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PASSIVE SINGLE-STATION TECHNIQUES APPLIED FOR DYNAMIC CHARACTERIZATION OF REINFORCED CONCRETE BUILDINGS

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Highlights

Seismic structures have always been a topical and critical issue in recent decades because the consequences of inadequate designs are often catastrophic and periodically recurring. The most widely used method for evaluating buildings behavior is the modal analysis with the response spectrum, where stiffnesses and masses determine the linear dynamic response of the model. The hypotheses that guide the designer are dictated by regulations and often allow approximations that lead mathematical models to deviate from reality. The dynamic characterization on an experimental basis, therefore, seeks to bridge this gap between model and reality.

Abstract

This work aims to better understand and improve the dynamic characterization of concrete frame buildings through the combined use of finite element modeling and applied seismology. The behavior of the FEM model is compared with values obtained directly in situ through non-invasive tests based on a sensor capable of detecting the seismic microtremor and provide direct information in terms of oscillation periods and displacements. The case study structure was measured using a seismometer, and, at the same time, modeled using SAP2000. By starting from extremely different initial data, multiple variations were made to the model to produce an increase in frequency, aligning it with the one detected instrumentally.

Keywords

Dynamic Characterization, Ambient Seismic Tremor, Single Pocket Seismometer, Structural Analysis, Targeted Structural Modeling.

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1. INTRODUCTION

The dynamic characterization of existing structures can be addressed with two different approaches.

The first approach is based on finite element (FE) digital modeling, which can simulate the behavior of the structure and derive characteristic frequencies and participating masses. In-depth analyses are necessary to obtain a good model and to know the dimensions of the load-bearing elements and mechanical characteristics of the materials that make those structural elements up. Accurate surveys and destructive material tests are generally the basis for modeling as faithful to reality as possible; in fact, the accuracy of the response given by computer calculations is very high but would undoubtedly be useless if the input data are not correct. Often many simplifications are made to create the model. The modal analysis is never used by itself, but it is the base for the assessment of the maximum acceleration that the system would receive in case of seismic events. To make this assessment, the regulations require the application of specific conditions and assumptions that are not always in line with the actual reality.

The second approach, less known, is based on the use of seismometers, to directly obtain experimental results. These instruments can identify the main modal frequencies with few measurements, also carried out non-simultaneously. Although simultaneous measurements allow getting the phase information, the main features of the dynamic behavior of a structure can be assessed even with a single instrument. For framed structures, it is relatively easy to determine the first three vibrating modes, while it is more complex and less accurate to gain information on modes above the third. In particular, the first three modes often involve different percentages of masses in flexures alongside the two main directions and a torsional rotation. The flexures associated to the first vibrating modes of the structure are characterized by a phase deformation, in which each node moves in the same direction. This type of measurement provides real data without having first known the characteristics of the materials used or the cross-sections of the load-bearing elements. It is, in fact, a non-destructive test that is performed on the whole building, without the need for artificial external loads, and which provides accurate data on the actual behavior of the building. The test is based on the measurement of microtremors. They are low amplitude environmental vibrations of the ground caused by human or atmospheric disturbances. Recorded microtremors can provide useful information about the dynamic properties of soils, such as specific periods and predominant amplitude.

Experimental data [1] have often highlighted the presence of a large gap between the frequency of the first mode of vibration of the computer-modeled structure and that detected by the seismometer, even if the two methods should lead to similar results. This discrepancy means that the FE model does not correctly describe the actual behavior of the building under microtremor conditions.

A real case study has been selected for an in-depth investigation to overcome this problem. In this actual case, the modal analysis data obtained through the use of SAP2000 software [2] have been compared with the in-situ measurement results, and a further effort has been made to bring the values to convergence. The intention is to understand which are the primary and secondary structural elements (parameters) that contribute most to the variation of stiffness.

2. STATE OF THE ART

The research aims to deepen the characterization of buildings from a dynamic point of view to optimize structural interventions on existing constructions, using tools that can provide satisfying results in a relatively short time. Although it is feasible to carry out structural core drilling and extrapolate samples to retrieve information on the resistance and the morphology of vertical and horizontal elements, it is now growing the need to find direct and prompt answers able to limit destructive-tests. Technological developments in the geological field seem to be promising in this direction. In particular, the instruments used for the detection of seismic environmental noise are worth mentioning. These instruments measure vibrations coming from natural and non-artificial sources and therefore defined as "*passive*". The HVSR (Horizontal to Vertical Spectral Ratios) method represents one of the main techniques in this field; it was first applied by Nogoshi and Igarashi in 1970 and developed by Nakamura in 1989 [3]. By evaluating the relationship between the horizontal and vertical components of surface waves, the method can identify the amplification frequencies of the subsoil, thus allowing to compare them with those of buildings, verifying the presence or absence of resonance phenomena [4].

