THE EFFECTS OF PARTITION WALLS ON DYNAMIC BEHAVIOR OF RC BUILDINGS

by

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B.S., Civil Engineering, Kocaeli University, 2013

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
the requirements for the degree of
Master of Science

Graduate Program in Civil Engineering
Boğaziçi University
2019

ACKNOWLEDGEMENTS

First of all, I would like to express my gratitude to my supervisor Associate Professor Serdar Soyöz who always helped me in my thesis process. His encouragement and endless patience motivated me to conclude my study.

I would also like to present my gratefulness to my co-supervisor Associate Professor Fuat Aras who accepted me for his project. Thanks to his support and contributions, I learned many things during the project in order to use it in my thesis.

I especially thank to Prof. Gülay Altay who helped me to take part in the project. Her guidance and support encouraged me to work.

I would like to show my appreciation to the members of my thesis jury for participating.

I owe special thanks to my close friends, workmates and Emrah Kurt for their great support.

My thesis has grown up with the support of the Scientific and Technological Research Council of Turkey (TUBITAK) under research Grant Number 214M235: Betonarme Yapılarda Bölme Duvarların Yapıların Dinamik Davranışlarına Etkilerinin Tam Ölçekli Ve Hasarlı Deneylerle Araştırılması. I am also thankful for the given support.

In conclusion, I must express the gratitude to my beloved members of my family for their endless support and unconditional love.

ABSTRACT

THE EFFECTS OF PARTITION WALLS ON DYNAMIC BEHAVIOR OF RC BUILDINGS

Reinforced concrete buildings constitute a significant part of the building stock of Turkey, and the investigation of their structural behavior is essential. This study aims to concentrate on the inclusion of infill walls into the numerical model of the studied building. For this reason, an existing six-story reinforced concrete building is considered as a reference structure to assess different modeling approaches defined for infill walls, such as shell and strut models. Dynamic properties of structures are inherent properties which depend on their mass and rigidity. Therefore these properties contain vital parameters for the assessment of structural behavior. In this study dominant frequencies and mode shapes of the building are determined by Ambient Vibration Testing. Acceleration records gathered from each story level are analyzed in the frequency domain with Peak Picking method. Modulus of elasticity, unit weight, compressive strength, and shear strength are determined by testing the wall specimens extracted from the existing building according to European Norms. Thereby the required data for the construction of a numerical model is completed. Numerical models for the studied building are constructed with finite element method by using different modeling techniques for infill walls. In the first model, the rigidity of infill walls is ignored as it is done in state-of-the practice. The second model is constructed by idealizing the infill walls by shell element. The results of these two models showed that partition walls significantly affect the dynamic behavior of the structures. Finally, two strut models are constructed separately as one directional single diagonal strut and two directional single diagonal struts for each infill walls. Eigen Value analyses performed for each model to analytically determine the modal parameters. Comparison of experimental and numerical results indicated the accuracy of modeling technique.

ÖZET

BÖLME DUVARLARIN BETONARME BİNALARIN DİNAMİK DAVRANIŞI ÜZERİNDEKİ ETKİLERİ

Betonarme binalar, Türkiye'deki bina stokunun önemli bir bölümünü oluşturmaktadır ve yapısal davranışlarının araştırılması önemlidir. Bu çalışma, incelenen binanın nümerik modeline dolgu duvarların dahil edilmesi üzerine odaklanmayı amaçlamaktadır. Bu nedenle, mevcut altı katlı betonarme bir yapı, kabuk ve çubuk modelleri gibi dolgu duvarlar için tanımlanan farklı modelleme yaklaşımlarını değerlendirmek için referans yapı olarak dikkate alınmıştır. Yapıların dinamik özellikleri, kütlelerine ve rijitliklerine bağlı doğal özellikleridir. Bu nedenle, bu özellikler yapısal davranışların değerlendirilmesi için çok önemli parametreler içermektedir. Bu çalışmada, binanın baskın frekansları ve mod şekilleri ortamsal titreşim testleri yapılarak belirlenmiştir. Her kat seviyesinden toplanan ivme kayıtları Peak Picking yöntemi ile frekans tanım alanında analiz edilmiştir. Mevcut binadan çıkarılan duvar numunelerinin Avrupa Normlarına göre test edilmesi ile elastisite modülü, birim ağırlık, basınç dayanımı ve kesme dayanımı belirlenmiştir. Böylece sayısal bir modelin oluşturulması için gerekli veriler tamamlanmıştır. İncelenen binanın nümerik modelleri, dolgu duvarlar için farklı modelleme teknikleri kullanılarak sonlu elemanlar yöntemi ile oluşturulmuştur. İlk modelde, dolgu duvarların rijitliği uygulamada olduğu gibi göz ardı edilmiştir. İkinci model, dolgu duvarlarının kabuk elemanı olarak idealleştirilmesiyle yapılmıştır. Bu iki modelin sonuçları bölme duvarların yapıların dinamik davranışını önemli ölçüde etkilediğini göstermiştir. Son olarak, her bir dolgu duvarı için tek yönlü ve iki yönlü tek çapraz çubuk olmak üzere iki çubuk modeli ayrı şekilde oluşturulmuştur. Dinamik modal parametrelerin analitik olarak belirlenmesi amacı ile, her model için öz değer analizleri yapılmıştır. Deneysel ve nümerik sonuçların karşılaştırılması, modelleme tekniğinin doğruluğunu göstermiştir.

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LIST OF SYMBOLS

A	Coefficient of thermal expansion
A_i	Loaded cross-section of an individual masonry specimen
a_{wall}	Width of the strut
b_w	Strut width
cm	Centimeter
dB	Decibel
E_c	Elasticity modulus of the frame concrete
E_i	Elasticity modulus of the masonry specimen
E_{wall}	Elasticity modulus of the partition wall
e	Distance between center lines of the mortar bed and the loading roller
F	Force applied to the specimen
$F_{i,max}$	Maximum load reached on an individual masonry specimen
f	Frequency
f	Mean compressive strength of the masonry
f_i	Compressive strength of an individual masonry specimen
$f_{i,min}$	The smallest compressive strength of an individual masonry specimen
f_k	Characteristic compressive strength of masonry
f_{vo}	Mean initial shear strength
f_{voi}	Shear strength of an individual sample
$f_{voi,min}$	The lowest shear strength of an individual sample
f_{vok}	Characteristic initial shear strength
G	Shear modulus
g	Acceleration
Hz	Heartz
h_1 *and* h_2	Heights of cut units
h_k	Height of the column
h_s	Height of the specimen

h_u	Height of unit brick
h_{wall}	Height of the partition wall
I_k	Inertia momentum of the frame column
kN	Kilonewton
k_{wall}	Axial rigidity of a diagonal compression strut element
l_s	Length of specimen
l_u	Length of unit brick
MPa	Megapascal
m	Meter
mm	Millimeter
N	Newton
r_{wall}	Diagonal length of the partition wall
$s\ and\ sec$	Second
T	Natural period of the structure
T_x	Fundamental period of the structure in x direction
T_y	Fundamental period of the structure in y direction
t_{bj}	Thickness of the bed joint
t_s	Thickness of specimen
t_s	Thickness of the steel loading plates
t_u	Thickness of unit brick
t_{wall}	Thickness of the partition wall
U	Poisson's ratio
ε_i	Mean strain of an individual masonry specimen
ϕ	Diameter of reinforcement
λ_{wall}	Coefficient of equivalent pressure bar
Θ	Angle between the diagonal and the horizontal axis

LIST OF ACRONYMS/ABBREVIATIONS

2D	Two Dimensional
3D	Three Dimensional
AVT	Ambient Vibration Test
DCR	Demand Capacity Ratio
EN	European Standards
EQ	Earthquake
FDD	Frequency Domain Decomposition
FEA	Finite Element Analysis
FEM	Finite Element Model
FFT	Fast Fourier Transform
FI	Full Infill
MAC	Modal Assurance Criterion
MDOF	Multi Degree of Freedom
OMA	Operational Modal Analysis
PGA	Peak Ground Acceleration
PI	Partial Infill
PP	Peak Picking
PSD	Power Spectral Density
RC	Reinforced Concrete
SDOF	Single Degree of Freedom
TEC	Turkish Earthquake Code
URM	Unreinforced Masonry

1. INTRODUCTION

1.1. Motivation

Partition (infill) walls are one of the most important non-structural elements for reinforced concrete buildings. Their effects on dynamic behavior of the buildings cannot be ignored because they significantly increase the stiffness of the buildings, so they directly affect the dynamic behaviors of the structures. On the other hand, generally they are not considered when creating 3D finite element models. In this respect, the effects of the partition walls into the dynamic behavior of reinforced concrete structures should be investigated with realistic experimental studies and supporting numerical analyses. Furthermore, there is a need for the evaluation of different modeling techniques of partition walls, such as shell and strut models.

Secondly, there is no satisfactory experimental research about the mechanical properties of partition walls. For their inclusion into numerical models, their modulus of elasticity, compressive strength and shear strength are in importance. In the literature, many experiments were carried out on test specimens created in laboratory conditions but they are far from representing the real behavior of partition walls in the buildings. This gap can be filled with in-situ testing of infill wall specimens or testing of real wall specimens extracted from real buildings in well-equipped laboratories.

The motivation of this thesis stem from the aforementioned needs. An existing six-story reinforced concrete building was used as a test specimen to evaluate the effects of partition walls on dynamic behavior of structures. Moreover, the efficiency of different infill wall modeling techniques is investigated. This goal requires some exact features of the building to be used in the evaluation of different modeling techniques. In this thesis, the dynamic properties of the studied building are grasped as the control parameters. They were determined by Ambient Vibration Testing (AVT) method which is a widely used practical full-scaled testing methodology. Numerically obtained dynamic properties, obtained with finite element models with a specific infill wall modeling, were compared to real dynamic properties of the building determined by AVT in order to evaluate the effectiveness of different infill wall models.

In order to leave the infill wall modeling technique as the unique questioned subject, the other parameters, such as, geometry, boundary conditions, existing mass and material properties should be determined and included into the numerical models with maximum precision. For this aim, an extensive testing program was followed for infill wall's mechanical properties. Wall specimens were extracted from the studied building and tested according to EN1052. The results were used in the numerical models.

The importance of this study is that a real world structure is used as test specimen. This situation allows obtaining more accurate results because data collected from the building reflects the real condition. Hence, the assumptions, ignorance or approximations did not take part in performed studies. In the constructed finite element models, real mechanical properties obtained from an extensive testing were used rather than the suggestions of the specifications.

1.2. Literature

In the literature, many studies were carried out about the effects of partition walls on dynamic behaviors of RC buildings. Some of these studies are summarized below in chronological order:

Shing and Mehrabi (2002) [1] studied on behavior and analysis of masonry infilled frames. Although infill walls are frequently used in structures, they are considered as non – structural elements and their connection with frames are ignored in design. Also, the behavior and performance of such walls are important during an earthquake. Many studies have been carried out on infill walls and it was pointed that a properly designed infilled frame could significantly contribute to the structures stiffness, strength and energy dissipation. However, presence of infill walls may sometimes lead to undesired structural behavior. For this reason, different analytical models were investigated to estimate the behavior of infilled structures. Unfortunately, most of analytical models offered today have been validated with limited experimental data and they have often showed different performances when compared with recent test data. In this respect, it is obvious that limit analysis methods considering a variety of possible failure mechanisms seem to be the most promising technique. Furthermore, limit analysis methods need to be further refined and comprehensively validated in a systematic aspect before being used in engineering applications. In the scope of this paper, behavior and modelling of infilled structures were investigated based on the recent findings and developments. To conclude, possible failure mechanisms of such frames were summarized in Figure 1.1.

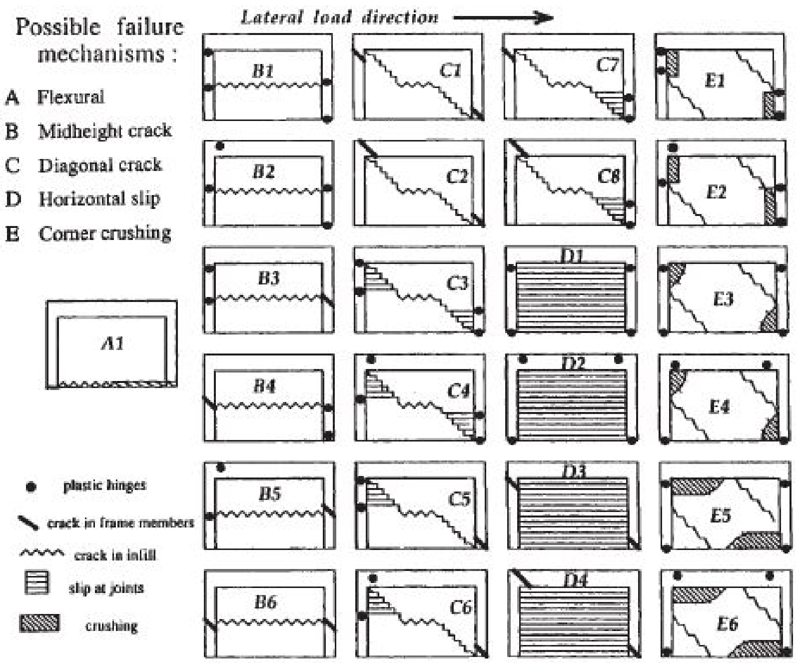

Figure 1.1. Failure mechanisms of infilled frames (Shing and Mehrabi 2002 [1]).

