified, followed by the positioning of 4 accelerometers (Fig. 3b), also connected to the cDAQ1, while on the first floor of block A four monitoring points have been identified: points 18 and 20 are connected to cDAQ2, while points 23 and 26 to cDAQ3 (Fig. 3b).



a) b)

Figure 3. a) Position of accelerometers, quote z=9,80 m; b) Position of accelerometers, quote z=6,10 m and z=4,90

Seven acquisitions of varying duration, ranging from 10 to 20 minutes, were made on the Municipality of Castellaneta. Each acquisition had a sampling time Δt =0,000977 s and a sampling frequency fs=1Δt ≅1024 Hz.

3. Results

3.1. FE model

One of the most important steps of the structural analysis is the idealization of the structure that allows you to move from the physical to the numerical model. This passage involves the reduction of the number of degrees of freedom which in the continuous medium are infinite, while, considering only the nodes of the structure, that are in finite number. In this way the structure is discretized; it means that you move from the real to an idealized structure on which it is possible to apply the finite element method, to obtain an engineering solution to the problem.

For the FEM analysis of the building in question, PRO_SAP software was used; this software allows to design concrete, steel, masonry and timber structures. The solver implemented in the software is called "e_SAP" and has been developed by 2S.I. in collaboration with the University of Ferrara and École Centrale Paris CM2. Specific functionalities for civil engineering in the light of new regulations, such as second-order effects analysis and buckling analysis are of immediate application. Nonlinear step analysis (load history) and nonlinear analysis for large displacements (ropes, membranes, tensile structures, ...) are some of the tools available to refine the design of the structure. (PRO_SAP User Manual, 2019).

The modeling of the structure consists in the identification of the static scheme of the structure itself and in the definition of the properties of all the elements that compose it. In particular, the static scheme is created solely using nodes and structural elements. Once the properties of each element were defined, the respective materials were identified. PRO_SAP software is equipped with a large archive of materials: material n.6 was used for pillars and beams, corresponding to class C35/45 reinforced concrete and with the following characteristics (Table 1):

Table 1. Reinforced Concrete properties.

Characteristic	Value
Stress class (Rck)	45.0 N/mm^2
Specific weight	2.5 daN/cm^3
Young's Modulus (E)	34,600.0 N/mm^2
Shear Modulus (G)	14,417.0 N/mm^2

For the walls, on the other hand, material n.94 was used, corresponding to a masonry with segments of soft stone (tuff, calcarenite, etc.), with the following properties (Table 2).

Starting from the ground floor up to the roof level, all the necessary elements were modeled, obtaining the result shown in Figure 4.

After the geometric model, we proceeded with the assignment of the loads. PRO_SAP software has some automatisms that allow the program to define certain types of loads without the need for user intervention. Regarding the structure's own weight, the program automatically determined the amount of the load based on the characteristics of the materials assigned to the structural elements.

Table 2. Masonry wall properties.

Characteristic	Value
Compressive stress (fm)	1.4 N/mm^2
Allowable shear stress (fvm)	0.056 N/mm^2
Ultimate shear stress	0.182 N/mm^2
Specific weight	1.6 daN/cm^3
Young Modulus (E)	1,080.00 N/mm^2
Shear Modulus (G)	360.0 N/mm^2

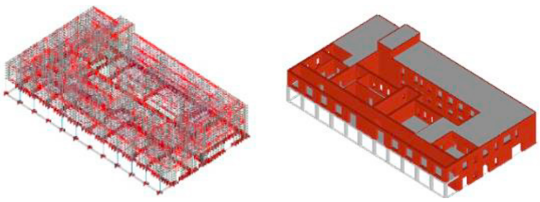


Figure 4. Finite Element Model

To define the characteristics of the seismic load, it was necessary to enter the geographical coordinates of the location where the building being analyzed is located, the category of the foundation soil and the geographical category. In the present case-study, with 9 modes of vibrating both conditions defined by the chapter §7.3.3.1 of the NTC 2018 have been respected. The additional eccentricity was set equal to 0.05 and represents the accidental eccentricity, defined in §7.6 of NTC 2018.

