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Ambient and forced vibration testing with numerical identification for RC buildings

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Abstract. Reinforced concrete buildings constitute the majority of the building stock of Turkey and much of them, do not comply the earthquake codes. Recently there is a great tendency for strengthening to heal their earthquake performance. The performance evaluations are usually executed by the numerical investigations performed in computer packages. However, the numerical models are often far from representing the real behaviour of the existing buildings. In this condition, experimental modal analysis fills a gap to correct the numerical models to be used in further analysis. On the other hand, there have been a few dynamic tests performed on the existing reinforced concrete buildings. Especially forced vibration survey is not preferred due to the inherent difficulties, high cost and probable risk of damage. This study applies both ambient and forced vibration surveys to investigate the dynamic properties of a six-story residential building in Istanbul. Mode shapes, modal frequencies and damping ration were determined. Later on numerical analysis with finite element method was performed. Based on the first three modes of the building, a model updating strategy was employed. The study enabled to compare the results of ambient and forced vibration surveys and check the accuracy of the numerical models used for the performance evaluation of the reinforced concrete buildings.

Keywords: ambient vibration; forced vibration; model tuning; reinforced concrete; experimental research

1. Introduction

In Turkey, more than 90% of building stock is consisted of reinforced concrete buildings. The majority of Turkey's urban population lives in multi-story apartment blocks constructed with reinforced concrete (Kaltakçı *et al*. 2007). During last earthquakes, a large number of these buildings in the epicentre regions were collapsed leading to widespread destruction and loss of life (Arslan and Korkmaz 2007). Consecutive earthquakes in Ercis and Edremit in Turkey caused 35,000 damaged or collapsed buildings in the city centre and the surrounding villages. In the area where the earthquakes occurred, almost all the reinforced concrete buildings were affected (Ateş *et al*. 2013). For these reasons, the general tendency is strengthening the existing reinforced concrete building stock which is not fulfilling the earthquake code requirements.

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Like in retrofit projects, numerical models are frequently constructed to investigate the all civil engineering problems related to design and assessment issues. Inherent characteristics of a structure are hidden in the dynamic properties of it since these properties are independent from external effects such as loading and they are related to the structure's stiffness, mass and support conditions. In that respect the modal properties such as modal periods and mode shapes are the key parameters to predict the structural behavior. However, there are always some uncertainties in modeling stemming from generalizations, assumptions and ignorance. At this point experimental modal analysis is frequently applied to determine the dynamic parameters of existing structures. Many applications of vibration analyses in the literature can be found for different types of structures (Nateghi 1997, Gentile and Saisi 2007, Altunışık *et al*. 2011, Aras *et al*. 2011, Sevim *et al*. 2012). Specifically in reinforced concrete buildings, vibration surveys were used for period estimations, damage surveys and detection of change in structural behavior against different effects (Güler *et al*. 2008, Ivanovic *et al*. 2000, Soyöz *et al*. 2013, Tischer *et al*. 2012, Arslan and Durmuş 2013).

In this study, Ambient Vibration Survey (AVS) and Forced Vibration Survey (FVS) were performed separately to obtain the dynamic behaviour of a reinforced concrete building. During the measurements, the building was unoccupied. FVS was performed by applying impact loads on the fourth story level of the building via construction demolishing machine. When the building was vibrating, the acceleration records were gathered. The first three mode shapes, modal frequencies and damping values were obtained and the consistency of the AVS and FVS were discussed with the noted differences in the dynamic properties.

Numerical dynamic analysis was also performed with mathematical model of the building constructed in SAP2000 (2012). Eigen value analysis was performed and the dynamic parameters of the building were obtained numerically. The results have been compared to those obtained experimentally and model tuning with two simple steps was implemented. The study resulted with the necessity of vibration survey for the evaluation of existing reinforced concrete building.

2. Characteristics of the studied building

The studied building is a representative of general apartment style reinforced concrete buildings, and was built in 1983 as a residential building. This type of buildings is known as twin flat apartment buildings and frequently constructed with its one axis of symmetry in Turkey (Fig. 1). The building has a partially embedded basement story surrounded by shear walls, a ground story and four ordinary floors. The story height of ground and ordinary floors is 2.90 m while the basement story has a height of 4.4 m and is used as depot to store heating and other equipment.

After the Kocaeli and Düzce Earthquakes, the earthquake performance of the building was evaluated by a team of scientific researchers and a detailed material, soil and earthquake performance investigation was carried out (Haktanır *et al*. 2000). Concrete class and steel grade were specified as C8 and S220 respectively and the earthquake performance was found as inadequate. The building was retrofitted by the addition of shear walls, jacketing of two columns and conversion of spread footings to the mat foundation. However during a previously completed research, the shear wall application has been removed back from the building by concrete sawing. As a result, the mentioned study was performed for the original form of the building except the rigid foundation and two jacketed columns seen with the enlarged dimensions on the plan lay out (Fig. 1).

