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CALIBRATION OF A COMPUTATION MODEL FOR A REINFORCED CONCRETE STRUCTURE AGAINST THE EXPERIMENTALLY DETERMINED DYNAMIC CHARACTERISTICS

BY

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Abstract. The dynamic characteristics of the structures range are depending on their mass and lateral stiffness. In the present paper a method for improving the computation model is proposed, thus creating a link between the dynamic characteristics obtained with the computation model based on the finite element method and the experimentally determined dynamic characteristics. The finite element model was obtained using the program ETABS and the experimental dynamic characteristics were obtained on a low vibration level. To intercept the dynamic response in situ five simultaneous Kinemetrics seismometers were used placed on the longitudinal and transversal direction. The dynamic identification was performed with the ModalVIEW program based on the Stochastic Subspace Identification. The calibration of the computation model with finite elements was accomplished by adjusting the brick masonry longitudinal elastic modulus.

Key words: dynamic identification; transducers; shape modes.

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1. Introduction

The concern of the engineers regarding the dynamic identification of structures has started for more than five decades, but the topic research was enhanced in the last 20 years due to the possibilities given by the technological development (Ştefan, 2001).

During the service stage the framing system, of any type, undergo some level of degradation due to the environmental action (Budescu *et al.*, 2004). A permanent monitoring of the structures to determine from an incipient stage the changes in the dynamic parameters could have significant benefits on limiting the degradations, performing an evaluation after an earthquake, warning when some of the safety parameters have been exceeded or extending the service life.

The identification of the dynamic properties and of the structures status in the frequency domain is based on measuring the natural frequencies and vibration modes (Gavriloaia & Gosav, 2006). To extract the modal properties the recorded signal is transformed into a digital one using the Fast Fourier Transform (FFT) and other similar algorithms. The dynamic identification can be approached based on the analysis of the vibration shape modes, comparing the deformed shapes expressed as displacements or rotations in different vibration modes. Two methods can generally be applied for the comparison of the vibration modes: the Modal Assurance Criterion (MAC) and the Coordinate Modal Assurance Criterion (COMAC) (Li *et al.*, 2011).

In this work the authors present a case study concerning the experimentally determined dynamic characteristics and the analytical analysis performed on a framed structure and look into developing an improved method for the computation.

2. Outline of the Structure

The structure analysed in the present paper is a reinforced concrete frame building with a height of 22.4 m (measured from ±0.00). The building has the height level composed of basement (2.4 m), underground floor (3.4 m), ground floor (4.9 m), first floor (3.9 m) and another four levels (3.7 m each).

The functional of the building consists of commercial spaces at the ground floor and apartments at the superior levels. The access is carried out through two stairways. The separating walls are made of hollow brick masonry with a thickness of 0.15 m and 0.25 m. The foundation is a foundation raft with the thickness of 0.50 m.

The structure has a regulate shape with the maximum dimensions of 23 × × 43.35 m. A walk able terrace is located at the top level. The terrace occupies one opening of the building thus differentiating the top level from the ones below (Fig. 1).

Fig. 1 – General view.

The longitudinal interaxial distances are of 1.8 m and 3.7 m while on the transverse direction are variable, between 1.55 m and 6.2 m (Fig. 2).