Although the invention of rudimentary seismographs dates back to the 2nd century AD, the authorship of the first seismograph, in a modern sense, can be attributed to the mathematician and philosopher Luigi Palmieri from Campania who laid the foundations for its subsequent development [5]. A seismometer is defined as an instrument that measures the temporal dependence of displacement, speed, or acceleration, using a mass that provides enough inertia. The seismometer produces a seismogram, i.e., a graph representing the dependence of the collected data on time. In structural assessments, a seismometer is generally used to detect acceleration, speed, or displacements depending on time, whose operation is based on sensors, amplifiers, and analog or digital recording instruments [6]. Typically, structural surveys are based on the use of accelerometers [7–9]. Under operation (or passive) modal analysis, it is assumed that the background noise is a white function. In the present research, we used a seismometer, also known as tromometer (from the Greek word for 'tremor'), tuned to record ambient noise. These instruments were originally developed to study the ground amplification properties (e.g., using the HVSR method) and were later found to apply also for the modal characterization of structures. The instrument has also been used for the dynamic characterization of a notable structure (the Eiffel tower [10]) and the soil-structure interaction during recent earthquakes [11–14]

3. CASE STUDY: ATHENS STUDENT HOUSE

Within the framework of the EU Project Pro-GET-onE [15], the design of an integrated seismic, architectural, and energy improvement system is currently in progress which is aimed at the realization of a prototype applied to a building belonging to the Zografou campus of the National and Kapodistrian University of Athens.

The in-line building is built in reinforced concrete (RC) frame with a total length of 58m. In the integrated

Fig. 1. *Ground floor plan of the Athens case study. In the blue box, separated by the expansion joint, the northern part of the building, and the different measuring stations.*

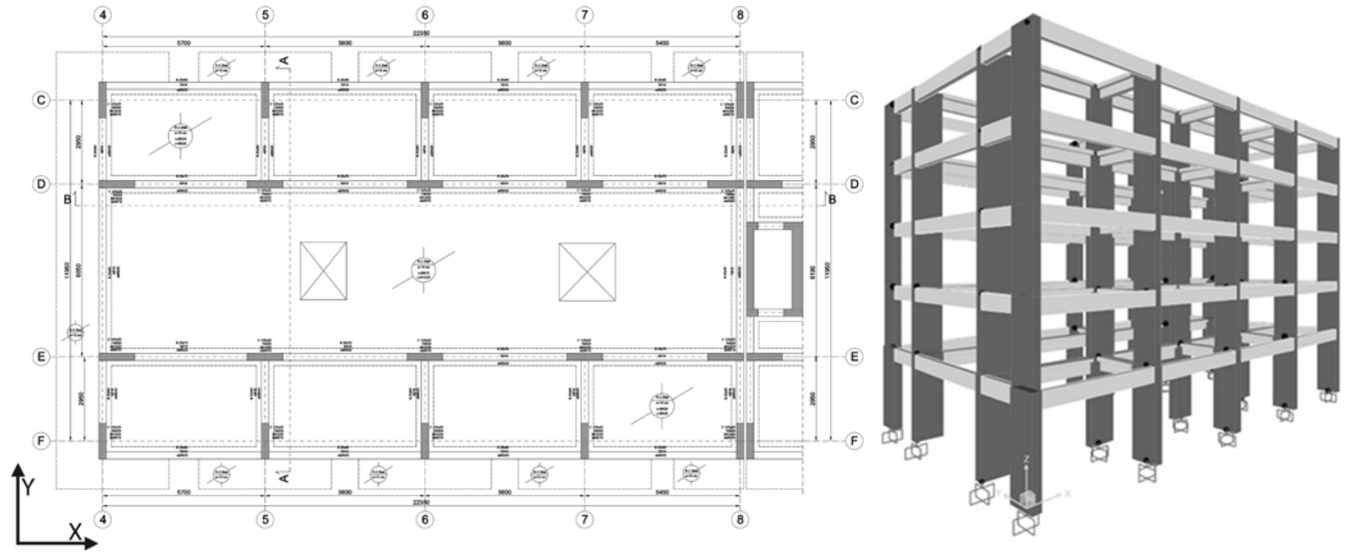

Fig. 2. *Structural plan of the type floor and a picture of the finite element model.*

system development as well as in this article, the northern portion of the building, separated by an expansion joint, will be taken into consideration (Fig. 1).

The structure consists of five floors, the basement 3,9 m high, and the four upper floors 3,0 m each; the plan dimensions of the Norther building are 22,35x12 m. The horizontal slabs are made of 18 cm thick RC and can be considered as diaphragm constraints. The vertical elements are RC walls of 125x25 cm arranged alternately in the two main directions, while the beams have different heights (from 55 cm to 70 cm) depending on the spans with a recurring web of 25 cm in width.