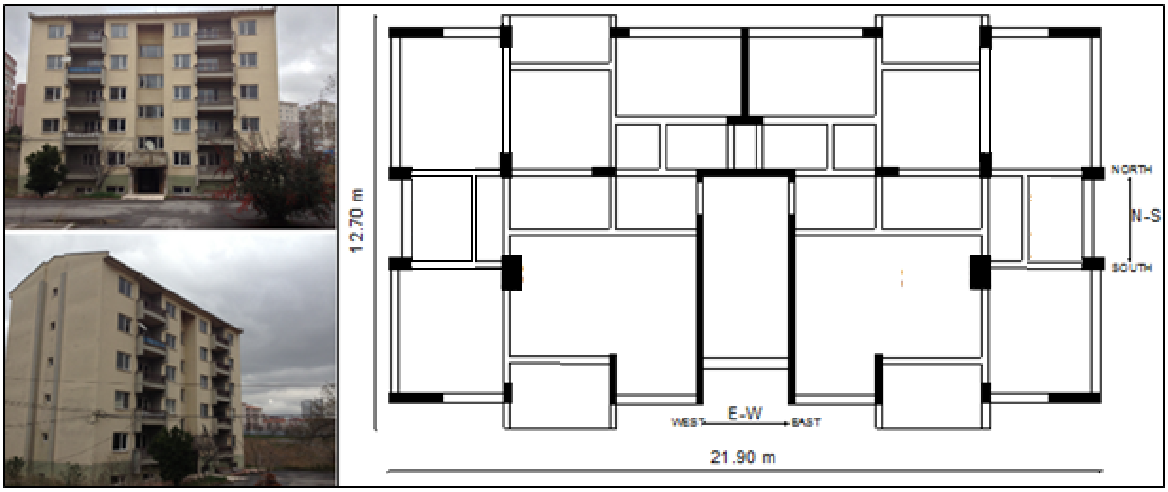


Fig. 1 Studied building and its plan lay-out

The structural system of the original building contained shear walls, columns and beams with one and two way beam supported slabs. The shear walls thickness was determined as 20 cm and 30 cm * 70 cm rectangular cross-section was used for the all exterior beams while the interior beam dimensions are 20 cm * 70 cm. Column dimensions vary between 20 cm and 160 cm.

The land of the building was transferred to the new established state university, Istanbul Medeniyet University and the building was planned to be demolished for the formation of new campus facilities. This event gave the chance of performing ambient and forced vibration analysis together.

3. Ambient vibration survey

The ambient vibration testing method is a widely applied, popular, full-scale testing method for the experimental definition of structural dynamic characteristics. This fast and relatively simple procedure is consisted of real time recording of the vibrations and processing of the records and used in this study.

3.1 Measurement of the vibrations

The ambient vibration measurements on the existing structure were performed on 13.01.2014. There was no traffic during the measurements. Five accelerometers containing three Colibrys type sensor for E-W, N-S and vertical directions were used. The linear full acceleration range of each sensor is ±3 g and its noise level is 0.3 μgrms/√Hz. (www.colibrys.com). One of the accelerometers was used as reference sensor and located on the centroid of the first floor permanently while the other four were travelled for story by story measurements by locating on the four corners of each story. Hence, five set of measurements were taken through the height of the building. Since the roof level was closed, no measurement could be taken. Each set of measurements contain 200 samples per second data with 20 minutes duration.

3.2. Analyses of the obtained measurements

For the modal parameter estimation of a structural system many signal processing techniques have been developed. These techniques are ranging from frequency domain algorithms based on the Fourier transform, such as peak pick and frequency domain decomposition, to time domain algorithms, such as the Eigensystem realization algorithms and the stochastic system identification (Min and Sun 2013, Peeters and De Roeck 2001). In that respect, the literature on modal identification methodologies is vast and a survey of the methodologies is out of the scope of this study. In this study, peak picking method was used to obtain the dynamic characteristics and the measurements were analysed by using Matlab computer program (2012). Basically the frequency domain representation for each measurement was obtained to see the dominant frequencies. Welch method is used with "pwelch" function of the software. No filtering was applied to the data. The analyses were carried out between 0 and 20 Hz, which is supposed adequate for modal identification of the studied building. Fig. 2 displays the power spectral densities in E-W (longitudinal) and N-S (transversal) direction measured on the fourth floor of the building. Frequency domain presentations of the other signals have same dominant frequencies with changing power values.

The analysis of the acceleration records resulted with identification of three dominant frequencies (peaks in frequency domain) in each direction. The mode shapes were identified by scaling the square root of the power spectral density magnitudes by reference accelerometer (Inman 2013). The first mode of the building was identified as the simple movement along with E-