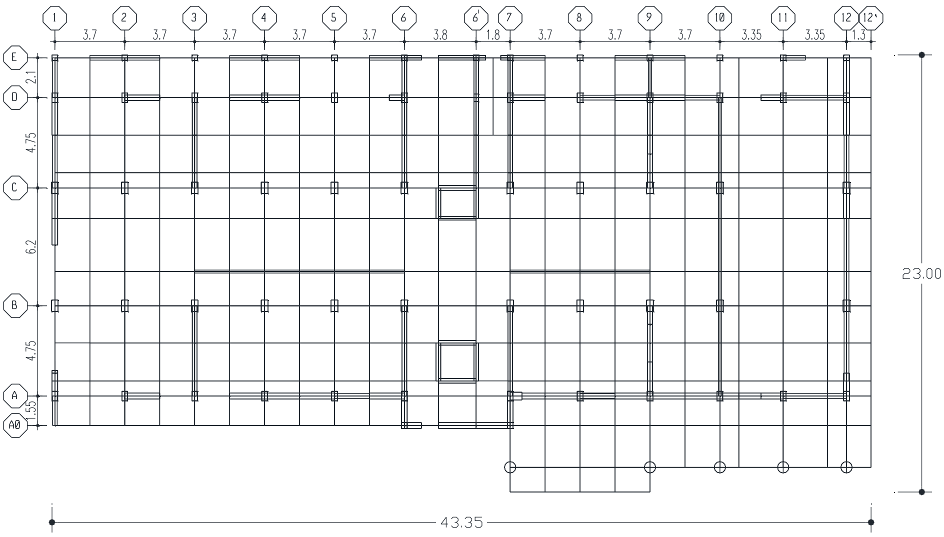


Fig. 2 – First floor plan (dimensions in m).

3. Experimental Procedure

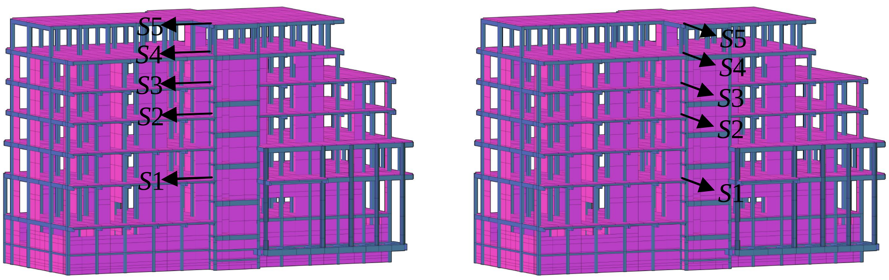
3.1. *In Situ* Dynamic Testing

The dynamic parameters have been measured using the ambient vibrations and the cyclic action of an excavator hitting the ground with the bucket, at intervals of 4...5 s.

Due to the reduced amplitude of the vibrations, the building is stressed in the elastic zone and the measured dynamic parameters were influenced by the stiffness of the non-structural elements. Five speed seismic transducers Kinemetrics, model *SS*1 (Fig. 3), have been used to capture the structures response. The transducers signal has been taken by a data acquisition system, amplified and transformed from analogic to digital signal, then visualized and recorded on the hard drive. Due to the reduced level of the electric signal the link between the transducers and the acquisition system was performed utilizing screened cables with low electrical resistance and small losses.

Fig. 3 – Positioning of seismometers *S*2 and *S*3.

The transducers have been positioned on the transversal and longitudinal direction along the height of the building, at the first, third, fourth, fifth level and on the terrace (Fig. 4), and also at the corners of the building at the fifth floor (Fig. 5).



Longitudinal direction Transversal direction

Fig. 4 – Positioning of the transducers on the vertical, axis 6-A0.

Over 20 measurements have been recorded with total time intervals between 1 and 3 min and a sampling interval (time interval between two consecutive readings) of 5 ms. From the recorded signal for the Fast Fourier Transform (FFT) analysis an area was extracted with a minimum number of stray signals, 2,048 values (2^{11}), corresponding to a time interval of 10.24 s, at a resolution of 0.0488 Hz.

Fig. 5 – Positioning of the transducers on the horizontal, 5th floor.

3.2. Experimental Results

The signals of the five transducers have been recorded simultaneously to evaluate their synchronism and identify the vibration modes. A graph of the recorded signal is presented in Fig. 6.

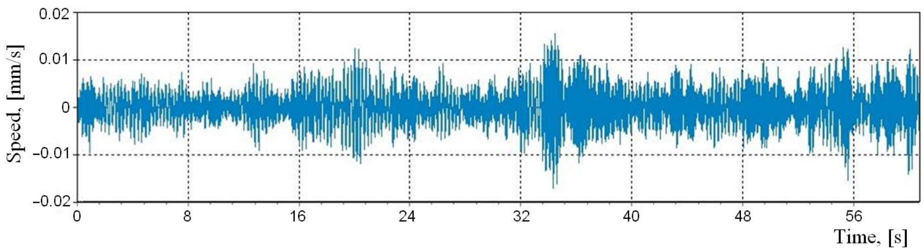


Fig. 6 – Signal recorded with the *S*5 transducer on the transversal direction.