Cunha and Caetano (2006) [2] conducted a comparative and experimental research about modal analysis of different type of civil engineering structures. With the development of finite element methods and software packages, it has become easier to accurately estimate and simulate structural behavior. Also, input (loading) – output (vibrations) modal identification methods allow to describe the dynamic parameters of structures, experimentally. However, it is difficult to excite complex and large civil structures like dams and bridges to obtain vibration values. Therefore, it is necessary to develop different experimental methods to determine the dynamic properties of such civil structures. This paper shows the evolution of experimental modal analysis in the structures, from input – output to output – only modal identification methods. The accuracy and efficiency of output – only modal identification method were demonstrated according to the experimental study results carried out on different type of structures.

Michel *et al.* (2008) [3] investigated the determination of dynamic parameters of structures by utilizing ambient vibration test. The use of ambient vibrations for modal analysis of structures has become widespread when compared to the traditional methods like forced vibrations. The frequency domain decomposition (FDD) is the most widely used method in modal analysis due to its simplicity and accuracy. In this study, the physical meaning of the FDD method was described to estimate the modal characteristics. The determination of dynamic parameters of a building is important because it allows calibrating analytical model, detecting the modification of the building after damage and estimating the dynamic behavior of the building under earthquakes. Therefore, two different situations were examined on nine – storey RC existing building located in France. Firstly, ambient vibrations were recorded to obtain fundamental frequencies of the building. The second study was carried out during bridge demolition in order to check the accuracy of modal parameters obtained from the ambient vibrations. It is observed that the frequency values of the ambient vibrations and the bridge demolition experiments are very close to each other at very small error rates. As a result, FDD technique is reliable and easy to perform method to determine the modal characteristics of the existing buildings.

Dolšek and Fajfar (2008) [4] proposed a research about the effect of infill walls on the seismic behavior of reinforced concrete frame including four floor. In this study, to evaluate the seismic response of a subjected frame N2 method was used. Briefly, N2 method allows combining pushover analysis of a multi - degree – of – freedom (MDOF) system with the response spectrum analysis of an equivalent single – degree – of – freedom (SDOF) system. Seismic evaluation of a four – storey RC frame including masonry infill walls was carried out on two different models. In the first model, openings on the infill walls were considered (partial infill model); however, openings were ignored for the other model (full infill model). In addition, to compare the models bare frame model was created. The results of the study show that if the deformation capacity of the masonry infill walls satisfies the seismic demand, the masonry infill walls significantly increase not only stiffness but also strength of a structure (Figure 1.2). In this respect, mathematical models including the masonry infills represent more realistic behavior of the structure. The infill walls also have useful effects on structural response via preventing shear failures of columns and damage limitation if they are placed regularly. Finally, the possibility of the masonry infills can totally change the distribution of damage along the structure.

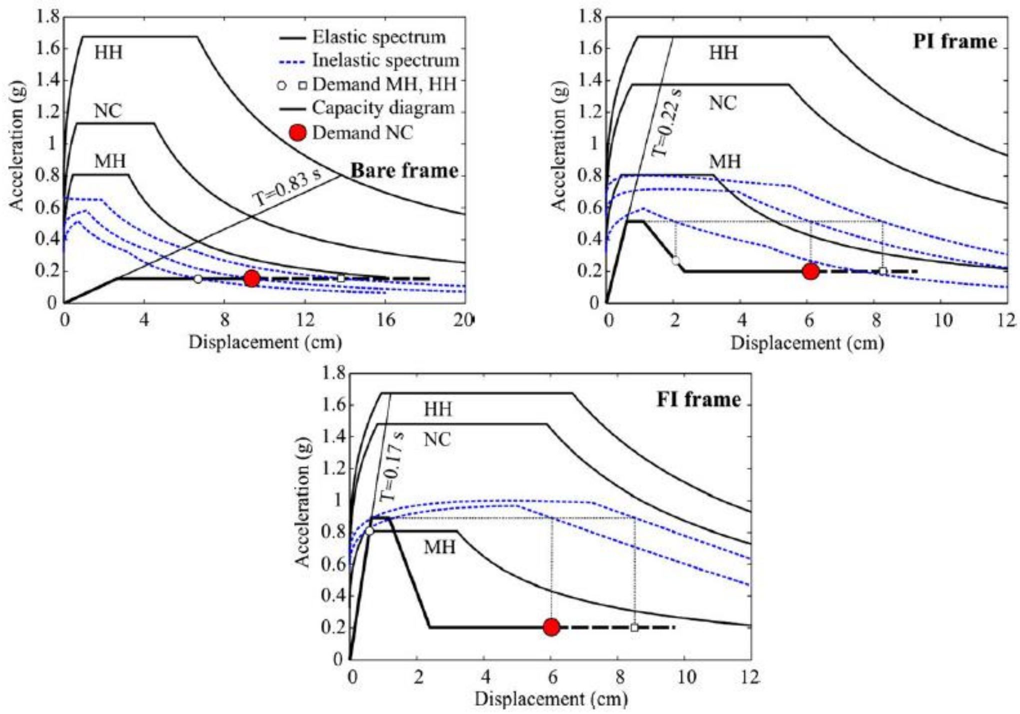

Figure 1.2. Demand spectra and capacity curves for idealized SDOF systems (Dolšek and Fajfar 2008 [4]).

Kaushik *et al.* (2008) [5] carried out comparative study considering six different mathematical models for infill walls (Figure 1.3). The main objective in this study was to determine the best analytical model representing the behavior of the infill walls. In this respect, bare frame model was created in Model - 1 and infill wall modeled as a shell element in Model - 2. In Model - 3, single equivalent diagonal strut was used for infill wall and infill wall was created using three diagonal struts in Model - 4. In Model – 5, partial infill wall was constructed using shell elements such that only half of length of column and beam was in contact with the infill wall. Similarly, partial infill wall modeled as shell elements in Model – 6, but only quarter of length of column and beam was in contact with the infill wall when compared to Model - 5. Linear and nonlinear static analyses were conducted to determine a suitable model for masonry infill walls in RC frames. As a result, when considered to predict global behavior of the system according to initial stiffness and ultimate failure load, the single strut model revealed results with acceptable accuracy. To estimate the force resultants in the frame members, 3 - strut model could be preferred because it gives more reliable data when compared to 1 - strut model. Full infill model (Model – 2) is first converted into Model – 5, and then into Model – 6 under increasing lateral loads. In model – 4, force resultants obtained in columns and beam are found to be similar with that obtained using partial shell models (Model – 5 and 6).

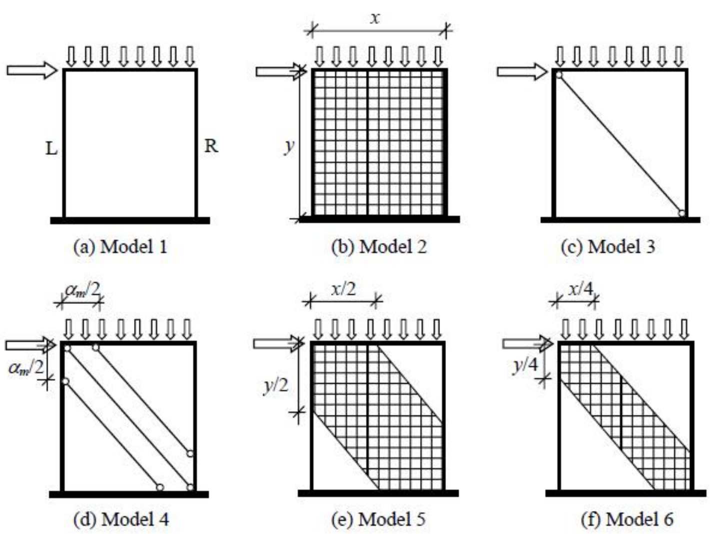

Figure 1.3. Studied analytical models (Kaushik *et al.* 2008 [5]).

Güler *et al.* (2008) [6] carried out a remarkable study about the evaluation of fundamental periods of RC buildings having a frame-type structural system and the effects of partition walls. Earthquake design codes give empirical formulas or approximations to estimate fundamental periods of buildings in determination of statically equivalent earthquake loads. In addition, some existing codes ignore the effects of infill walls on fundamental periods of the buildings. However, it is significant to use real periods for performance and design evaluation of existing buildings. In the scope of the study, experimental and analytical methods were used to investigate the relationship between height and fundamental period of the building. Also, a constant coefficient was described to magnify the empirical formula in order to reach periods specified in design codes. In 12-storey RC building (Building – 1), ambient vibration surveys were performed in three different construction phases including bare frame, frame with infill wall and frame with plastered infill wall. Fundamental periods in two main directions obtained from surveys ared in Table 1.1.

Table 1.1. Fundamental periods of Building – 1 (Güler *et al.* 2008 [6]).

Case	**Bare Frame**	**Frame with Infill Wall**	**Frame with Plastered Infill Wall**
T_x[**Weak**]	1.020 s	0.930 s	0.600 s
T_y[**Strong**]	0.680 s	0.640 s	0.510 s

Furthermore, the fundamental periods of the building were calculated numerically using 3D model with diagonal struts for partition walls. Numerical analyses carried out on Building - 1 were repeated for seven different cases by changing the height of the building. As a result, these analyses helped to find a relationship to estimate the fundamental periods of infilled RC frame-type structures for ambient vibrations. Then, experimental studies were conducted on five more buildings (Building – 2, 3, 4, 5 and 6) in order to evaluate the accuracy of the relationship obtained before. The experimentally obtained fundamental periods of these buildings were satisfied with estimated periods using the relationship. The average difference was approximately 13%. According to the results, the relationship could be used for the estimation of the fundamental periods of mid-rise RC frame-type buildings in Turkey.

Tsai and Huang (2009) [7] aimed to examine the effect of infill walls on the progressive collapse potential of a RC building. 3D finite element models were created and fourteen different cases due to sudden column loss were considered. In this paper, partition walls were taken into account as compression struts to investigate their effect on the progressive collapse potential of an EQ resistant RC structure. Also, linear static analyses were performed in order to observe the change in demand – capacity ratio (DCR) and axial force change in the beams connected to the removed column. The study showed that the presence of brick infill walls led to reduction in DCR and this reduction directly depends on the location of infill walls. It is pointed out that the shorter the span, the better the infill wall contribution. However, the presences of infill walls on both sides of the removed column may cause increase the axial tension of the two – span beam bridging the removed column.

Kose (2009) [8] aimed to determine the parameters affecting the fundamental period of reinforced concrete buildings with partition walls. In the scope of study, 189 building models were analyzed by taking into account the changes in selected parameters such as height of building, number of bays, ratio of shear wall area and ratio of infill wall. It was observed that building height was the main parameter affecting the fundamental period of the building. The ratio of shear wall area was also found to be the second significant factor on fundamental periods. Furthermore, it was pointed out that ratio of infill walls and number of bays showed similar effect on period of the structures. The presence of shear walls and infill walls significantly increases the rigidity of the buildings; also they contribute reducing fundamental periods. For this reason, to ignore the effects of shear walls and infill walls, when calculating fundamental periods of building will cause error. In this paper a new formula was offered to estimate the fundamental period of structures based on results obtained from analyses. This formula is a function of investigated parameters and it gives better results when compared with the equations suggested in codes.

Pujol and Fick (2010) [9] performed full scale experimental study on a three – storey RC building including masonry infill walls. This study aimed to investigate the effects of partition walls on vulnerability degree of the RC structures under strong ground motion. In this respect, two different full scale model with and without infill walls were compared under laboratory conditions. In the scope of this study, cyclic lateral loads were applied at each floor of the structures and only in – plane loadings were considered. Firstly, bare frame structure was tested and punching shear failure at column – slab connection was observed in the structure. Then, brick infill walls were built at the structure to investigate the effects of infill walls and same test procedures were repeated. The ratio of cross – sectional area of infill walls to the total floor area was calculated as 0.27%. It was investigated that the infill walls significantly increased the lateral stiffness (by approximately 500%) and base shear strength (by approximately 100%) of the structure. It is obvious that the structure with infill wall have shorter fundamental periods than the bare frame structure. Furthermore, the infill walls under the slab where punching shear failure was observed in the first test had severe damage at the end of the second test.