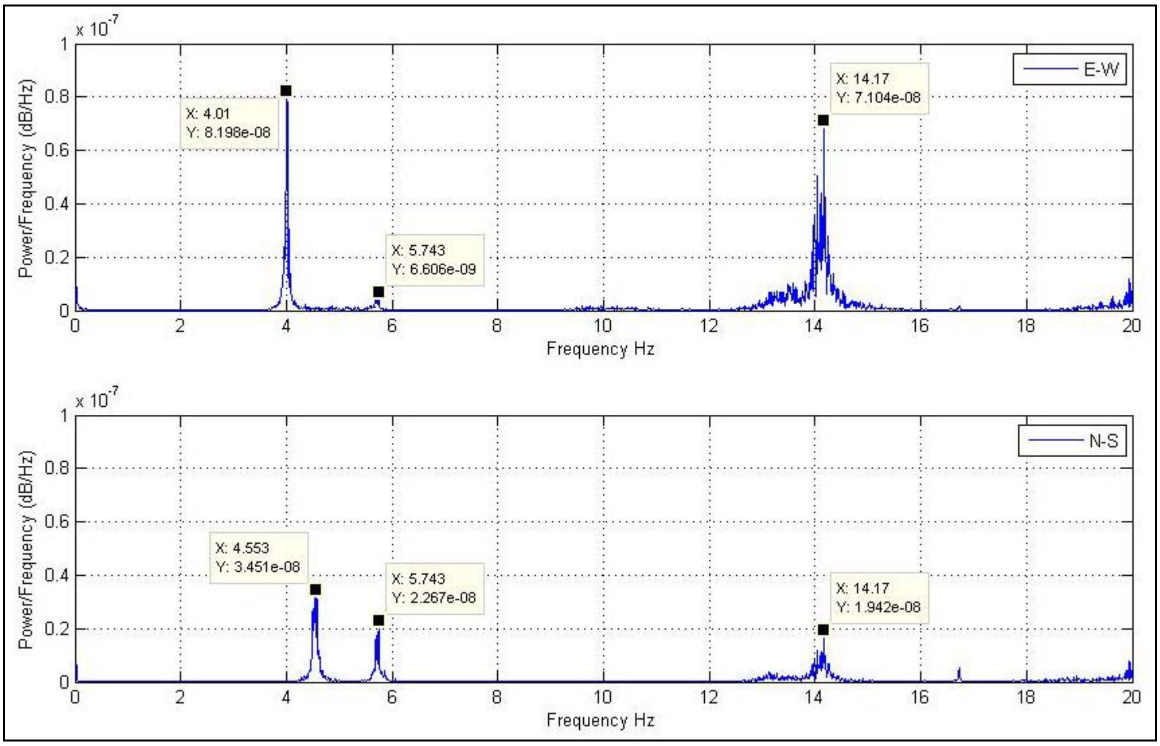


Fig. 2 Dominant frequencies in E-W and N-S directions obtained from AVS

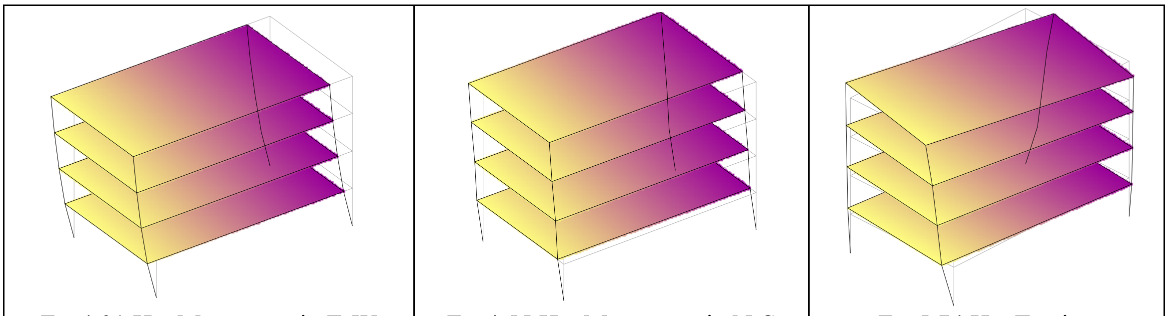


Fig. 3 Experimentally obtained mode shapes (AVS)

W direction with a frequency of 4.01 Hz while the second mode is the simple movement along with N-S direction with a frequency of 4.55 Hz. Finally the third mode is the torsional mode with 5.74 Hz dominant frequencies (Fig. 3).

4. Forced vibration survey

The forced vibration measurements on the building were performed on 23.01.2014. A set of impact load were applied to the building by a construction demolishing vehicle (Fig. 4), After a five-minute data acquisition in existing conditions, the first impact was applied to the building and then four more impacts were applied with two minutes interval. Total record time was set to 20 minutes. Figs. 5 and 6 shows the acceleration records and close looks of impacts measured on the fourth floor of the building in E-W and N-S directions respectively. The magnitude of impacts can be as high as 0.35 m/s^2.

Dominant frequencies of the building were obtained by analyzing the complete vibration records and power spectral density spectrums are illustrated in Fig. 7 for E-W and N-S directions. The reduced level of the noise and wider hills with higher energy were detected at the first glance. The first dominant frequencies were identified as 3.92 Hz in E-W direction and 4.42 Hz in

Fig. 4 Application of the impact loads to the building

N-S direction. Like in AVS, the mutual frequency corresponds to the second peak in each spectrum with 5.59 Hz dominant frequency. It can be concluded that, FVS gave the same mode shapes as AVS. Secondly, FVS gave the modal frequencies 2-3 % less than AVS did. Gentile and Saisi (2013) reached the same conclusion after testing historical masonry bell tower. They applied vibration survey under ambient conditions and when the bell of the tower was oscillating. The decrease in the modal frequencies was reported between 1.7% and 2.7% as the result of the increase in the vibration magnitude resulting from the bell oscillation.