Analysing the Fourier spectrums (Figs. 7 and 8), the first vibration mode with the frequency of 3.84 Hz is noticed in the consistency of the signal on the transversal direction and the second vibration mode is observed on the longitudinal direction with the frequency of 4.27 Hz.

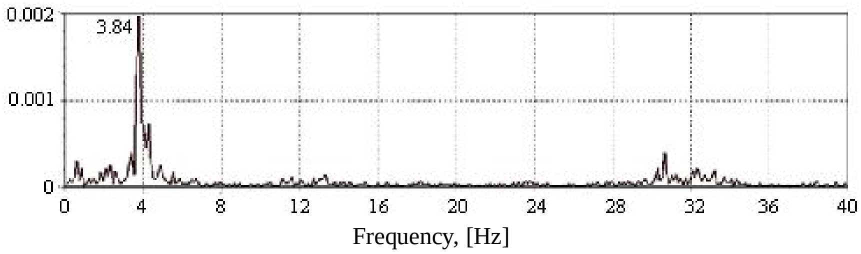


Fig. 7 – Fourier spectrum recorded with the *S*5 transducer on the transversal direction.

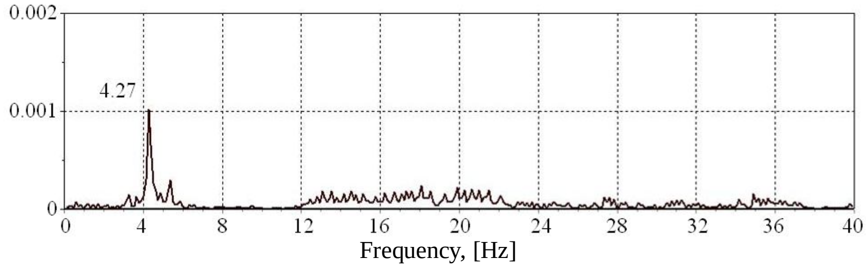


Fig. 8 – Fourier spectrum recorded with the *S*5 transducer on the longitudinal direction.

The first three modes of vibration have been identified using the identification program ModalView. This program is based on the Stochastic Subspace Identification technique (Yu & Ren, 2005).

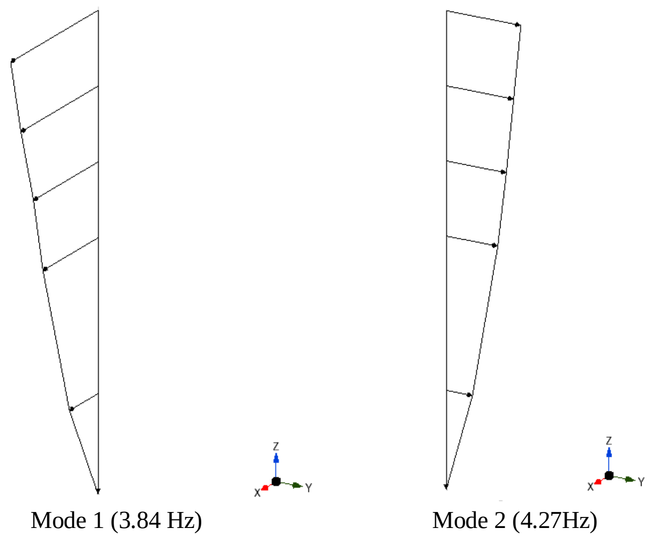


Fig. 9 – Vibration modes identified through measurements on the vertical.