Oliveira and Navarro (2010) [10] conducted a research about obtaining fundamental periods of RC building from in – situ experimental and numerical techniques. Different structures built in Portugal were investigated in the scope of the study. It is clear that damping ratios and vibration periods are important characteristics to determine the dynamic behavior of structures. To measure these characteristics different methods like induced vibration, record of earthquakes or explosions and ambient vibration or microtremors could be used. This paper indicates that in-situ ambient vibration recordings can be used as a simple method with a single sensor at the top of the structure to determine the natural periods and damping of the building for low amplitude input motions. The study also introduces changes in natural periods and damping for larger input motion. Another point of interest is to determine the uncertainties in the evaluations and their impacts on the results of dynamic analysis. The dynamic properties of three prototype RC buildings of different typologies were evaluated by creating structural analytical models. These examples show the importance of the experimental methods versus the analytical modelling due to the remarkable frequency value differences between the engineering design practice and the in-situ measured values. More examples are needed for a more powerful calibration. Furthermore, two models were investigated for two different approaches, one assuming that the partition walls were not participating in the stiffness of the structure, and the second, corresponding to the real structure placing the partition walls at their locations. It is observed that the results are so close when comparing the experimental values with the second model with infill walls. Also, different types of RC buildings with and without infill brick walls were analyzed in the scope of this study. According to the results, frequency values may increase 70–100% in relation to building without partition walls in MRF buildings. In addition, numerical model could reach results very close to the measured ones if infill walls are modelled. The simple method used in this study is the best approach to determine the dynamic properties of RC buildings rapidly. Thus, it provides big advantages in terms of effectiveness, accuracy and cost.

Uva *et al.* (2012) [11] conducted a case study on existing RC framed building having a good level of information with including extensive experimental data. First a reference frame was selected from the existing RC framed building, and then nonlinear static analyses were performed to investigate some significant parameters about the modelling of the infill walls. Particularly, sensitivity analysis was carried out to determine specific parameters as strut width (b_w), Force – Displacement relationship of the panel and strut number (single or multiple) because such parameters play a key role on definition of the equivalent strut models. The analyses revealed that, even if frame – panel system having same mechanical properties and geometry, when infill panels created with high values of b_w, system reaches greater strength peak but it represents brittle behavior. However, the lower values of strut width support a ductile behavior. According to this study, the type of failure mechanism for the panel significantly depends on the constitutive Force – Displacement relationship of the panel. A brittle shear mechanism was observed at the nodes of the frames when using the multiple struts due to the interaction between RC columns and the infill walls. Also, nonlinear static analysis of frame was performed by using double equivalent strut model. To compare the findings with the analyses observed with a single equivalent strut model, the effect of the local frame – infill wall interaction at the nodes results in significant reduction of the strength due to the formation of story mechanism.

Adukadukam and Sengupta (2013) [12] conducted a study about equivalent strut method for the modeling of masonry infill walls. In the first part of the study, availability of elastic and ultimate limit approaches in the literature for concerning the properties of the diagonal struts was handled. Elastic analysis approach gives adequate results for researches and also such approach is more understandable in calculations. To estimate base shear of the system with elastic analysis demonstrates conservative results because this method gives larger strut width. However, using ultimate limit approach gives lower strut width, so it allows reducing the stiffness of the strut. This situation provides more rational results to estimate of the inelastic deflection. When considered pushover analysis only strength and strut width knowledge is insufficient; in addition to these data, axial load – deformation relationship of strut is also essential. At the beginning of the second part of the study, axial load – deformation relationship was constructed and then usage of the hinge property was indicated on two different case studies. Regular shaped building (Building I) designed for EQ loads was performed in the first case and irregular shaped building (Building II) not designed for resisting EQ loads was conducted in the second case. Within the scope of this paper, single equivalent strut model was used for representing the infill walls and pushover analysis was performed. Although nonlinear axial load – deformation relationship exist, idealized linear relationship was used for the hinge property of the strut due to the difficulties on application of non-linearity. If linear elastic relationship is used for strut property, hinge formation could only be occurred in the struts, so it may cause the termination of the pushover analysis. As a result of this, nonlinear behavior of the building will not be seen clearly at the end of the analysis. When a nonlinear relationship is considered even if idealized linear relationship is used, formation of hinges occurs first in the frame then in the struts. This reveals more realistic and meaningful results with satisfying the aim of the pushover analysis. Pushover curves obtained from analyses are illustrated in Figure 1.4.

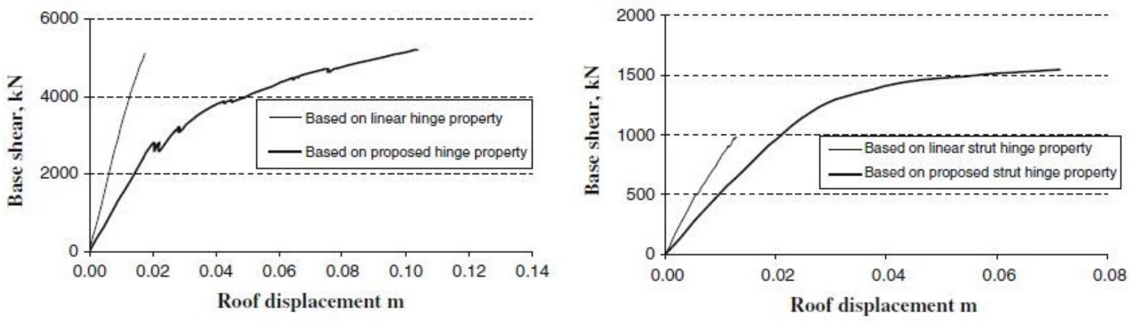

Figure 1.4. Pushover curves for Building I and II (Adukadukam and Sengupta 2013 [12]).

Haldar *et al.* (2013) [13] performed a research about seismic failure modes of masonry infill walls and simulation of seismic behavior of frames including such walls. Detailed information about the seismic behavior of infilled frames is still limited and also their modelling techniques are not properly defined in the design codes. In this respect, past earthquake damage reports, experimental surveys, design codes and analytical model methods were taken into account in this paper for determining failure modes of masonry infilled frame buildings. Earthquake damage surveys have demonstrated that interaction between frames and infill walls affect the behavior of infilled frame systems considerably, when compared to bare frame systems. If such interaction is ignored designing the structures, some undesirable results may occur not only at member level (short column effect, flexural and shear failure of columns) but also at the whole structure (soft story, torsional effects). Hence, these kinds of failure mechanisms generally lead to inadequate structural performance. According to detailed assessments, four significant failure modes of the masonry infill walls were identified as bed – joint sliding shear failure, diagonal compression failure, diagonal tension cracking and corner crushing of infill walls. Also, many models were generated to evaluate the strength of the equivalent diagonal strut in these failure types. Equivalent strut method is widely used to simulate the infill walls and single, double or multiple struts are well known modelling types for this target. Observations showed that construction methodology of infill walls have a huge impact on the failure mechanism of structures because ideal contact between the frame and infill wall is assumed in analytical models but it could not be possible in reality. Post-earthquake damage studies proved that the availability of the gaps at the intersection of the columns and infill walls cause the shear failure for the columns. As a result, the comparison of analytical studies with the experimental surveys and field investigations revealed that the realistic way of predicting the failure mode of infilled frame systems is simulating infill walls using single diagonal struts connected to columns eccentrically. The availability of infill walls significantly reduces the shear failure at column and beam intersections because of the reduction of the shear forces. Also, using the infill walls changes the combination of axial force – moment interactions in the infilled frame columns.

Devin *et al.* (2015) [14] investigated the effects of non – structural elements such as partition walls and external claddings on floor modal parameters. Although they could have important stiffening effects on the vertical vibration behavior of floor in structures, it is hard to calculate and take into account these effects when designing. Two full – scale floors in a structurally identical, with same modal characteristics and located in real building were investigated to quantify such effects. The measurements were made on the same floors before and after the installation of interior partition walls and cladding panels. These experimental surveys were carried out on four – storey RC office building. It was observed that the fundamental frequencies of the subjected floor increased by 30% after the installation of the partition walls and cladding panels. According to the results, partition walls not only add mass and damping to the structure, but also contribute to stiffness. Therefore, there is the potential for the partition walls to be used as an effective and predictive means of passive control of floor vibrations.

1.3. Scope

The scope of this thesis was planned as follows:

In Chapter 1, basic information about the effects of the partition walls into the dynamic behavior of reinforced concrete buildings, the motivation and the importance of preferred topic were described. This chapter also included literature review, the organization and content of the thesis.

In Chapter 2, general information regarding the studied building, on-site building survey details and material investigation study of the building were summarized.

In Chapter 3, application of ambient vibration test (AVT) on real world structure and ambient vibration test details for the determining the natural frequencies of an existing building were presented.

In Chapter 4, experimental study details completed in Boğaziçi University Structures Laboratory was shown and obtaining the mechanical properties of the partition walls was investigated. Implementations of the EN 1052 – 1 and EN 1052 – 3 specifications were explained. Also, detailed information about the compression and shear tests was given in this chapter.

In Chapter 5, four types of 3D finite element models were created to determine modeling approach which represents the real dynamic behavior of reinforced concrete building the best. Bare frame model was created to understand more clearly how the partition walls affect the dynamic behavior of the structures. In the second model, partition walls were modeled as the shell elements. The partition walls were considered as one – way equivalent single strut elements in the third finite element model. In the last analytical model, the partition walls were considered as two – way equivalent single strut elements. Chapter 5 is also composed of detailed analysis results of the four types of 3D models.

In Chapter 6, the results obtained from laboratory tests, numerical analyses and site surveys were summarized. Also, recommendations for the future works were discussed in this chapter.

2. BUILDING INFORMATION

The studied building was built in 1980s and used as a residential building for the staff of General Directorate of Highways. The building is located in Göztepe, Turkey. The building represents the twin flat apartment buildings that frequently constructed in Turkey. Also, Figure 2.1 and Figure 2.2 show the front and rear view of the studied building.

Figure 2.1. Front view of the building.

It consists of a basement floor, a ground floor and four typical floors. The story height of the ground and typical floors is 2.90 m, and the basement story has a height of 3.9 m. The ground and typical floors have the same architectural plan and symmetrically located two residential units at each floor. However, the basement floor contains different non-symmetrical parts like a small residential unit and depot to store heating and other equipment.

Basically, the structural system of the studied building is composed of column – beam frames and one-way and two-way slabs with 10 cm thick. In addition, the front side of the building includes basement wall.

Figure 2.2. Rear view of the building.

There are four types of columns having a rectangular cross-section in the building with dimensions of 25 cm × 45 cm, 25 cm × 50 cm, 30 cm × 50 cm and 25 cm × 100 cm. Also, interior beam dimensions are 20 cm × 50 cm and exterior beam dimensions are 25 cm × 50 cm. The thicknesses of the all interior and exterior partition walls were determined as 20 cm. Partition wall configurations of basement floor and typical floors are demonstrated in Figure 2.3 and Figure 2.4, respectively. To compare with the existing projects of the building, on-site building survey was conducted.