In the structural layout of the model floor (Fig. 2), all around the perimeter of the structure, there are balconies, made with a 15 cm thick slab, protruding 1 m on the larger sides and 1,5 m on the smaller ones. All the information concerning the building structure has been taken from a technical report made to obtain an intermediate level of knowledge. The reference categories for establishing imposed loads refer to Eurocode 1 [16]. Category A for parts of the floor dedicated to the rooms of the students (2 kN/m^2) and C3 for the central corridors (5 kN/m^2). Balconies were considered as loads applied on the perimeter beams of the frame. Hence a distributed torsional and shear loads were determined based on a cantilever scheme, also considering the imposed loads of 4 kN/m^2. Finally, for this discussion, it is essential to indicate the real stiffness values adopted, considering the cracking phenomenon. These values (Table 1) refer to the approach presented in the table 10-5 of the ASCE 41-13 [17]. Because of a lack of data about existing foundations, the FE model has been simplified by inserting rigid joints at the base.

Element	Flexural Rigidity	Shear Rigidity
Beam	0,3 EI	0,4 GA
Column	0,7 EI	0,4 GA

Tab. 1. *Effective stiffness values due to cracking.*

3.1. FIRST RESULTS FROM MODAL ANALYSIS

The evaluation of the dynamic characteristics of the building is carried out through a modal analysis that follows the indications provided by Eurocode 8 [18] (Tab. 2). By analyzing the results, it is possible to state that the first vibrating mode of the building has a frequency of 1,24 s$^{-1,}$ and it is a flexure in the transverse direction (y) that activates a mass percentage equal to almost 58% of the total mass of the building. The second vibrating mode is mainly torsional around the z-axis, with about 60% of the total mass activated with a frequency of 1,26 s^{-1}. The third vibrating mode is flexural in the direction of the longitudinal development of the building (x), with a frequency of 1,40 s^{-1}, and an activated mass equal to 85,7% of the total (Fig. 3).

Mode	Frequency (s^{-1})	Participating Mass		
		Ux	Uy	Rz
1	1,238	0	57,9%	26,0%
2	1,264	0	25,6%	57,9%
3	1,396	85,7%	0	0
4	4,151	≈ 0	6,6%	5,0%
5	4,261	≈ 0	5,3%	6,5%

Tab. 2. *Athens student house. Main results in terms of frequencies from the modal analysis.*

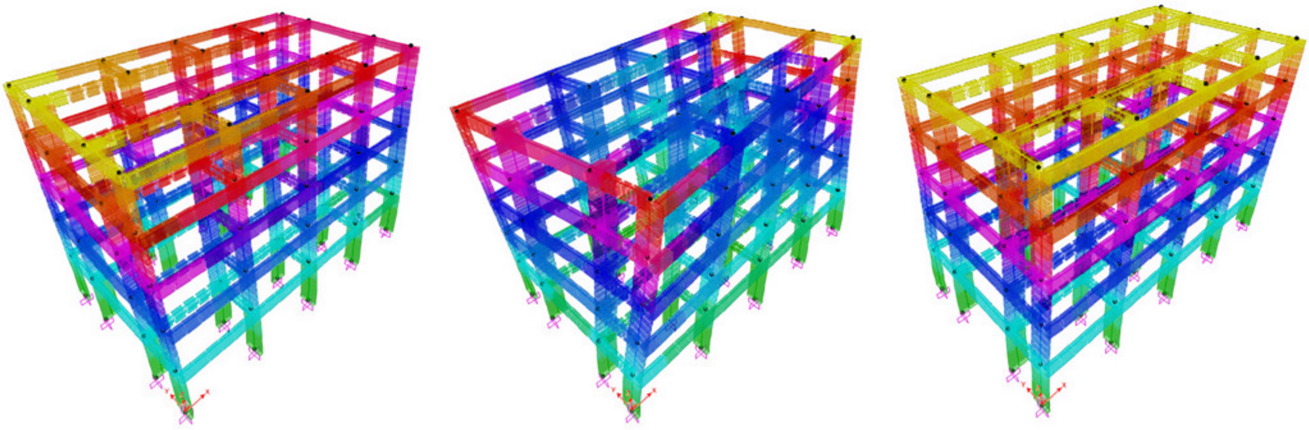

Fig. 3. *Deformable shapes of the first three vibrating modes. Respectively in ascending order from left to right.*

3.2. EXPERIMENTAL SURVEY

The survey of the building provided four measurements per each floor so that the structure could be analyzed as if it consisted of two independent parts separated by the expansion joint. Two measurements per section were taken, one in a central position for the single structure and one along the perimeter. Two stations for floor allow better studying the behavior of the building, also highlighting possible torsional modes. The measurements were made with Tromino® in positions A, B, C, and D (Fig. 1) on each floor on the same vertical axes. The acquisitions made in point A on different stories were superposed, and normalized to the basement level measurement, to highlight the relative displacements. In measurements on buildings, the smoothing of the curves is set to 2% so that the peaks are visible (Fig. 4).