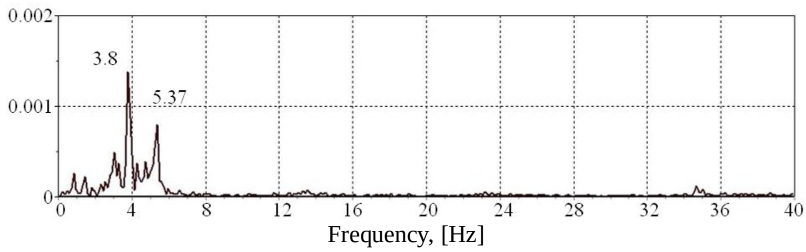
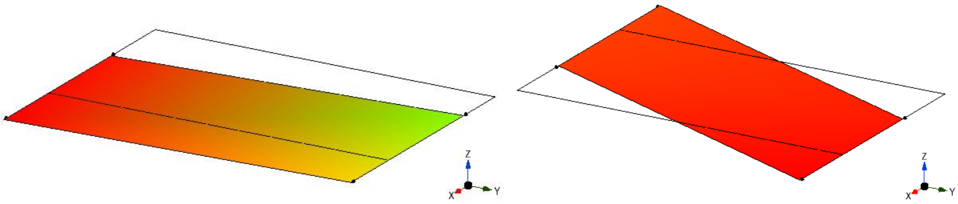


Fig. 10 – Fourier spectrum recorded with the *S*1 transducer on the transversal direction.

The first and third modes of vibration with frequencies of 3.8 Hz and 5.37 Hz, respectively, can be observed in the spectrum recorded on the transversal direction in the horizontal plane. The spectrum is shown in Fig. 10.



Mode 1 (3.8 Hz) Mode 3 (5.37 Hz)

Fig. 11 – Vibration modes identified through measurements in the horizontal plan.

Fig. 11 exhibits the identified modes of vibration for the first and third mode. Mode *1* presents the twisting of the structure due to its structural irregularity and, as expected, mode *3* is represented by torsion.

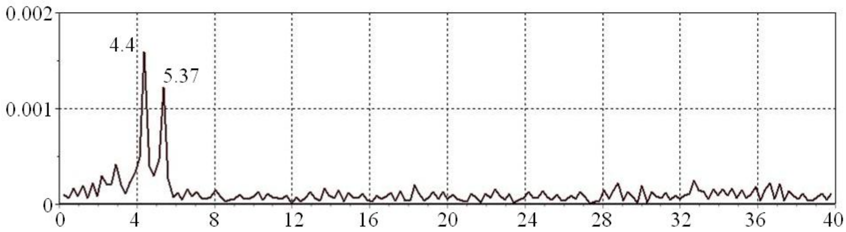
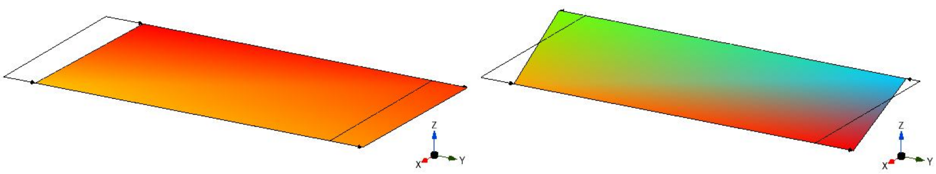


Fig. 12 – Fourier spectrum recorded with the *S*1 transducer on the longitudinal direction.



Mode 2 (4.4 Hz) Mode 3 (5.37 Hz)

Fig. 13 – Vibration modes identified through measurements in the horizontal plan.

The three dimensional renderings in Fig. 13 show the effect of torsion in the second mode of vibration, due to the structural irregularity, effect that is also present in the longitudinal direction.

The first three vibration modes have been identified from the experimentally measured data. To validate the accuracy of the identified vibration modes their orthogonality has to be analysed using the Modal Assurance Criterion (MAC) (Allemang,2003), when the identification of the frequency domain technique is based only on the response, without knowing the excitation source.