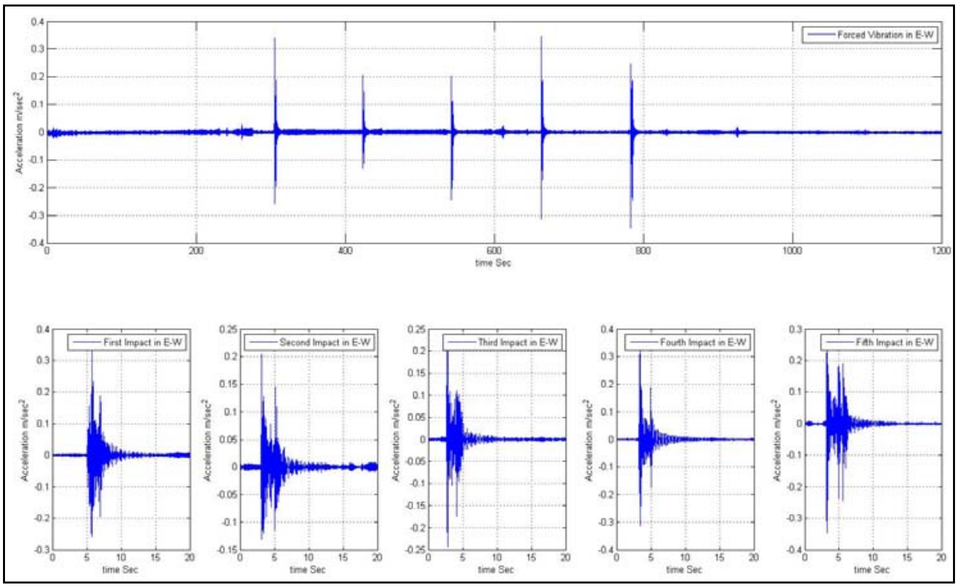


Fig. 5 Recorded vibration data and the impact details along with E-W directions

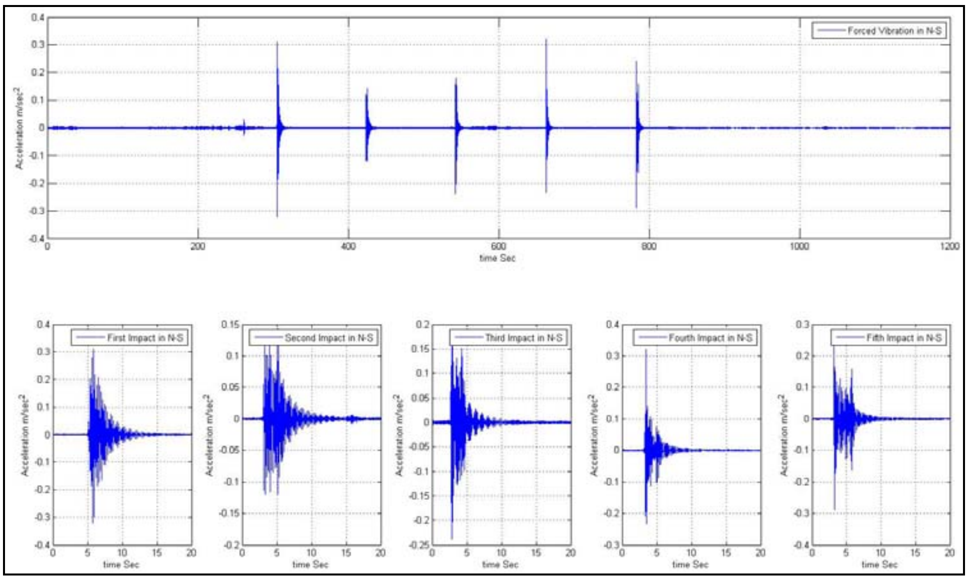


Fig. 6 Recorded vibration data and the impact details along with N-S directions

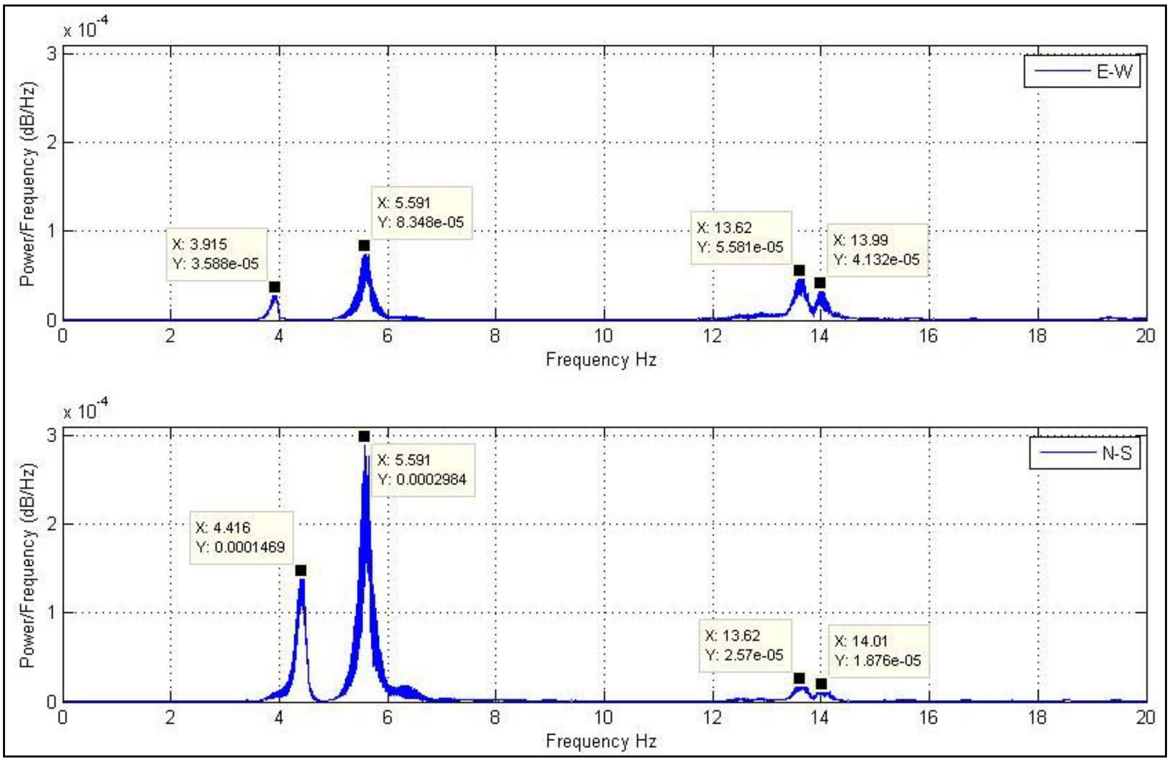


Fig. 7 Dominant frequencies in E-W and N-S directions for the original building (FVS)

5. Evaluation of damping

Another important property in determining the dynamic response of structures is the damping ratio. It is commonly determined by the half-power bandwidth method based on the frequency response curve measured from vibration tests. The half-power bandwidth method originates from the frequency response of a single degree of freedom system (Chopra 2007) and has the advantage of simplicity in application. The method is based on interpreting three values of frequency, of which there are three characteristic points corresponding to extreme of function and points lying at the height of $(1/2)^{1/2}$ extreme. On the basis of interpreted frequency, damping, ξ was calculated by the Eq. (1) (Fig. 8)

$$\xi = \frac{w_2 - w_1}{2w_0} \tag{1}$$

Based on the AVS, the damping ratio for the frequencies of 4.01 Hz, 4.55 Hz, and 5.74 Hz were calculated as 0.5%, 0.2% and 0.2% respectively. Since the vibration level is too low, the uncertainty in these values is high and damping ratios identified from the ambient vibration surveys are not used in the seismic performance assessment of the structures. Wider hills seen in the frequency