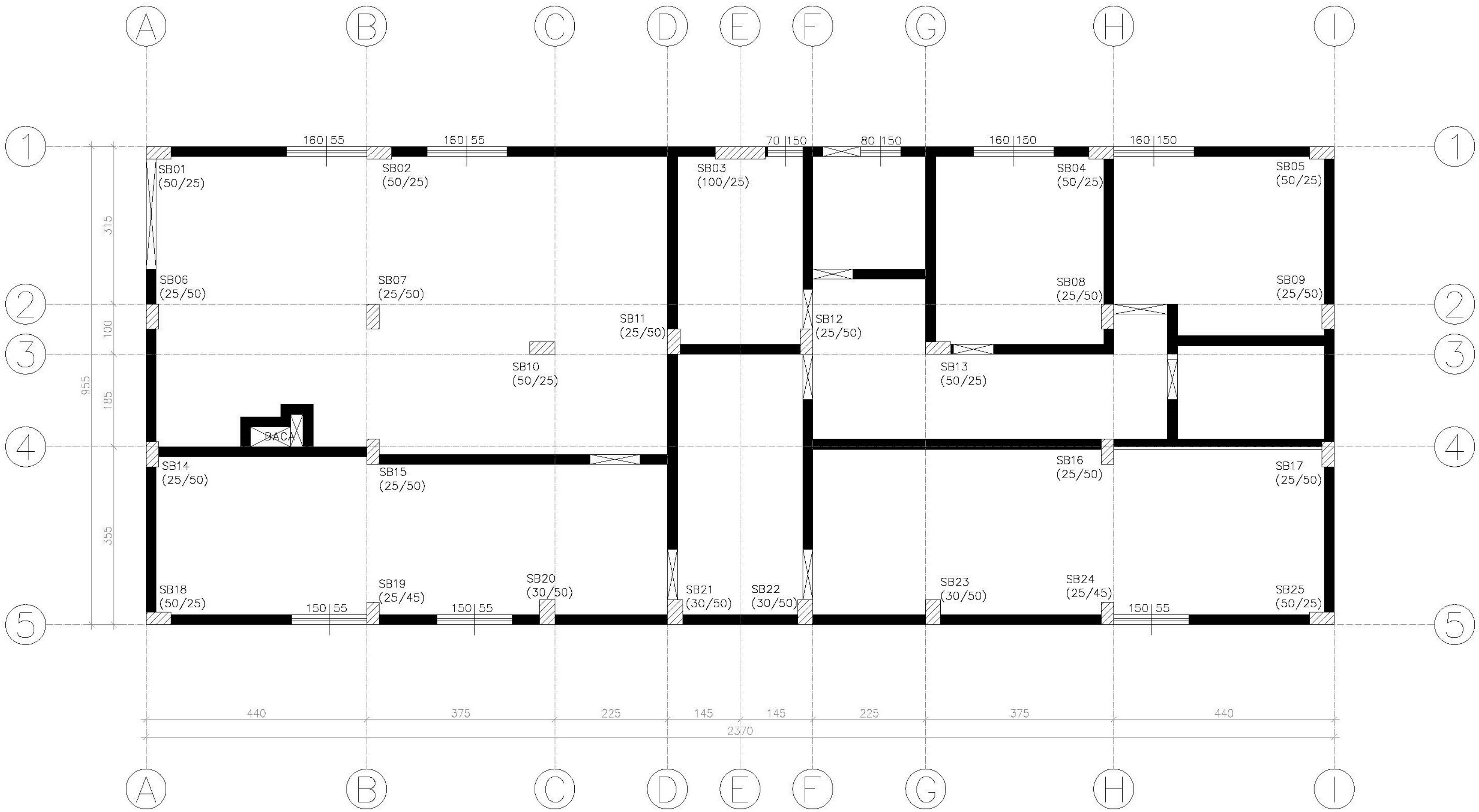

Figure 2.3. Partition wall configuration of basement floor.

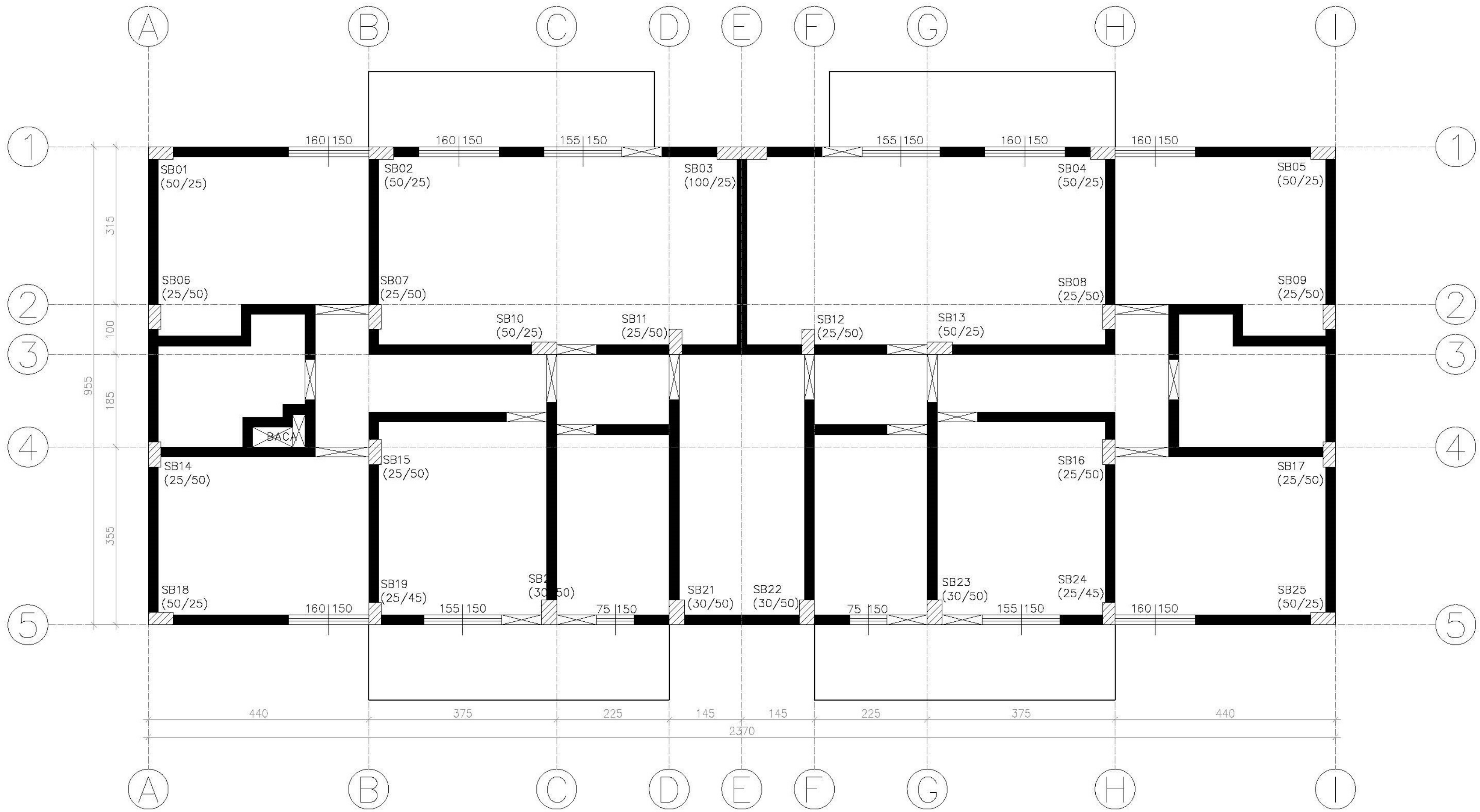

Figure 2.4. Partition wall configuration of typical floors.

A detailed material investigation of the building was performed for concrete and reinforcement details according to Turkish Earthquake Code. As a result of the material investigation, the average compressive strength of concrete was found to be 13.40 MPa and the steel grade was determined as S220. Moreover, site investigations revealed that $\phi16$ reinforcements for longitudinal bars and $\phi8$ reinforcements for stirrups were used in columns and beams. Distances between stirrups were measured as 25 – 30 cm. In short, very poor seismic detailing with low-grade reinforcement and low-strength concrete was observed in the building. Conducted site investigations are illustrated in Figure 2.5.

Figure 2.5. Site investigations in the building.

3. VIBRATION TESTS

It is obvious that evaluation studies of existing buildings include many uncertainties. Generally, non-destructive methods are preferred to estimate real dynamic behavior of the structures. In this respect, ambient vibration test (AVT) is practical and widely used non-destructive method to assess the existing buildings.

Ambient vibration test is used in buildings and other civil engineering structures in order to determine their fundamental periods. Such test is also used to determine the dynamic behavior of structures (frequencies and modal shapes). Another benefit of ambient vibration test is that it allows the calibration of numerical models (model updating). Ambient vibrations are produced by natural sources such as local atmospheric conditions (e.g. the wind) or by human activities (e.g. traffic). In that respect, this method was frequently used in the researches before [15-26].

In this part, natural frequencies of the building for each direction (X, Y and Z) obtained by AVT is presented. For this purpose, Kinemetrics TSA-SMA triaxial accelerometers shown in Figure 3.1 were used, and also vibration surveys were repeated different times to obtain the most reliable results.

Figure 3.1. Kinemetrics TSA-SMA triaxial accelerometer used in AVT.

In vibration analysis, the corner points were considered at each story level in order to reveal the torsional mode of the building as well. Sensor orientations and building position are indicated in Figure 3.2. At each story level, 20-min-long recordings are found to be enough for having reliable vibration data.

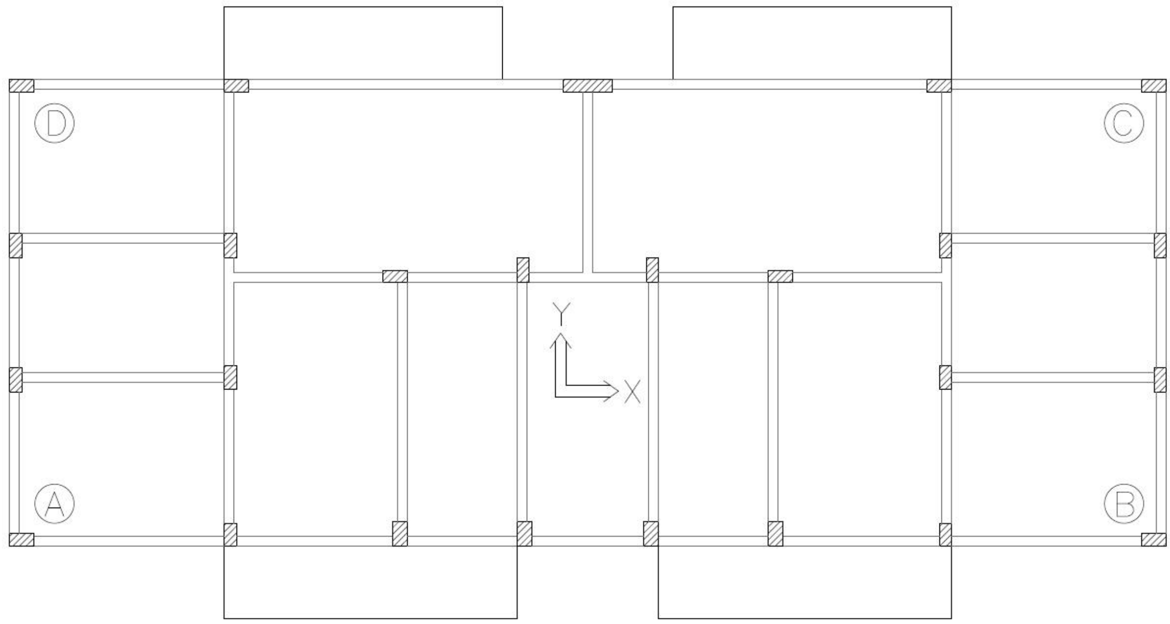

Figure 3.2. The locations of sensors used in AVT.

When the AVT is completed, the building-specific acceleration data is obtained using sensors. However, such data obtained from AVT is inadequate to interpret dynamic properties of the building due to the data format. For this reason, the dynamic properties of the building were extracted from AVT by making use of Fast Fourier Transform (FFT). The FFT is commonly used process to convert a signal from its original domain (time or space) to frequency domain and vice versa. After that process, Power Spectral Density (PSD) - modal frequency charts shown in Figure 3.3 and Figure 3.4 are created to determine the dynamic properties of the building.

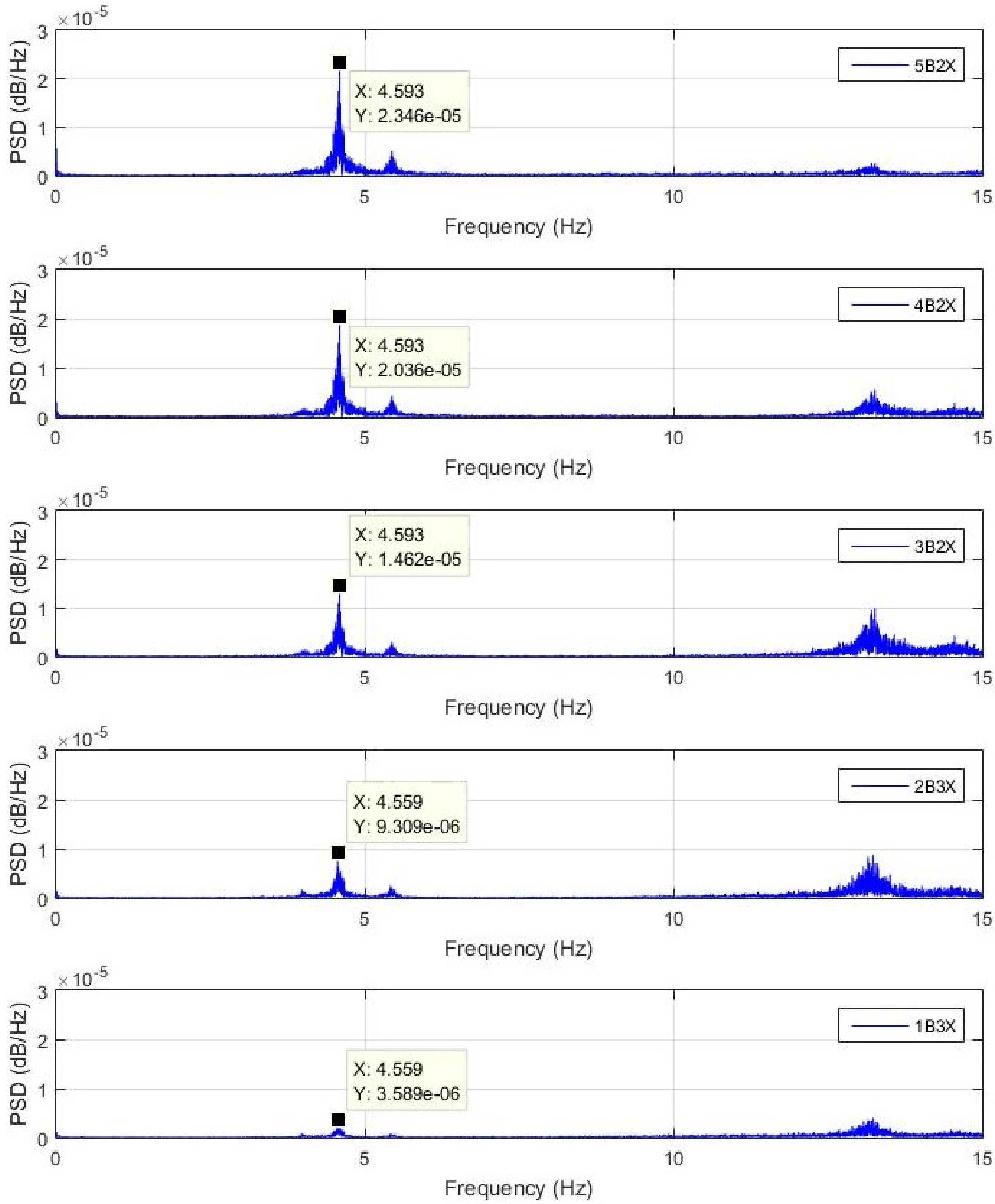

Figure 3.3. Frequency values in the X direction of corner B at each story level.