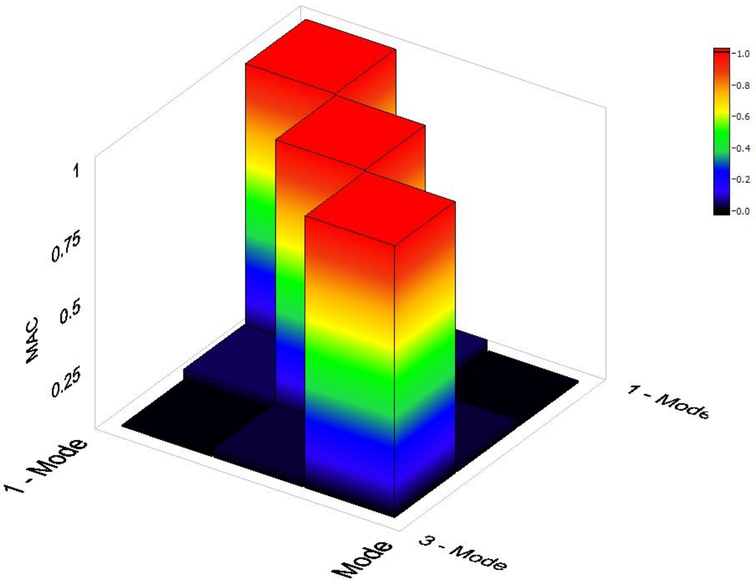


Fig. 14 – Mode shapes (MAC values).

The results based on the MAC analysis (Fig. 14) reveal the orthogonality of the three vibration modes and that the maximum value of the criterion for modes *1* and *2* is 0.044. The obtained value is less than 0.1, the minimum allowed value for the orthogonal modes.

3.3. Computation Model

The computation model is based on the finite element method performed with the program ETABS. 10,740 nodes, 5,751 “bar” type elements (girders, columns) and 10,602 “shell” type elements (walls, slabs, foundation raft) have been used in the structural simulation. The three dimensional model of the structure is presented in Fig. 15.

In the computation model the supports stiffness has been considered as infinite and the influence of the foundation ground was not taken into account.

Due to the reduced level of the vibrations, the behaviour of the structure was linear and the non-structural elements (separating walls, woodwork) had an influence on the period and the dynamic characteristics of the structure.

The stiffness of the foundation ground is an important factor which can exert an influence on the structures dynamic characteristics. This effect can be analysed from the shapes of the vibration modes identified in the dynamic measurements.

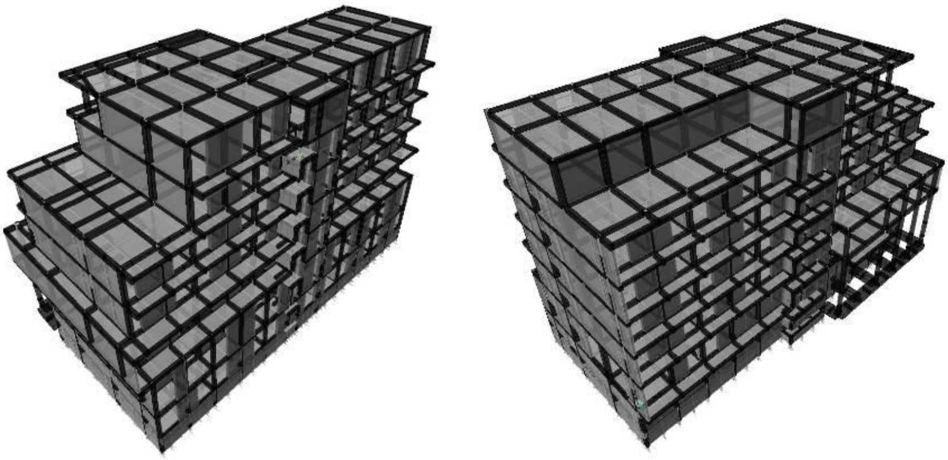


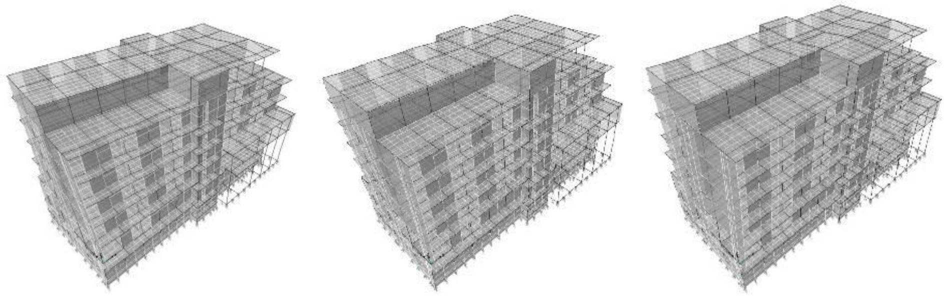
Fig. 15 – Computation model.