spectrums of FVS are the indication of greater damping values. The damping ratios for the all frequencies were calculated as 1.1%. Impact details in Figure 5 and 6 illustrate the diminishing of the vibration, but they include the vibration of all modes.

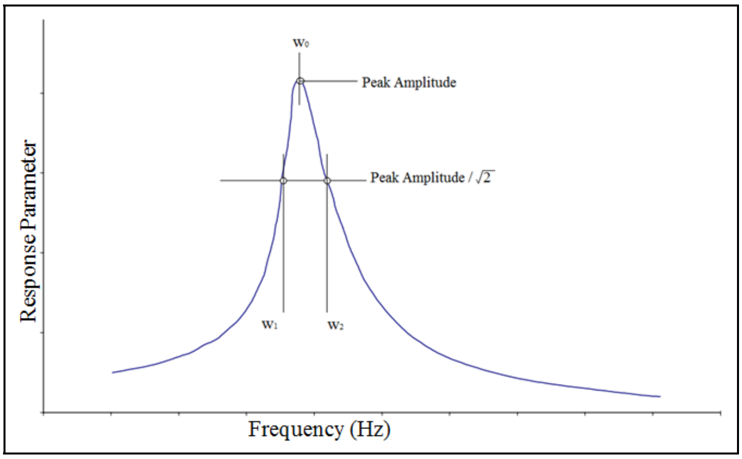


Fig. 8 Half power width method for the determination of damping

6. Numerical analysis and model tuning

Finite element models (FEMs) are usually constructed to estimate dynamic properties. For the modelling of an existing reinforced concrete structure, the most important parameters are geometry, mass, boundary conditions and material properties. In this study, the geometry of the building with the structural and non-structural elements was available and the necessary controls were performed thereby the existing mass was also checked. The mat foundation justified the assumption that the superstructure of the building is a fixed based one. Moreover, the vibration measurements taken from the building and ground showed no relation. Finally the concrete class of the building was identified by general concrete testing procedure as 8 MPa. However, limited number of coring (three cores per story) and the expected heterogeneity in concrete properties are worth to note. In that respect, the modulus of elasticity of the concrete can be determined by code given formulas, the value is doubtful. Partition walls are the second materials that its modulus of elasticity should be specified but no data is available for them.