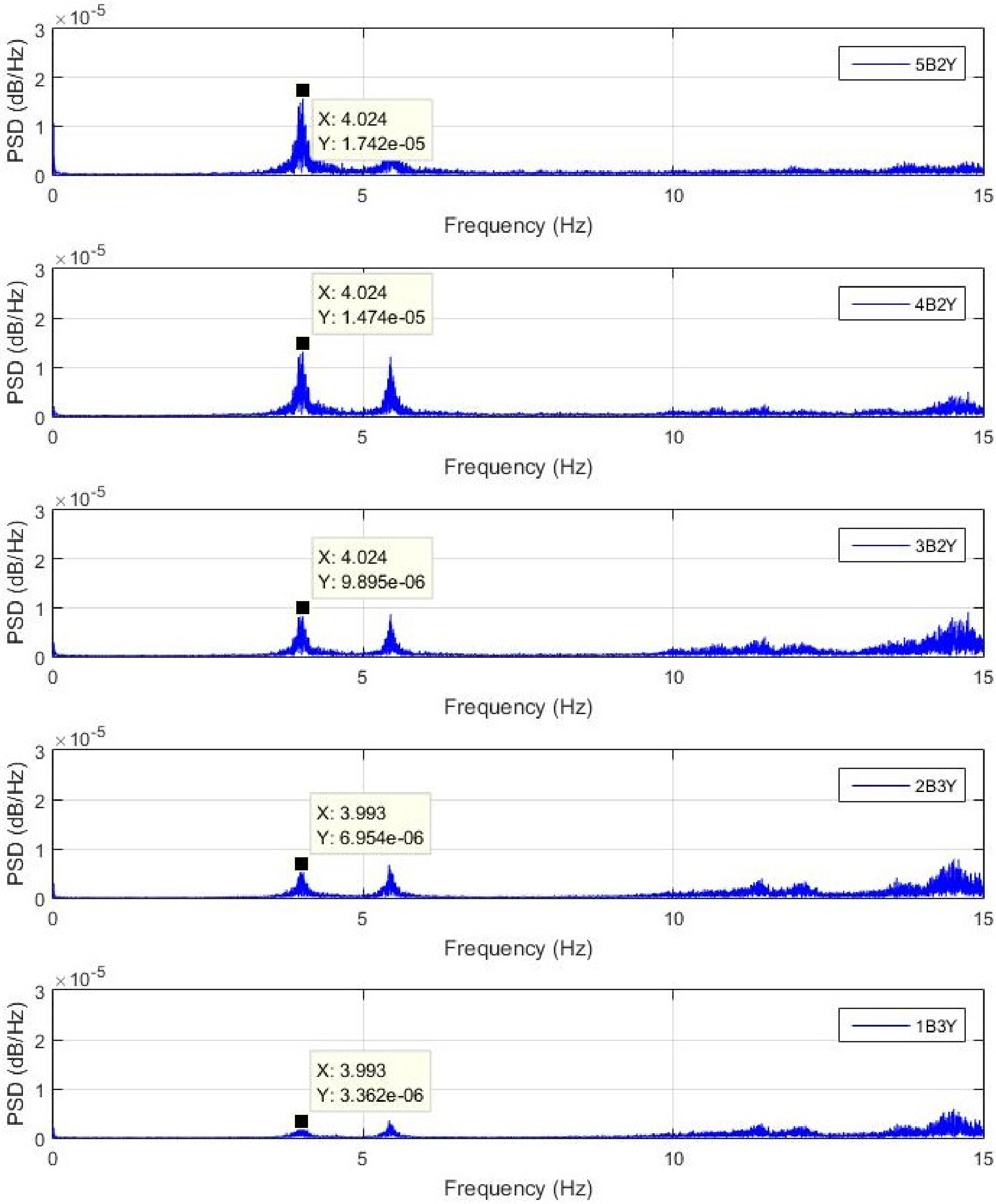

Figure 3.4. Frequency values in the Y direction of corner B at each story level.

As a result of Ambient Vibration Test, obtained modal frequencies of studied building are given below for each direction:

- $f_1 = 4.60$ Hz (X directional modal frequency)
- $f_2 = 4.02$ Hz (Y directional modal frequency)
- $f_3 = 5.44$ Hz (Torsional modal frequency)

Also, Peak Picking (PP) method, the simplest way to identify the frequencies, was used in the study. It comprises in taking the frequency peaks of average spectra for each sensor placed at different points. The PP method is the basis of frequency domain decomposition (FDD). In order to determine the frequency values of X and Y directions, the dominant peak values in frequency domain were taken into account. Then, the mutual peak points of each direction were considered as torsional frequency.

Moreover, the mode shapes of the building were drawn as illustrated in Figure 3.5. Normalized modal displacements were calculated by scaling the square root of the power spectral density (PSD) values obtained from sensor placed at corner B of the building.

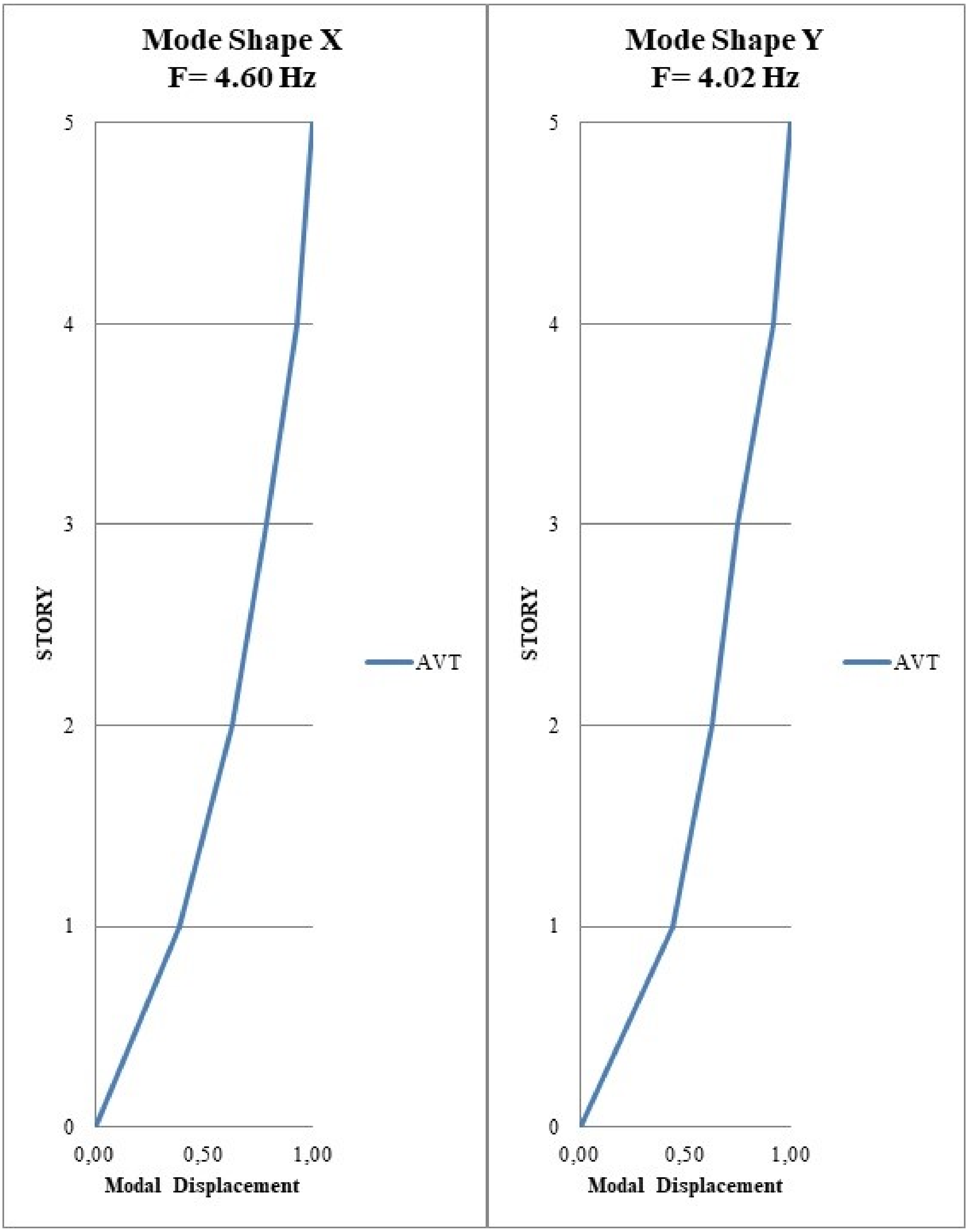

Figure 3.5. Normalized modal displacements obtained from AVT.

4. LABORATORY TESTS

The ultimate objective of the experimental study is to obtain the mechanical properties of the infill walls. Basically, shear and compression tests are conducted to determine the shear strength, compressive strength and modulus of elasticity for this purpose. Laboratory tests play an important role within the scope of this thesis because obtained mechanical properties (real mechanical properties) will be used in analytical models rather than assumptions. In this respect, EN 1052 – 1 and EN 1052 – 3 specifications are considered as a reference. The former is used to determine the compressive strength of infill walls and the latter is used to gain the initial shear strength of such walls. Also, considered specifications inform about preparation of test specimens and curing but tested specimens are removed from a real structure. Because of this reason, sections including this type of information are ignored.

Before extracting test specimens from the building, the size of the brick units and the exterior and interior walls to be used in laboratory tests were determined. Hollow brick with dimensions of 130 × 180 × 180 mm [height, length, and thickness, respectively] had been used with almost 10-mm mortar and plaster thicknesses in both exterior and interior walls. Specimen extraction process is illustrated in Figure 4.1, Figure 4.2 and Figure 4.3.

Figure 4.1. Extraction of compression and shear test specimens by sawing.

Figure 4.2. Compression and shear test specimens extracted from the building.

Figure 4.3. Gap in the partition wall due to extraction of the test specimen.

4.1. Compression Test

According to EN 1052 – 1, minimum three specimens are required for compression test and specimen dimensions are given in Table 4.1. For compression test, six specimens have been extracted from the studied building by sawing. Also, extracted specimens were five bricks in height and three bricks in length. As shown in the Table 4.1, test specimen size directly depends on the single brick used in partition wall. The subscript "*u*" symbolizes the unit brick, subscript "*s*" symbolizes specimen, "*h*" is height, "*l*" is length and "*t*" is thickness. Compression test is illustrated in Figure 4.4.

Table 4.1. Specimen sizes for testing the compressive strength of masonry (EN 1052–1).