To benefit from a more realistic result, the separating walls (represented with grey colour, Fig. 15), made of brick masonry, have been taken into account in the computation. The influence of the plaster and of the woodwork has been considered as well. The loads applied in the computation of the modal mass have been evaluated only from the self weight of these elements.

To determine the equivalent elastic modulus for the masonry in the separating walls two computation models have been adopted: one with the elastic modulus of 5,000 N/mm^2 and one with the elastic modulus of 1,000 N/mm^2. The basic idea consisted in the analysis of the measured frequencies and their comparison with the frequencies of the two computation models. Following, through linear interpolation the equivalent elastic modulus of the walls on the two directions corresponding to the measured frequencies has been determined.

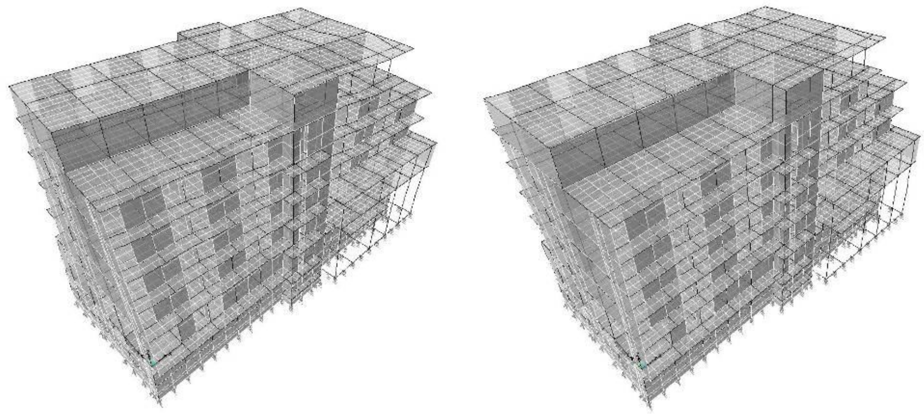
Fig. 16 illustrates the modes of vibration for the model with the value of the elastic modulus of 5,000 N/mm^2.

From the analysis of the computed vibration modes has been detected that the frequencies for the three identified modes range between the values of the two adopted models. Thus, through linear interpolation the equivalent elastic modulus of the masonry in the separating walls on each main direction has been determined. After the interpolation, the calibration of the computation model is performed through the modification of the elastic modulus of the separating walls so the frequency is almost equal with the one obtained in the measurements. Two of the corrections applied are presented in Fig. 17.



Mode 1 (3.921 Hz) Mode 2 (4.862 Hz) Mode 3 (6.233 Hz)

Fig. 16 – Vibration modes for $E_z = 5{,}000$ N/mm^2.



Mode 1 (3.84 Hz), E_z = 4,565 N/mm^2 Mode 2 (4.27 Hz), E_z = 1784 N/mm^2

Fig. 17 – Corrected modes of vibration.

3.4. Mode Shape Comparison

The correction applied to the computation model with respect to the experimentally determined dynamic characteristics using finite elements resulted in similar vibration frequencies to the ones determined in situ. The shape modes of the first two vibration modes have been analysed and normalized to unit to perform the comparison (Tables 1 and 2).

Table 1
Comparison for the First Vibration Shape Mode

Mode *1* (3.48 Hz)	Finite element	Experiment	Difference, [%]
Terrace	1.000	1.000	0.00
5^{th} floor	0.941	0.878	6.70
4^{th} floor	0.755	0.734	2.73
3^{rd} floor	0.594	0.627	5.52
1^{st} floor	0.274	0.324	18.27

Table 2
Comparison for the Second Vibration Shape Mode

Mode *2* (4.27 Hz)	Finite element	Experiment	Difference, [%]
Terrace	1.000	1.000	0.00
5^{th} floor	0.847	0.904	6.70
4^{th} floor	0.713	0.806	13.02
3^{rd} floor	0.550	0.691	25.65
1^{st} floor	0.203	0.354	74.13

The analysis of the vibration modes obtained *in situ* disclosed that in mode *1*, on the transversal direction, the two shapes are close, with a maximum difference recorded at the first floor.