The numerical model of the building was constructed in SAP2000 (2012). Structural walls and floor slabs were modelled using shell elements, whereas frame elements were used for the beams and columns. The walls and columns were assumed to be restrained at the foundation level with full fixity. Initially the elastic modulus concrete was estimated by Eq. (2) (ACI-318, 2008) which relates the compression strength of concrete (f_c) to its modulus of elasticity (E) as 13435 MPa

$$E = 4750\sqrt{f_c} \qquad \text{(in MPa)} \tag{2}$$

The unit weight of reinforced concrete was assigned as 25 kN/m^3 for the structural members (beams, columns, walls, and floor slabs). A distributed weight and mass was assigned to the slab to account extra covering and plasters and a distributed roof weight of 0.75 kN/m^2 was also assigned. Partition walls were not included into the model initially.

Eigen value analysis was performed and the first three modes of the building were determined as the torsional mode (1.90 Hz), simple movement along with E-W direction (2.16 Hz) and the simple movement along with N-S direction (3.08 Hz) respectively. When these results were compared to those obtained experimentally, mode order difference and lower stiffness were clearly noted. The difference also indicates the necessity of model tuning. The alterations in the initial numerical model of the building can be summarized in two themes, i.e., inclusion of the partition walls with an acceptable modulus of elasticity and questioning the correctness of the modulus of elasticity of concrete determined from material survey. These two subjects are handled below as two different steps of model calibration. Model tuning has been performed according to FVS.

6.1 The first step of the model tuning, effects of the partition walls

Since the experimental modal analysis was performed under small amplitude vibrations, the determined mode shapes also includes the effects of the partition walls. Consequently, the infill walls should also be added into the numerical model. However, for the studied building' partition walls no material data was available. According to wide literature survey, the modulus of elasticity of the hollow-brick partition walls can vary between 1000 and 8000 MPa (Köse 2009, Baloevic *et al*. 2013, Cavaleri and Trapani 2014, Lourenco *et al*. 2010). In this study, the modulus of elasticity of the partition walls was selected such that it reveals the experimental mode shapes. In other words, the lack of information about the modulus of elasticity of the partition walls gave an opportunity to select an acceptable value for a better representation of experimental results.

In conventional models, the infill walls are included into models by diagonal elements (struts) which are effective under compression loads. This modelling technique is appropriate for earthquake performance evaluation. However, it is not straightforward due to existence of openings. Modelling of partition walls with shell elements to investigate the dynamic behaviour is more practical. Therefore, the partition walls were modelled by shell elements in this study. Fig. 9 shows the numerical model of the studied building with and without the partition walls.

The effects of the partition walls on the dynamic behaviour of the building were searched by increasing the modulus of elasticity of them between 200 MPa and 14000 MPa. In order to observe the changes in the modal frequencies and the mode shapes Figure 10 was drawn. The first step of the model tuning was devoted to determine the acceptable modulus of elasticity for the partition walls which gives correct mode shapes order (obtained in the experimental study). In this step the ratio of the frequencies for the numerical analysis should be close enough to that of experimental analysis for the first three modes. In this condition scaling of the modulus of

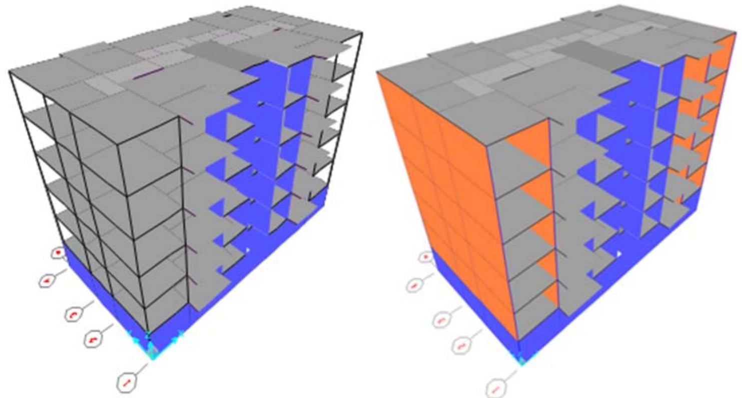


Fig. 9 Numerical model of the original building without and with partition walls

elasticity for concrete and partition walls in the numerical model constitutes the second step of the calibration.

The target value for the modulus of elasticity of partition walls is the one giving the first mode shape as the transition in E-W, the second mode as the transition in N-S and third mode as the torsional mode. Moreover, these values should also give the ratio of the second mode frequency to the first mode frequency as 4.42/3.92=1.128 and the ratio of the third mode frequency to first mode frequency as 5.59/3.92=1.426 in order to make the second step of the calibration to correct the numerical model completely according to the FVS.