Face size of unit		Masonry specimen size			
l_u (mm)	h_u (mm)	Length l_s	Height h_s		Thickness t_s
≤ 300	≤ 150	$\geq (2 \times l_u)$	$\geq 5h_u$	$\geq 3\ t_s$ and $\leq 15t_s$ and $\geq l_s$	$\geq t_u$
	> 150		$\geq 3h_u$		
> 300	≤ 150	$\geq (1{,}5 \times l_u)$	$\geq 5h_u$		
	> 150		$\geq 3h_u$		

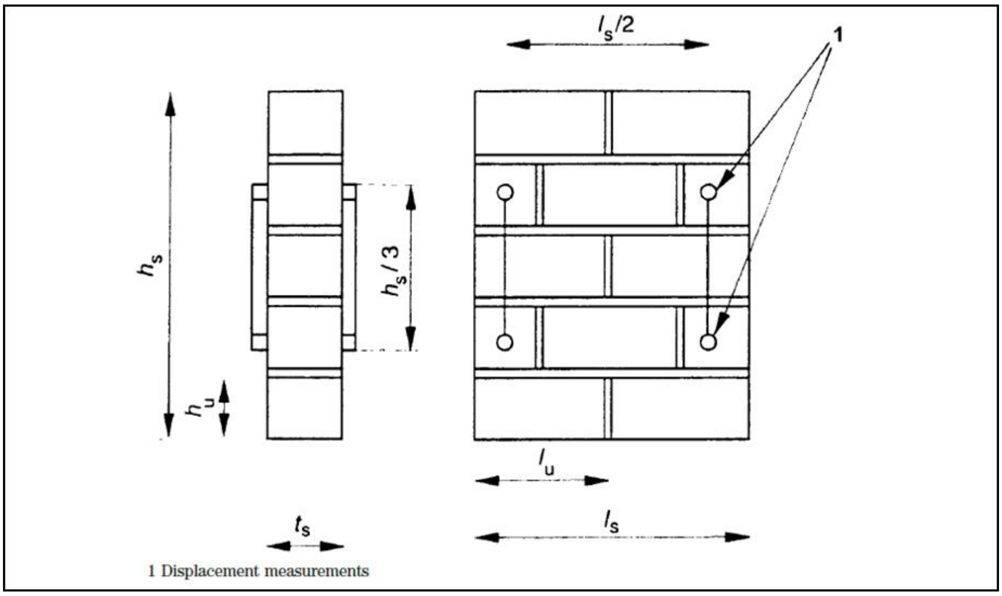

Figure 4.4. Masonry specimen (EN 1052 – 1).

After each compression test, the compressive strength of an individual masonry specimen (N/mm^2) is calculated, as shown below.

$$f_i = \frac{F_{i,max}}{A_i} \tag{4.1}$$

where $F_{i,max}$ is the maximum load reached on an individual masonry specimen (N), A_i is the loaded cross – section of an individual masonry specimen (mm^2). Using the smaller value of two equations given in 4.2 and 4.3, to the nearest 0.1 N/mm^2, the characteristic compressive strength of masonry can be calculated.

$$f_k = \frac{f}{1.2} \tag{4.2}$$

$$f_k = f_{i,min} \tag{4.3}$$

where f is the mean compressive strength of the masonry (N/mm^2) and $f_{i,min}$ is the smallest compressive strength of an individual masonry specimen (N/mm^2). To determine modulus of elasticity E_i, the masonry specimens shall be fitted with measuring devices as shown in Figure 4.4 in order to measure the change in height. Four measuring points located on each side of the specimen will be enough for calculating strain value. Modulus of elasticity is a secant modulus from the mean of the strains of all four measuring positions occurring at a stress equal to one third of the maximum stress achieved. Briefly, modulus of elasticity (N/mm^2) can be obtained using equation given in 4.4.

$$E_i = \frac{F_{i,max}}{3\varepsilon_i A_i} \tag{4.4}$$

where ε_i is the mean strain in an individual masonry specimen.

Within the scope of compression test, six specimens were tested and it was observed that all specimens demonstrated brittle failure. Figure 4.5 and Figure 4.6 indicate the testing mechanism and brittle failure of the test specimen.

Figure 4.5. Compression test mechanism.

Figure 4.6. Brittle failure of the partition wall specimen.

According to obtained test results, only the deformation data in the tests of four specimens could be read properly. The deformation values of other two specimens did not give reliable results and these specimens were not considered. Based on the findings, stress – strain relationships were created as shown in Figure 4.7 to calculate the modulus of elasticity. Also, Table 4.2 demonstrates the numerical data obtained from compression tests.

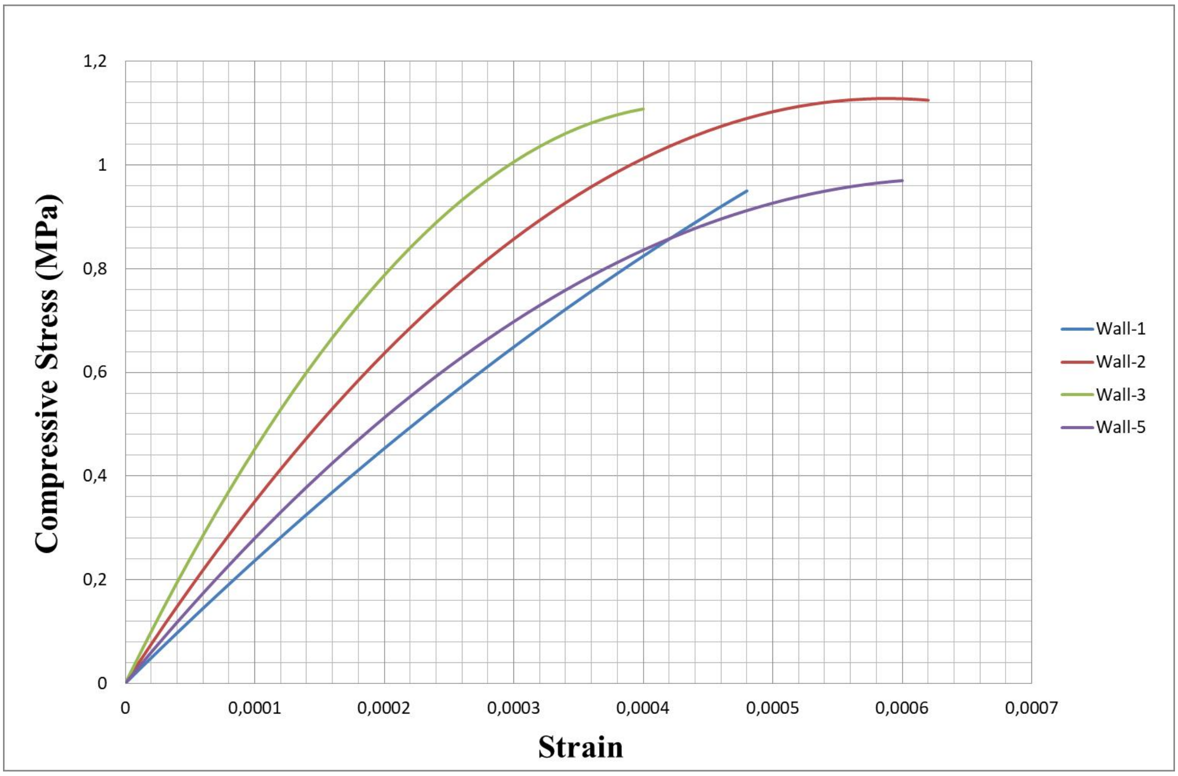

Figure 4.7. Stress – strain relationships of partition wall specimens.

Table 4.2. Mechanical properties of wall specimens.

Wall Specimen Number	**Wall Type**	**Compressive Strength (MPa)**	**Ultimate Strain**	**Modulus of Elasticity (MPa)**
Wall - 1	Exterior Wall	1.00	0.00051	2300
Wall - 2	Exterior Wall	1.10	0.00065	3600
Wall - 3	Exterior Wall	1.20	0.00042	4600
Wall - 4	Interior Wall	1.00	–	–
Wall - 5	Interior Wall	1.00	0.00054	3000
Wall - 6	Interior Wall	0.90	–	–

At the end of the compression tests, close results were obtained in terms of the compressive strength in six specimens. The minimum and mean compressive strength values were 0.9 MPa and 1.0 MPa, respectively. Hence, the characteristic compressive strength was calculated as 0.8 MPa using equation 4.2, rounding the value to one decimal. The modulus of elasticity of partition walls was assumed as the average of the four tests and calculated as 3400 MPa. Calculated numerical value was rounded to the nearest 100 MPa, as specified in EN 1052 – 1.

When calculated modulus of elasticity values compared each other, remarkable differences could be observed. For instance, the modulus of elasticity of Wall – 3 is twice that of Wall – 1. It may be beneficial to increase the number of specimens. However, unexpected results may not be resolved in this way because the infill wall specimens were extracted from a real building. As a result, structural uncertainties due to the age of the building will always lead to unexpected results.

4.2. Shear Test

EN 1052 – 3 gives detailed information about the determination of initial shear strength for masonry. The initial shear strength of masonry will be derived from the strength of small masonry specimens tested to destruction. Eight specimens have been extracted from the building by sawing for shear test. Also, extracted specimens were one brick in height and five bricks in length. Shear test specimen dimensions with types are given in Table 4.3.

Table 4.3. Dimensions and type of shear test specimens (EN 1052 – 3).

Unit length	**Specimen type and dimensions**	
l_u mm	Type according to Figure 1	Dimensions mm
≤ 300	I	$l_s = l_u$
> 300	I	$300 < l_s < 350$
≤ 300	II	$h_1 = 200$ $l_s = l_u$
> 300	II	$h_1 = 200$ $300 < l_s < 350$

Only if the height of the unit brick (h_u) > 200 mm, Type – II specimens may be used and Type – I specimens are valid for the others. Dimensions of shear test specimen including Type – I and Type – II are shown in Figure 4.8.

Also, two types of specimen geometry and shear test mechanism are illustrated in Figure 4.9.

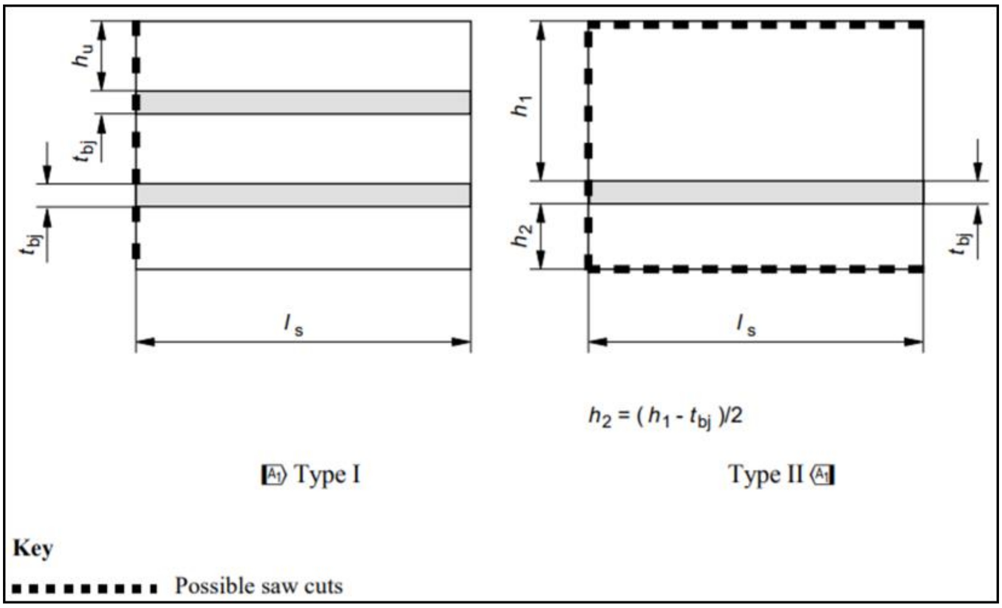

Figure 4.8. Dimensions of shear test specimen (EN 1052 – 3).

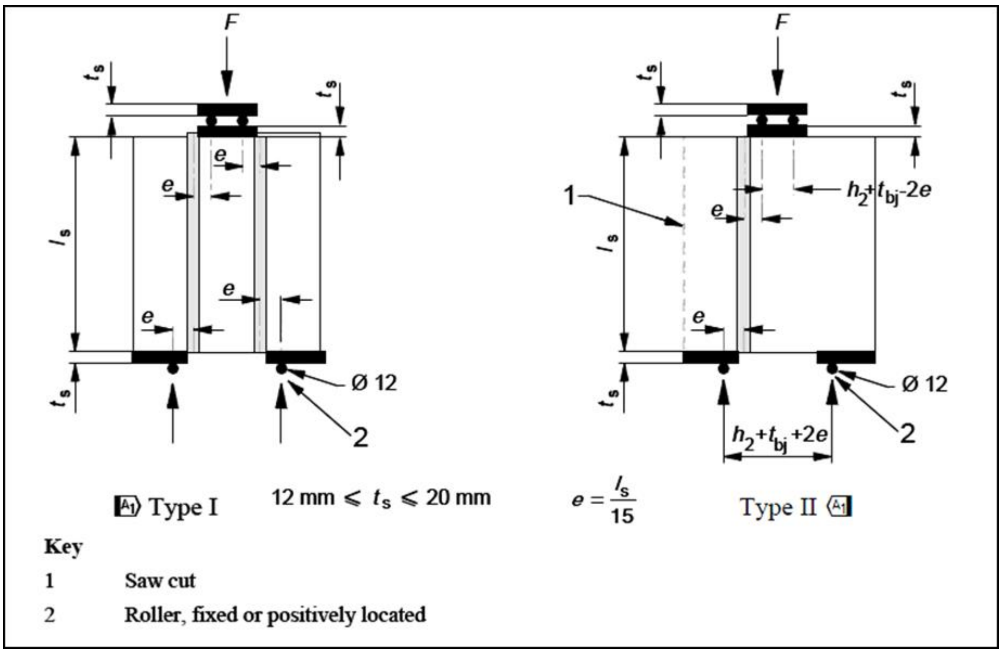

Figure 4.9. Loading of shear specimen (EN 1052 – 3).