Grilla software [19] was used to process the data. Each measurement is highlighted with a different color: the displacement is more significant as the floors increases for both the N-S and the E-W components. There is a peak around the frequency of 4,5 Hz for the E-W component, which indicates flexural behavior as the first vibrating mode. A second peak is detectable at 5 Hz frequency for the N-S component; this maximum means a flexure as the second vibrating mode. Other peaks with smaller amplitudes can be detected at 11 Hz, 14 Hz, 15 Hz, 21 Hz, and 25 Hz frequencies, indicating subsequent vibrating modes. Frequencies peaks above 50 Hz are of no longer interest in the analysis because they are too close to typical human activity frequencies. It is advisable to compare central and perimeter measurements to verify the presence of torsional peaks choosing the curves of the roof that have the highest amplitudes (Fig. 5).

By comparing these two measurements, it is possible to notice that the previously identified peaks are also present for the measurement of point B (blue curve).

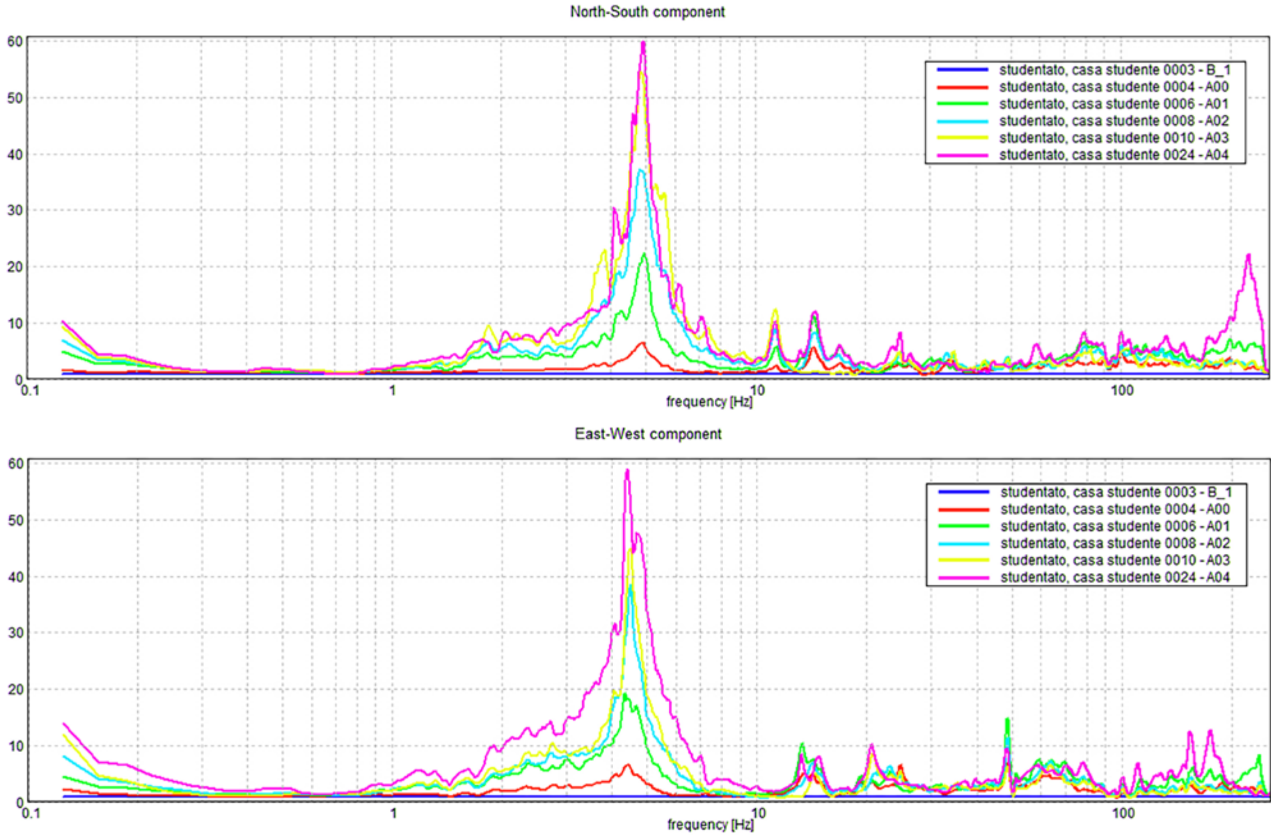

Fig. 4. *The spectral ratio between the N-S component recorded at different levels at site A on the structure and the spectral component recorded at the ground level (top). Same for the E-W component (bottom).*