The evaluation of Fig. 10 showed that mode order disagreement was corrected after at 1750 MPa of modulus of elasticity for the partition walls. After that value the ratio of the frequency of second mode to that of first mode varies between 1.38 and 1.46, and the ratio of the frequency of third mode to that of first mode varies between 1.4 and 1.76. In these conditions, selection of an appropriate modulus of elasticity conforming two desired ratio of frequencies is impossible because the second mode frequency was obtained as a higher value than the expected. In this condition the modulus of elasticity of the partition walls was determined as 2250 MPa to catch the desired frequency ratio for the third and first modes.

Eigen value analysis of the obtained numerical model after the first step of the calibration (E=13435 MPa, E_{PW}=2250 MPa) showed that the first mode of the building is the transition mode in E-W direction with a frequency of 3.32 Hz, the second mode is the transition mode in N-S direction with a frequency of 4.6 Hz, and the third mode is the torsional mode with a frequency of 4.75 Hz. An additional step should be incorporated to adjust the frequency values of the numerical model to those of experimental study since the first calibration step just adjusted the mode order and the ratio of the frequencies for the first and the third modes.

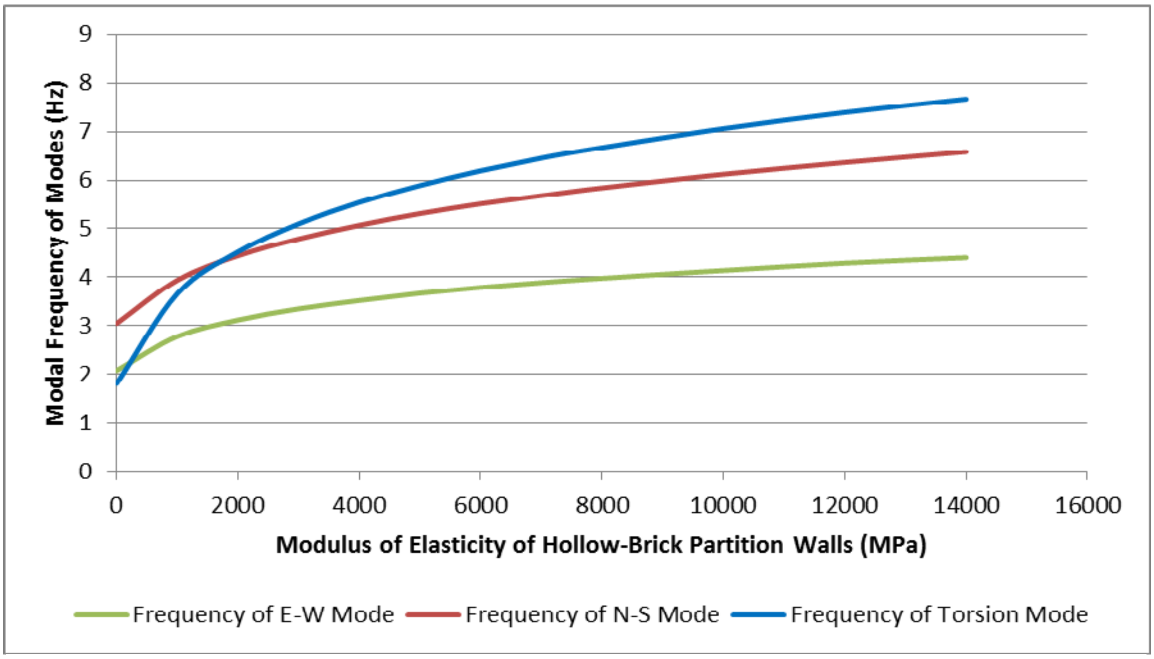


Fig. 10 Variation of modal frequencies with respect to modulus of elasticity of the partition walls in numerical model

6.2. Second step of the modal tuning, Young modulus of concrete and partition walls

The second step of the calibration was required to conceal the discrepancies between the modal frequencies of the numerical model and those of experimental study. A basic alteration in the modulus of elasticity of the all material in the numerical model can equalize the modal frequencies since the first step of the calibration has arranged the mode order and frequency ratios as appropriate as possible. In that respect a calibration coefficient was defined and applied to the numerical model to determine the modulus of elasticity of the concrete and partition walls. Due to the well-known relationship presented in Eq. (3) (Chopra 2007), the calibration coefficient, C was calculated as the square of the ratio of the frequencies as shown in Eq. (4)