According to EN 1052 – 3, two different procedures of loading can be used for shear test. In Procedure – A, at least three specimens under precompression load (horizontal stress) are required for testing. In Procedure – B, at least six specimens at zero precompression shall be used. To determine the initial shear strength of each specimen, the rules of the Procedure – B were considered in shear test.

For each shear test specimen, the shear strength of an individual sample (N/mm^2), to the nearest 0.01 N/mm^2, is calculated, as shown below.

$$f_{voi} = \frac{F_{i,max}}{2A_i} \tag{4.5}$$

where $F_{i,max}$ is the maximum shear force (N) and A_i is the cross - sectional area of a specimen parallel to the bed joints (mm^2). The characteristic shear strength, f_{vok}, can be calculated using equation given in 4.6.

$$f_{vok} = 0.8 f_{vo} \tag{4.6}$$

where f_{vo} is the mean initial shear strength (N/mm^2), or the characteristic shear strength shall be considered as the lowest individual shear strength result whichever is the lower and shall be given to the nearest 0.01 N/mm^2. In short, to calculate the characteristic shear strength, the smaller value of two equations given in 4.6 and 4.7 is used.

$$f_{vok} = f_{voi,min} \tag{4.7}$$

Within the scope of shear test, eight specimens were tested to determine the initial shear strength. The loading was applied to steel plate above the middle brick and supports were placed under two adjacent bricks, so that the required shear test apparatus was created in accordance with EN 1052 – 3 to slide the middle unit. Figure 4.10 and Figure 4.11 illustrate the shear test apparatus and shear failure of the test specimen.

Figure 4.10. Shear test apparatus.

Figure 4.11. Shear failure of the test specimen (EN 1052 – 3).

As a result of shear tests, half of the specimens failed due to brick cracks. This kind of undesired failure is a common situation in the shear tests. However, expected shear failure modes as specified in EN 1052 – 3 appendix part were observed at the other half of the specimens. Based on the findings, the characteristic shear strength was calculated as 0.21 MPa using equation 4.7. When calculating the characteristic shear strength, specimens having undesired failure like brick cracks were not considered. Also, Table 4.4 illustrates the numerical data obtained from the shear tests.

Table 4.4. Result table of the shear tests.

Wall Specimen Number	**Shear Area** (cm^2)	**Failure Load (kN)**	**Failure Type**	**Shear Stress (MPa)**
Wall – 1	559	2.70	Brick	–
Wall – 2	611	3.52	Brick	–
Wall – 3	598	1.75	Brick	–
Wall – 4	563	12.00	Shear	0.21
Wall – 5	577	13.10	Shear	0.23
Wall – 6	662	35.00	Shear	0.53
Wall – 7	597	8.00	Brick	–
Wall – 8	572	20.00	Shear	0.35

5. MATHEMATICAL MODELS

Four different mathematical models are created using SAP2000 as bare frame, shell, single strut – 1 and single strut – 2 models. In the bare frame model, the rigidity of infill walls is ignored as it is done in state-of-the practice. The shell model is constructed by idealizing the infill walls by shell element. In addition, two strut models are constructed separately as one directional single diagonal strut and two directional single diagonal struts for each infill walls. The ultimate objective of the numerical analyses is to determine the effects of partition walls on dynamic behaviors of RC buildings. Also, these analyses will help to find the best model representing the actual behavior of the studied building when compared to the AVT results.

Since no soil investigation was conducted within the scope of this study, soil flexibility is not known. As a result, soil – structure interaction is not taken into consideration.

5.1. Bare Frame Model

This mathematical model is created to control the effects of a shell and single strut models on the dynamic behavior of the reinforced concrete structures. Bare frame model includes only structural elements like beams, columns, slabs and shear walls (Figure 5.1). In this respect, just partition walls are ignored in finite element model. Thus, the model will help us understand more clearly how the non-structural walls, which are not considered in the design, actually affect the dynamic behavior of the structures. As a common practice when designing a new building, only the weights of the partition walls are taken into account, and these calculated weights are loaded as a distributed load on the beam located under the partition wall. Although this approach seems to be accurate in terms of static calculations, it causes to create a analytic model which does not fully represent the real behavior of the structure. Because, as the partition walls are not a real structural element, they provide rigidity to the structure when they are assumed to act in-plane.

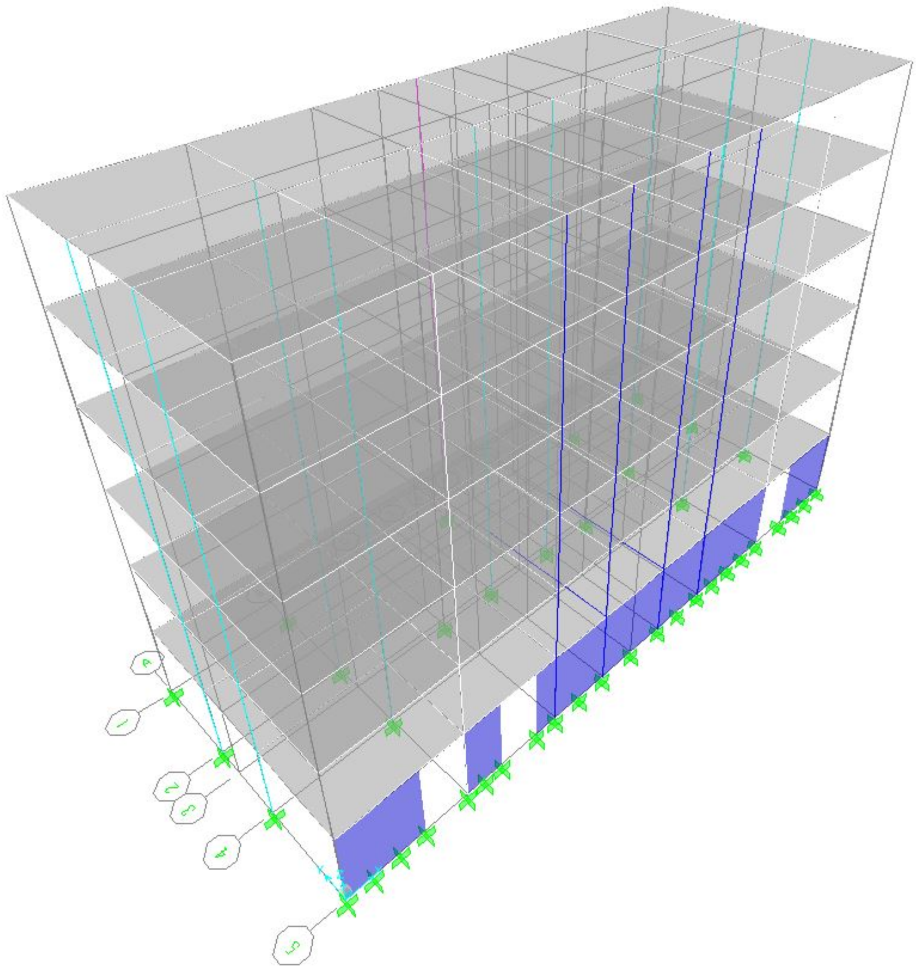

Figure 5.1. 3D view of Bare Frame Model.

Briefly, the structural analysis model, in which the partition walls are not physically located but only their effects are taken into account, will not be able to fully demonstrate the real dynamic behaviors that the structure will actually exhibit, even if it correctly demonstrates the internal forces that may occur in the structural elements like beams and columns.

Table 5.1. Dynamic properties of studied building obtained from Bare Frame Model.

Mode Number	Period (sec)	Frequency (Hz)	Mass Participation X-dir.	Mass Participation Y-dir.	Total Mass Participation X-dir.	Total Mass Participation Y-dir.
1	0.728	1.374	0.001	0.807	0.001	0.807
2	0.722	1.386	0.681	0.001	0.682	0.808
3	0.609	1.642	0.042	0.000	0.724	0.808
4	0.240	4.165	0.000	0.113	0.724	0.921
5	0.233	4.301	0.086	0.000	0.810	0.921
6	0.202	4.945	0.003	0.000	0.812	0.921
7	0.141	7.109	0.000	0.045	0.812	0.966
8	0.133	7.527	0.032	0.000	0.844	0.966
9	0.120	8.326	0.000	0.000	0.844	0.966
10	0.102	9.846	0.000	0.005	0.844	0.971

According to the AVT results given in detail in Chapter 3, the x directional modal frequency was found to be 4.60 Hz, the y directional modal frequency was found to be 4.02 Hz and the torsional modal frequency was found to be 5.44 Hz for the examined building. In the finite element model that the partition walls are ignored, modal analysis was performed using SAP2000.According to the analysis results given in Table 5.1, the x directional modal frequency (the second mode of the building) was obtained as 1.386 Hz, y directional modal frequency (the first mode of the building) was obtained as 1.374 Hz and torsional modal frequency (the third mode of the building) was obtained as 1.642 Hz. Also, the mode shapes obtained from numerical analysis of the first three modes are shown in Figure 5.2, Figure 5.3 and Figure 5.4, respectively.

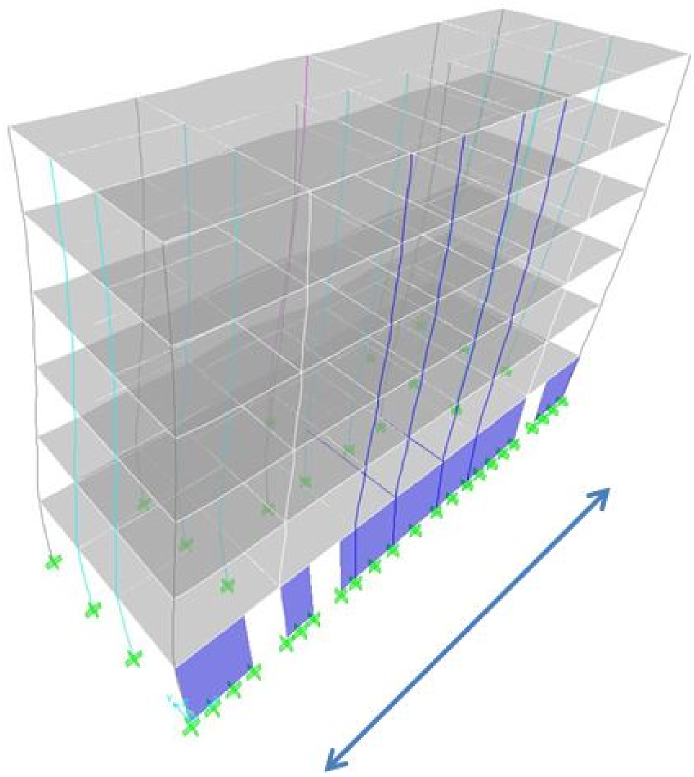

Figure 5.2. The illustration of the X directional movement: f=1.386 Hz; T=0.722 sec.

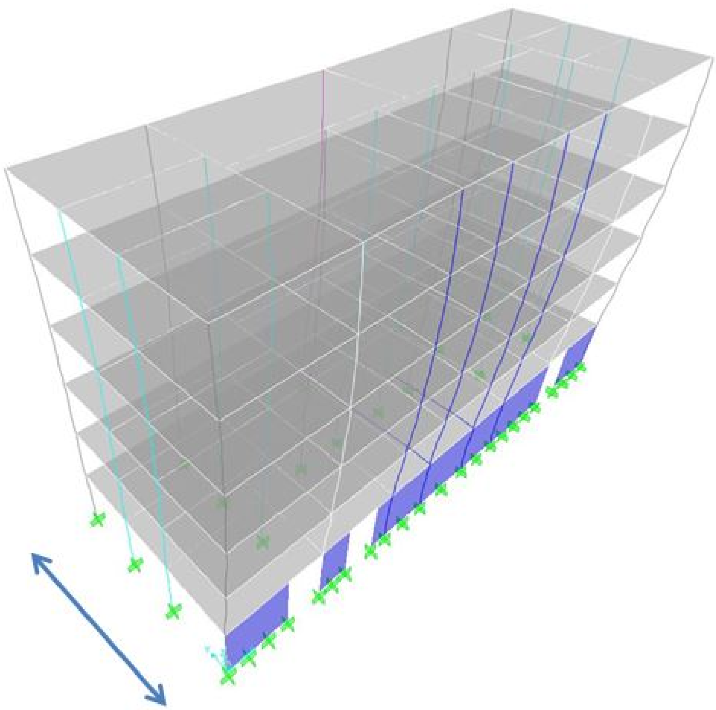

Figure 5.3. The illustration of the Y directional movement: f=1.374 Hz; T=0.728 sec.