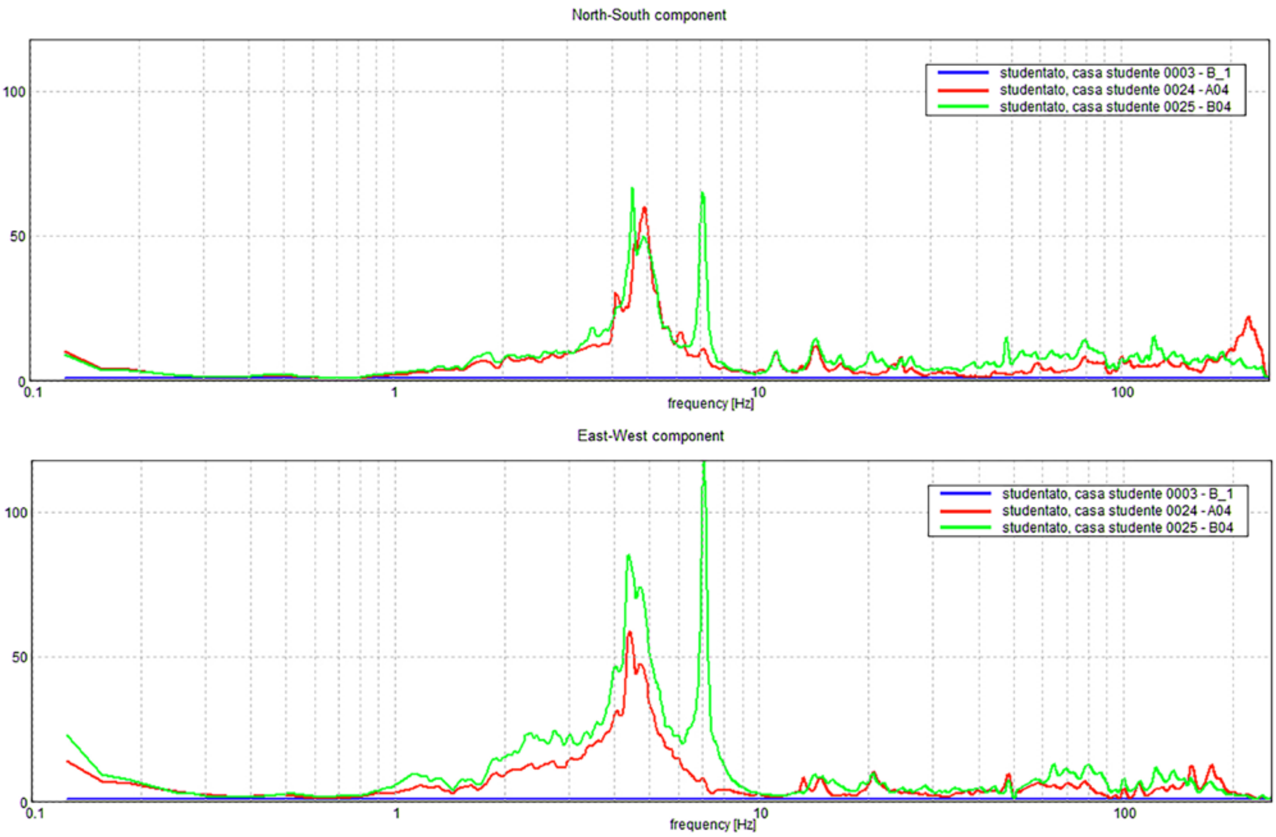

Fig. 5. *The spectral ratio between the N-S component recorded at the roof level at site A on the structure and the spectral component recorded at the roof level at site B. Same for the E-W component (bottom). A highlight of the torsional peak.*

From the graph of the East-West component, it is possible to register that the maximum at 4,5 Hz probably has a torsional component, highlighted by the fact that the amplitude of the B curve is much larger than the amplitude of the A curve. It is confirmed that the first vibrating mode at 4,5 Hz is predominantly flexural but with a relevant torsional component. The peak at 5 Hz, on the other hand, has a similar trend for both curves, so the second vibrating mode is a flexure. In comparison, at the frequency of 7 Hz, there is a maximum of curve B that is not present on curve A, indicating exclusively torsional behavior. Therefore, the third vibrating mode is the first torsional mode. As the frequency increases, the precision in reading the data is lower, so it is superfluous to go beyond the 5th mode (Tab. 3). Due also to thermal fluctuations of the structural parameters, the uncertainty in the assessment of the modal frequencies is around 2-3%.

Mode	Frequency (s^{-1})	Behavior
1	4,5	1° mode – flexure/torsion (E-W)
2	5,0	1° mode – flexure (N-S)
3	7,0	1° mode – torsion
4	11,0	2° mode – flexure (N-S)
5	14,0	3° mode flexure/torsion (N-S)

Tab. 3. *Athens student house. Dynamic characterization carried on with Tromino®.*