Finally, for the definition of the seismic analysis, the seismic masses used for the modal analysis were defined. The program automatically calculated the seismic masses from the vertical loads: in particular, the software automatically applied the necessary coefficients considering a unit weight for the permanent loads and the psi2 coefficient for the variables of the floors and for the snow load of the floors coverage.

At the conclusion of the assignment of the loads to the structure, the calculation combinations were defined: the software automatically generated the calculation combinations as required by the Ministerial Decree 2018.

Fig. 5 shows the modal deformations of the first three vibration modes of the structure, obtained from the FEM analysis conducted.

In particular, the analysis carried out showed that the first mode of vibrating is the prevailing mode in the y-direction, the second is the one prevailing in the x-direction, while the third mode corresponds to the main torsional mode of the structure. The frequencies and periods of the nine modes of vibrations of the structure, obtained from the first analysis are reported in Table 3.

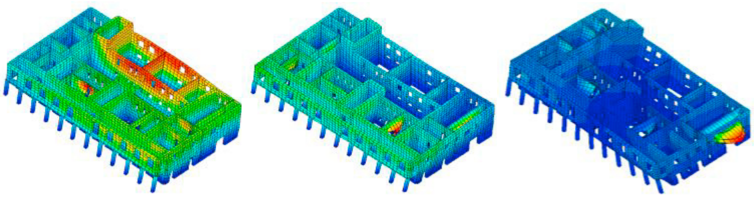


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Figure 5. – a) Deformed shape, Mode 1; b) deformed shape, Mode 2; c) deformed shape, Mode 3

Table 3. Frequencies and Periods for the first four modes.

Mode	Frequency [Hz]	Period [s]	%MX	%MY	%MZ	%MRZ
1	4.52	0.22	0	78	0	0
2	4.67	0.21	77	0	0	28
3	5.42	0.18	4	0	0	45
4	5.68	0.18	0	3	0	2

The results obtained from the finite element model were subsequently compared with the results obtained from the experimental model.The comparison between the numerical model and the experimental one was first made on the analysis of the dynamic behavior of the two models, and subsequently on the comparison between the frequencies of the respective modes.

In two models they have different modal deformations for the first two modes of vibration: the experimental model has the first flexural mode along the x-axis and the second flexural mode along the y-axis; on the contrary, the analytical model has the first flexural mode along the y-axis and the second flexural mode along the x-axis. It was found that this difference is because in the analytical model the partitions present within block B were modeled as load-bearing elements.

Then, the frequencies of each mode were compared, obtained from the different tests carried out for the experimental analysis and from the starting test relating to the numerical analysis.

We intervened on the elastic modulus characterizing the load-bearing masonry. The first analysis carried out on the FEM model was carried out by adopting a value E = 10800 daN/cm^2. Reference was made to NTC 2018, which for load-bearing masonry structures establishes the minimum and maximum values of the elastic deformation E to which improving or pejorative coefficients can be attributed based on the state of the art. Subsequently, through the procedure defined for model up-dating, it became possible to calibrate the behavior of the numerical model on the basis of the results obtained from the experimental analysis, varying from time to time the value of the elastic modulus of the masonry, until the analysis has reported values of the acceptable modal characteristics and as close as possible to those of the experimental model, but in any case being very careful to maintain the characteristics of the construction materials realistic and plausible according to the reference legislation. The value that made it possible to obtain a finite element model close to the experimental model, therefore used in the analysis, was E = 19000 daN/cm^2.

3.2 Experimental identification.

The model used to carry out the experimental analysis is shown in Fig. 6, which highlights all 42 nodes considered and the position and direction of the 22 accelerometers, indicated with arrows.

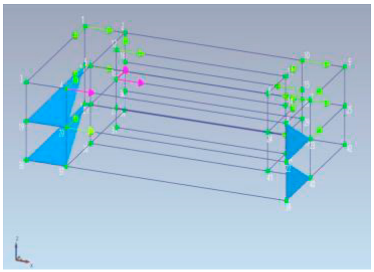


Figure 6. Experimental model of the building

Subsequently, the acceleration values sampled by the respective accelerometers were associated with each node.