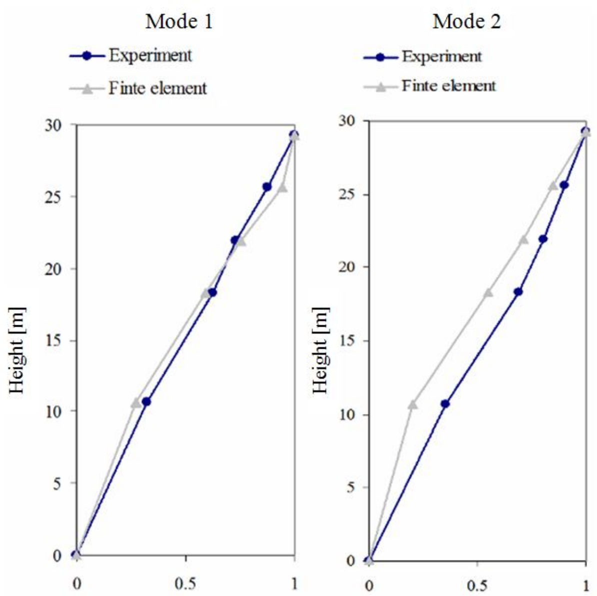


Fig. 18 – Comparison of the vibration shape modes.

The graphs in Fig. 18 present the variation of shape modes resulted from the experimental measurements, finite element analysis and linear shape.

The *in situ* measurements are closer to the linear shape. On the longitudinal direction (mode *2*) higher differences between the two modal shapes are present, the maximum difference of 74.13% also present at the first floor.

In the finite element model there is a more reduced horizontal stiffness of the structure when compared to the experimental measurements performed on the floors 1 to 5. The experimental shape modes presented a closer horizontal stiffness for the floors superior to the first level than for the ones below.

4. Conclusions

The experimental dynamic measurements joined with the dynamic identification of the structures have an important character in the constructions assurance level. The vibrations of the structures give information on the vibration periods and shapes, the uncontrolled interactions with the nearby buildings and the real dynamic behaviour of the structure.

The shape modes obtained from the experimental measurements on the transversal (mode *1*) and longitudinal (mode *2*) direction and the ones computed with the finite element analysis are close; the maximum difference was recorded on both directions at the first floor. In the finite element model the horizontal stiffness of the structure (from the first floor up) is more reduced than the one determined *in situ*.

For a reduced level of the vibrations the structures have an elastic behaviour and the non-structural elements increase their stiffness. For a reinforced concrete frame building subjected to low vibrations (ambient vibrations) the masonry separating walls highly influence the period of vibration.

Increasing the stiffness of the separating walls and taking into account the stiffness of the foundation soil a higher proximity can be reached between the analytical and experimental shape modes.

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CALIBRAREA MODELULUI DE CALCUL A UNEI STRUCTURI DIN BETON ARMAT ÎN FUNCȚIE DE CARACTERISTICILE DINAMICE DETERMINATE EXPERIMENTAL

(Rezumat)

Caracteristicile dinamice ale unei structuri variază în funcție de masa şi de rigiditatea laterală a acesteia. Se propune o metodă de îmbunătățire a modelului de calcul urmărind realizarea unei corelații între caracteristicile dinamice obținute cu ajutorul modelului de calcul bazat pe metoda elementelor finite şi caracteristicile dinamice ale structurii determinate experimental. Modelul cu elemente finite a fost realizat cu ajutorul programului ETABS, iar caracteristicile dinamice experimentale au fost obținute pentru un nivel scăzut al vibrațiilor. Pentru captarea *in situ* a răspunsului dinamic s-au utilizat simultan cinci seismometre Kinemetrics, dispuse pe direcție transversală şi longitudinală. Identificarea dinamică a fost realizată cu programul de identificare dinamică ModalVIEW, utilizând metoda identificării aleatoare a subspațiului. Calibrarea modelului de calcul cu elemente finite s-a realizat prin modificarea modului de elasticitate longitudinal al zidăriei din cărămidă a pereților despărțitori.