The frequencies detected in situ are almost four times higher than in FE modeling. The first vibrating mode is activated at about 4,5 Hz in microtremor conditions, while at 1,2 Hz in the software modal analysis, with a difference of 3,3 Hz. The in-situ frequency is 275% times higher than the one registered in the FE model. The same is true for the modes after the first, which for the FE model appear at 1,3 and 1,4 Hz frequencies, while the measurements are at 5 and 7 Hz frequencies. This discrepancy is considerable, but it is not the only one. The data measured under microtremor conditions show an inversion between the second and third modes of the structure. While for the analysis performed with the FE model, the second vibrating mode is mainly torsional, the experimental data show that the third mode is torsional. These discrepancies are due to several factors. Since under microtremor conditions, the loads are minimal, non-structural elements can contribute to stiffening the structure. During an earthquake, when this contribution fails, the building's frequency may be lower than that measured under microtremor conditions. At the same time, it may be relevant to report also that the FE models that are created for the dynamic assessment of structures are subject to strong approximations and are often using safety parameters or coefficients that bring to different results. This simplification brings to an underestimation of the real frequency, which is higher than that calculated by the model. Especially considering frequent earthquakes, with a low return period.

4. INFLUENCE OF PARAMETERS ON MODELING

The most important parameters that affect the modal analysis of the structure will be analyzed: characteristics of the construction system and materials, the magnitude of the loads, evaluation of the curtain walls, modeling of floors, and the underground level with related restrains. For this study, reference was made to the master thesis developed in collaboration with Carretti C., Fusco G., and Marini L. on the passive dynamic characterization of reinforced concrete structures for the calibration of numerical models [20–22].

The identification and characterization of the building construction system under examination is the first step to be taken to reach a level of knowledge, such as to be able to estimate its behavior. The definition of the spatial distribution of the load-bearing elements and their geometry is essential to create the stiffness matrix that determines the vibrating modes. A laser scanner survey showed that the walls in the basement were larger than those provided by the project reports. Instead of having a constant horizontal cross-section of 25x125 cm, the walls are 40x125 cm at the underground level and 35x125 cm on the upper floors. This variation was attributed to the vertical elements. Again, regarding the RC walls, the replacement of beam elements with shell elements indeed represents another possible, if not necessary, variation based on the dimensional ratios of the elements. The presence of the balconies was initially considered only through loads applied on the perimetral beams; as a third variation linked to the characteristics of the building, their insertion as cantilever slabs was evaluated. The uncertainty about the materials used during construction can also be very high and, at the same time, decisive on the results. The impact on the dynamic modal analysis depends directly on the elastic modulus (E) of the materials that characterize the structure and, consequently, on the degree of cracking. It can be considered in the software through a general reduction of E, or, as in this case, in a more specific way, on the resistant elements flexural and shear stiffness. Specifically, the elastic modulus has been varied from the initial precautional value given by the technical report of 29962 N/mm^2 up to a maximum of 45420 N/mm^2 relative to a C80/90 concrete. Concerning the cracking phenomenon, it was decided instead to eliminate the precautional reduction since it is immediately perceptible from the current conditions of the building that the assumed cracking values are too high for the real conditions of the structure.

Once the characteristics of the load-bearing structure have been established, the next step is to determine the loads it must bear. In the analysis aimed at identifying the seismic vulnerability, it is necessary to comply with the indications provided by the regulations. However, the aim of the regulations is the life-safety of inhabitants. The expected loads can, therefore, be much higher than those that the structure bears and very different from the crowded condition assumed by the regulations. Therefore, different load reduction solutions have been assessed, ranging from a drastic reduction to complete removal. Besides, the stratigraphies of the floor were progressively detailed, obtaining the precise value of permanent loads carried by also removing the generic distributed load resulting from the internal partitions.

The curtain walls are among the others, the most influential components on the stiffness of the structures and the consequent vibrating modes. It has been demonstrated how these can be relevant to the overall behavior of a building [23, 24]. Three solutions were considered: linear loads applied to the beams, equivalent X-braces, or shell elements inserted in the frame spans affected by the presence of significant masonry walls. The thickness of the masonry is 25 cm considering an elastic modulus of 3000 N/mm^2.

Like what happens with curtain walls, there are different ways of representing floor slabs. If sufficiently rigid, they can be compared to a rigid diaphragm constraint that connects all the points belonging to the same plane. In this case, modeling using shell elements, with weight and stiffness characteristics like those of the material arranged on-site, is one of the possible alternatives to a rigid diaphragm.

Finally, the presence of the underground floor implies the need to evaluate the incidence of the variation of the ground level for the application of seismic loads. Its presence, or absence, affects the global behavior of the structure, and it must be evaluated in the modeling variables. Not having enough data on the soil characteristics, it was difficult to model its behavior in detail. Therefore, different hypotheses of restraint of the vertical elements at ground level have been evaluated. The use of a rigid link represents the most extreme and relevant (in terms of vibrating frequency results), situation. Under microtremor conditions, it simulates the correct behavior of the building as directly connected to the ground.