The identification has been carried out by using SSI-UPC [Artemis Modal] estimation method: Fig. 7 shows the results carried out for Test 1 in the range [0-10] Hz where four frequencies have been identified.

The identification results for Tests 1, 2 and 3 are reported in Table 4, where the extreme repeatability of the experimental results in the various tests carried out is evident, guaranteeing the truthfulness of the identified values.

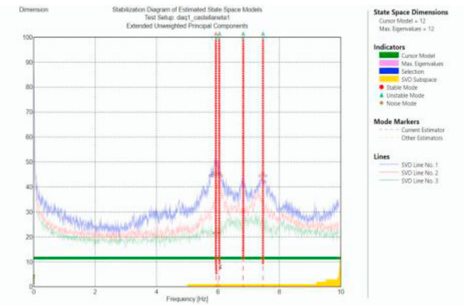
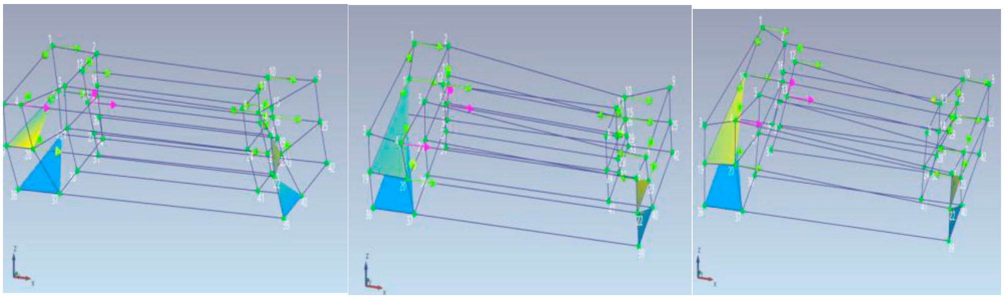


Figure 7. Identification by using SSI-UPC method for Test 1, four frequencies identified.

Table 4. Frequencies obtained from the experimental identification for the Tests 1, 2 and 3 and mean value.

Frequency [Hz]	Test 1	Test 2	Test 3	Mean Value
1st	5.94	5.89	5.95	5.93
2nd	6.04	6.02	6.05	6.04
3rd	6.85	6.80	6.79	6.81
4th	7.47	7.50	7.49	7.49

Furthermore, from the various tests, it was found that the mode shape corresponding to the 1st frequency is flexural along x-axis, the mode shape corresponding to the 2nd frequency is flexural along y-axis and the mode shape corresponding to the 3rd frequency is torsional. The mode shapes related to Test 1 are shown in Fig. 8.



a) b) c)

Figure 8. – Mode shapes: a) 1st frequency; b) 2nd frequency c) 3rd frequency.

3.3 Comparison between theoretical and experimental identification.

The comparison between the first three experimental and theoretical frequencies, after the updating procedure, corresponding to the first flexional modes and to the first torsional one is reported in Table 5 where it is also indicated the percentage error between them.

Table 5. Comparison between experimental and theoretical frequencies and relative percentage error.

Frequency [Hz]	Test 1	Test 2	Test 3	Experimental mean value	FEM value	Percentage error
1st	5.94	5.89	5.95	5.93	5.88	1.18%
2nd	6.04	6.02	6.05	6.04	6.01	0.66%
3rd	6.85	6.80	6.79	6.81	6.78	0.15%

3. Conclusions

The case study dealt with in the present research refers to the dynamic identification of a squat but complex structure, for which there was no certainty about the real possibility of successfully identifying the natural frequencies and mode shapes of the structure. Instead, not only it was possible to identify in a repeatable way the first four mode shapes of the structure, but it was also possible to create an FE model that reproduces similar results in dynamic terms. Further the model will be improved with an in-depth updating of the model, aimed at using the data obtained to carry out an analysis of the structure's seismic vulnerability.

Acknowledgments

Research activities were performed in the framework of the project “PON Governance 2014-2020 project on the reduction of seismic volcanic and hydrogeological risk for civil protection purposes – CIG 6980737E65 – CUP J59G16000160006” funded by the Presidency of the Council of Ministers Department of Civil Protection.

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