$$f = 2\pi\sqrt{\frac{k}{m}} \tag{3}$$

$$C = \left(\frac{f_E}{f_N}\right)^2 \tag{4}$$

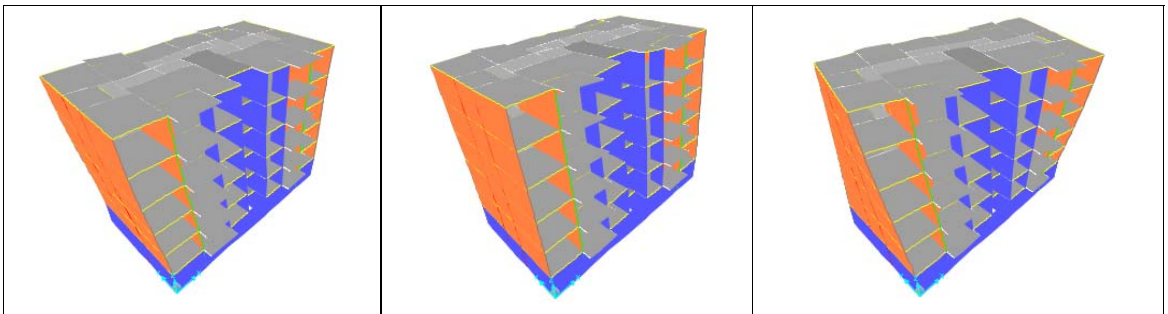
where, f is frequency, k is the stiffness, m is mass, f_E is the frequency obtained from experimental study and f_N is the frequency obtained from numerical analysis. Representing the stiffness term the modulus of elasticity of the all material in the numerical model should be decreased by calibration coefficient which can be computed as $(3.92/3.32)^2=1.39$. As the results of the second step of the model tuning, new numerical model was formed with concrete with 18670 MPa and partition walls with 3125 MPa.

Eigen value analysis of the calibrated model showed that the first mode of the building is the transition in E-W direction with a frequency of 3.92 Hz, the second mode is the transition mode in N-S direction with a frequency of 5.42 Hz and the third mode is the torsional mode with a frequency of 5.59 Hz. Table 1 shows the numerical values for first ten modes.

The comparison of the calibrated numerical and experimental study shows that the first and the third mode shapes and their frequencies were estimated well while the second mode frequency was overvalued. The general characteristics of the building, such as, old age, low strength concrete, and having experienced with strong earthquakes, make it impossible to obtain the exact frequencies and mode shapes for the first three modes of the building by single concrete and partition wall mechanical properties. Moreover, concrete is not a regular material and its properties may differ in various places. Besides, the use of different concrete properties for different elements in the numerical model is not practical. In these respects, the author does not intent to perform additional tuning steps and assumed that the calibrated numerical model reflects dynamic behaviour of the building well. As seen in Table 1, numerical model gives the frequency of the higher modes beyond 10 Hz as expected from the results of the experimental dynamic analysis.

If the tuning of the numerical model was performed according to results of AVS, the final modulus of elasticity of concrete would be 19600 MPa and the final modulus of elasticity of partition walls would be 3280 MPa.

The comparison of the calibrated numerical model and the rough model of the building proves the importance of experimental modal analysis for the validation of numerical model of existing buildings. Two numerical models displayed completely different structural behaviour as presented. Moreover, the modulus of elasticity of the concrete in the building was 1.39 times greater than the value determined by concrete coring. In other words, the material investigation performed by



F_1=3.92 Hz, Movement in E-W | F_2=5.56 Hz, Movement in N-S | F_3=5.59 Hz, Torsion

Fig. 11 Numerically obtained mode shapes with tuned numerical model

Table 1 Dynamic properties of the building obtained from the calibrated numerical model

Mode	Numerical Model without Partition Walls			
	Frequency	**Period**	**Mass Participation**	
	Hz	Sec	X	Y
1	3.915	0.255	0.688	0.000
2	5.424	0.184	0.000	0.683
3	5.594	0.179	0.011	0.000
4	12.899	0.078	0.099	0.000
5	16.800	0.060	0.003	0.000
6	17.297	0.058	0.000	0.139
7	18.207	0.055	0.000	0.002
8	18.588	0.054	0.000	0.000
9	21.451	0.047	0.001	0.001
10	21.615	0.046	0.003	0.004

concrete coring (in 2001) is misleading. When the compressive strength of the concrete is re-calculated by Eq. (2) with the identified modulus of elasticity of concrete, it was found as 15.45 MPa.

7. Conclusions

As a full scale experimental research with a numerical identification the performed study was resulted with the following findings.

• The performed AVS and FVS gave the same mode shapes. It was noted that, the natural frequencies decreases as the excitation magnitude increases. Determined modal frequencies from FVS were 2-3% less than those obtained from AVS. For this reason, as a practical and easy testing, AVS can predict the modal properties accurately.

• AVS results indicated that the damping ratios for the first three modes are 0.5%, 0.2% and 0.2% respectively while FVS gave them 1.1% for each mode. These finding comply the general agreement related to damping which it increases with the magnitude of the vibration experienced by the building.

• Dynamic properties, obtained from conventionally constructed numerical model were different than those obtained experimentally. Model tuning showed that the modulus of elasticity of concrete was underestimated by the conventional material evaluation processes. The most appropriate numerical model requires 39% increase in the identified modulus of elasticity obtained according to ACI-318 as a function of compressive strength of concrete. Estimated as 13435 MPa, the modulus of elasticity of concrete was corrected as 18670 MPa by FVS and as 19600 MPa by AVS. As a practical and easy way AVS should be used to correct the numerical model of existing building.

• The modulus of elasticity of the partition walls of the building was also estimated. The tuning of the numerical model according to FVS revealed the modulus of elasticity of the partition walls as 3125 MPa while the tuning performed by AVS indicated it as 3280 MPa.

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