Parameters	Frequency increase (%)	Mass variation (%)	Order variation
Constructive system	12,0	7,0	No
Shell for modeling walls	1,4	20,0	No
Modeling of balconies	2,0	28,2	No
Modulus of elasticity	23,1	-	No
Cracking	44,1	19,1	No
Loads	7,9	17,9	Yes
Curtain walls and internal partitions	16,0	16,0	Yes
Modeling of floor slabs	16,0	23,6	No
Underground floor	27,9	25,6	No

Tab. 4. *Incidence of the most relevant parameters on the characteristic frequencies.*

4.1.RESULTS AND SUMMARY

It is worth remembering how the analyses carried out refer to an existing building on which surveys and measurements have been carried out to determine specific parameters as useful variables to understand their impact on the model's behavior. After having analyzed the impact on the modal analysis in terms of frequencies, participating masses, and order of vibration modes, it is possible to illustrate the summary of the most relevant data. By using the acquired data, it is possible to intervene in the initial model to make it converge on the experimental data collected. It is possible to notice that the most relevant parameters on the modal analysis of the building are the cracking, which affects the frequency up to 44%, the elastic modulus (23%), and the evaluations on the underground floor, about 30% (Tab. 4).

4.2. FINAL MODEL

Based on the analyses conducted and data collected on the key parameters, various modifications were made to the initial FE model:

- Use of C20/25 for all the elements of the structure without stiffness reduction due to cracking.
- Use of rigid diaphragm constraints and floors considered as equivalent loads distributed on the RC beams.
- "Real" configuration of permanent and imposed loads (G_2=1,4 kN/m^2 on each type floor while for the roof G_{2roof}=1,85 kN/m^2; Q_k=0,5 kN/m^2 for the corridors and 1 kN/m^2 for the rooms).
- The extension joint that divides the buildings is eliminated in the model, and a concrete slabs connection is considered. In seismic microtremor conditions, the whole building vibrates together.
- Walls modeled using shell elements with the horizontal cross-sections obtained by the laser scanner survey.
- Balconies are considered in the rigid diaphragm constraints around the perimeter of the building with the proper loads' definition.
- Insertion of the stairwell and elevator spaces using shell elements to simulate the walls considering the whole structure in the model.
- The underground floor is considered by inserting perimeter partitions and considering the influence of the ground with perimeter supports (restraining the translations) at the level of the external soil.
- The external curtain walls and the main internal partitions have been inserted considering a 30 cm masonry walls modeled with shell elements.

The modal analysis of the modified model led to the results presented in the following table (Table 5), together with the results of the initial model, carried out following the indications of the regulations, and the experimental data obtained with Tromino®.

N° Mode	Frequency (s^{-1}) – Main direction		
	1° Model	Final model	Experimental data
1	1,238 – Uy	4,542 – Uy	4,5 – Uy
2	1,264 – Rz	4,818 – Ux	5,0 – Ux
3	1,396 – Ux	4,849 – Rz	7,0 – Rz
4	4,151 – Uy	15,931 – Ux	11,0 – Ux
5	4,261 – Rz	17,674 – Rz	14,0 – Rz

Tab. 5. *Comparison of characteristic frequencies between the initial and final models with the experimental reference data.*

The results show that in the final model, the first vibrating mode occurs at a frequency of 4,54 s^{-1} and mainly involves a flexure along the y-axis that activates a participating mass equal to 68% of the total mass of the building. The second vibrating mode, at a frequency of 4,82 s^{-1}, consists of flexure in the x-direction, activating 78,5% of the mass. The third vibrating mode is a torsion around the z-axis, at 4,85 s^{-1}frequency, which activates the 67,2% of the total mass. It is possible to notice how all the modifications applied to the model allow getting values very close to the experimental data both in terms of vibrating modes frequencies and directions.

5. CONCLUSIONS

The research shows how it is possible to calibrate the numerical model realized during the design phase, bringing the characteristic frequencies to convergence with the experimental data measured by a single station survey. The discrepancy that can be found seems to be mainly due to the presence of curtain walls, partitions, and considerations related to cracking and modulus of elasticity of the material, parameters often not considered or reduced based on prudent assumptions. The most incisive parameter on the variations appears to be the external curtain walls, especially where they are present in a significant way and have significant dimensions. Considering them induces changes not only to the participating frequencies and masses but also to the order of the vibrating modes. The discrepancy between the frequency values of the structures modeled according to the regulations and the experimental data found on-site raises questions about the safety assessment. By applying different periods to the response spectra, it is possible to obtain significantly different values of reference pseudo accelerations. On the one hand, these differences, lead to an increase in the structure resistance, due to an increasing number of resisting elements and a reduction in displacements due to a considerably increased stiffness; on the other hand, a lower period increases the seismic demand and an overall structure ductility reduction. Last, beyond the difference between experimentally assessed modal frequencies and frequencies computed through numerical models, another aspect to consider is the resonance frequencies of the ground, particularly when they are close to those of the structure.

6. ACKNOWLEDGMENTS

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7. REFERENCES

[1] Pentaris F (2014) A novel horizontal to vertical spectral ratio approach in a wired structural health monitoring system. J Sens Syst 3:145–165. https://doi.org/10.5194/jsss-3-145-2014
[2] Computers & Structures (2018). In: SAP2000 v20.2.0, Integrated Finite Element Analysis and Design of Structure. Berkeley (CA, USA)
[3] Nakamura Y (1989) A method for dynamic characteristics estimation of subsurface using microtremor on the ground surface. Railw Tech Res Inst Q Rep 30(1):25–30
[4] Albarello D, Castellaro S (2011) Tecniche sismiche passive: indagini a stazione singola. Ing SISMICA 28:32–62
[5] Flores E (1932) Luigi Palmieri. Tipografia editrice A. Miccolli, Napoli
[6] Descrizione e utilizzo dello strumento. http://ionos.ingv.it/ASL2017/Sismometro.html. Accessed 2 May 2019
[7] Rytter A (1993) Vibrational Based Inspection of Civil Engineering Structures. PhD. Thesis, Aalborg University, Aalborg, Denmark
[8] Testoni N, Zonzini F, Marzani A, et al (2019) A Tilt Sensor Node Embedding a Data-Fusion Algorithm for Vibration-Based SHM. Electronics 8:45. https://doi.org/10.3390/electronics8010045
[9] Quqa S, Landi L, Diotallevi PP (2018) On the Use of Singular Vectors for the Flexibility-Based Damage Detection under the Assumption of Unknown Structural Masses. In: Shock Vib. https://www.hindawi.com/journals/sv/2018/9837694/abs/. Accessed 2 May 2019
[10] Castellaro S, Isani S, Bartolomei M, Perricone L (2013) La Torre Eiffel. Struct Model 13/14:8–14
[11] Castellaro S, Padrón LA, Mulargia F (2014) The different response of apparently identical structures: a far-field lesson from the Mirandola 20th May 2012 earthquake. Bull Earthq Eng 12:2481–2493. https://doi.org/10.1007/s10518-013-9505-9
[12] Castellaro S, Negri N, Rovere M, Vigliotti L (2012) Interazione suolo-struttura e struttura-struttura: una lezione dal terremoto di Mirandola del 20.05.2012. Il Geologo 45:47–52
[13] Pesci A, Teza G, Casula G, et al (2011) Esperienza di misura mediante lo strumento Tromino® per lo studio delle vibrazioni e delle sollecitazioni naturali e antropiche. http://istituto.ingv.it/l-ingv/produzione-scientifica/rapporti-tecnici-ingv/copy_of_numeri-pubblicati-2010/2011-09-19.4136663709
[14] Ditommaso R, Vona M, Gallipoli MR, Mucciarelli M (2013) Evaluation and considerations about fundamental periods of damaged reinforced concrete buildings. Nat Hazards Earth Syst Sci 13:1903–1912. https://doi.org/10.5194/nhess-13-1903-2013
[15] ProGETonE | solutions for more comfortable, efficient, and seismic-safe homes. https://www.progetone.eu/. Accessed 16 Apr 2020
[16] (2010) Directive 2010/31/EU of the European Parliament and of the Council of 19 May 2010 on the energy performance of buildings
[17] American Society of Civil Engineers (2017) ASCE standard, ASCE/SEI, 41-17, seismic evaluation and retrofit of existing buildings. American Society of Civil Engineers, Reston, Virginia
[18] European Committee for Standardization (CEN) (2004) "EN 1998-1: Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings"
[19] Castellaro S Grilla Software. http://tromino.it/Italiano/frameset-grilla.htm. Accessed 16 Apr 2020
[20] Fusco G (2018) Caratterizzazione dinamica passiva di strutture in calcestruzzo armato per la calibrazione di modelli numerici. Il caso studio del blocco laboratori della facoltà di

chimica industriale. Master's Degree Thesis, Alma Mater Studiorum - Università di Bologna

[21] Carretti C (2018) Caratterizzazione dinamica passiva di strutture in calcestruzzo armato per la calibrazione di modelli numerici. Il caso studio dell'edificio sito a Reggio Emilia in via Campagnoni civici 34, 36, 38. Master's Degree Thesis, Alma Mater Studiorum - Università di Bologna

[22] Marini L (2018) Caratterizzazione dinamica passiva di strutture in calcestruzzo armato per la calibrazione di modelli numerici. Il caso studio dell'edificio sito a Reggio Emilia in via Campagnoni civici 56, 58. Master's Degree Thesis, Alma Mater Studiorum - Università di Bologna

[23] Monetti G (2015) Risposta Sismica di Telai Tamponati in c.a.: Analisi numeriche e Valutazioni Sperimentali Preliminari. Master's Degree Thesis, Università degli Studi di Pavia

[24] Pompa N (2012) Analisi sismica non lineare di telai in c.c.a. con tamponamenti. Master's Degree Thesis, Alma Mater Studiorum - Università di Bologna