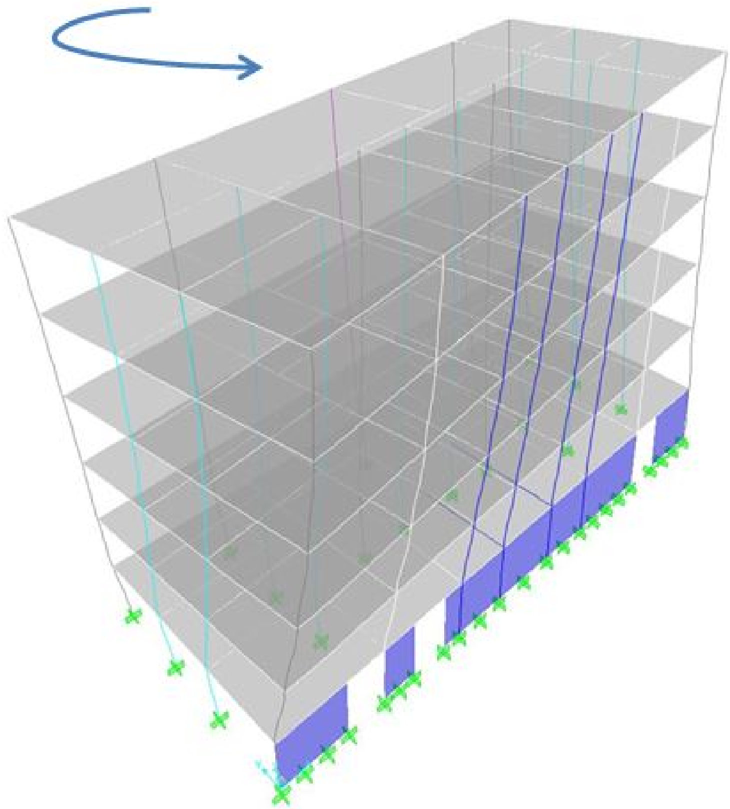

Figure 5.4. The illustration of the torsional movement: f=1.642 Hz; T=0.609 sec.

Table 5.2. Comparison of Bare Frame Model and AVT results in terms of dynamic properties.

Mode Number (Direction)	Bare Frame Model Analysis Results		Vibration Analysis Results		Difference
	T(s)	**f(Hz)**	**T(s)**	**f(Hz)**	**(%)**
Mode 1 (Y)	0.728	1.374	0.249	4.02	65.82
Mode 2 (X)	0.722	1.386	0.217	4.60	69.87
Mode 3 (Torsion)	0.609	1.642	0.184	5.44	69.82

According to the results summarized in Table 5.2, the results of the ambient vibration test and the bare frame model are very different from each other as expected. The differences were found to be 69.87% in the x direction, 65.82% in the y direction and 69.82% in the torsional direction. As a result, it is clear that infill walls increase the rigidity of RC buildings.

The mode shapes of the bare frame model were drawn as illustrated in Figure 5.5. Normalized modal displacements were computed by scaling the displacement values obtained from corner B of the bare frame model in SAP2000.

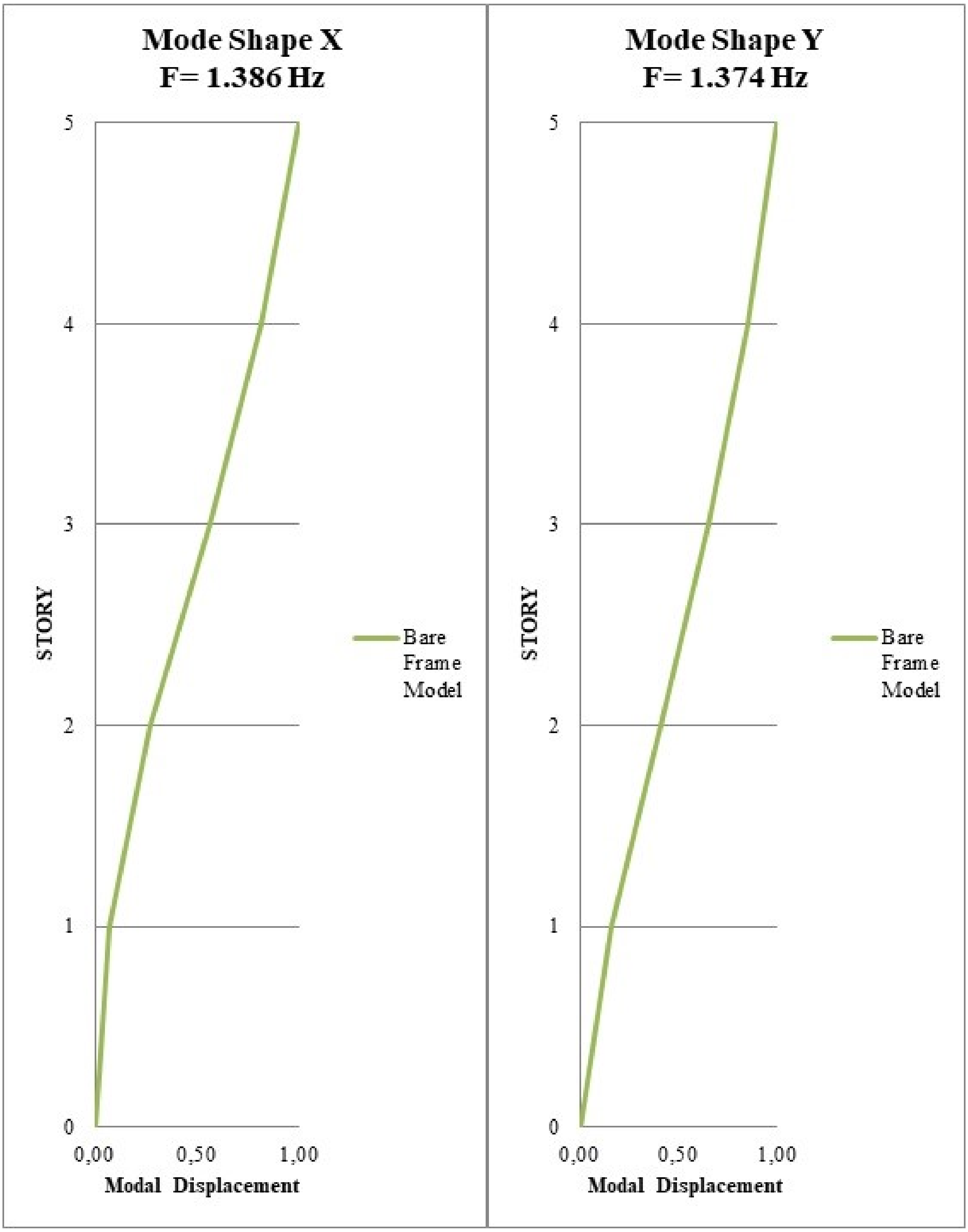

Figure 5.5. Normalized modal displacements obtained from Bare Frame Model.

5.2. Shell Model

In the second mathematical model designed to examine the effect of partition walls on structural behavior, partition walls are considered as the shell elements. Figure 5.6 and Figure 5.7 indicate the views of the 3D shell model. To estimate the real dynamic behavior of investigated building, finite element model including partition walls is created using SAP2000 software. In addition to structural elements such as columns and beams, non-structural partition walls are also modeled as shell elements like slabs. If there is any opening due to doors or windows these parts are left blank. As the ground floor columns are connected to the soil with fixed supports, the bottom nodal points of the non-structural walls on the ground floor must be connected to the soil to the fixed supports.

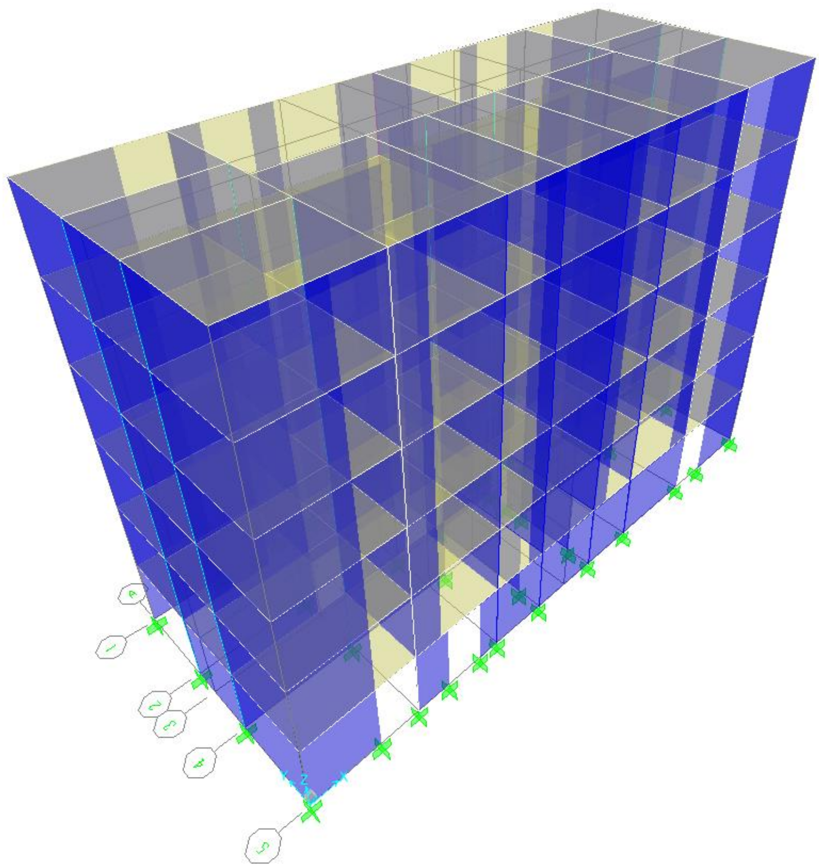

Figure 5.6. Front view of Shell Model obtained from SAP2000.

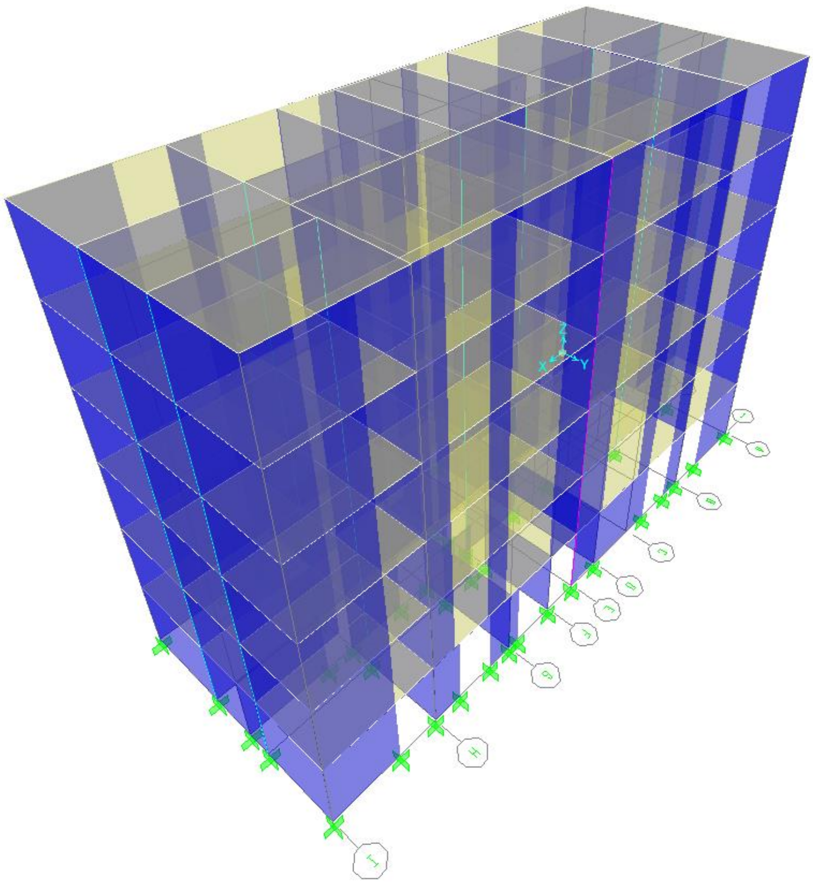

Figure 5.7. Rear view of Shell Model obtained from SAP2000.

The mechanical properties of the partition walls modeled in the finite element analysis were obtained from the laboratory experiments described in detail in Chapter 4. As a result of these experiments, the elasticity modulus of the partition walls was considered as 3400 MPa on average and the unit weight of such walls was considered as 9.45 kN/m^3. Infill wall material properties can be seen in Figure 5.8.

Live load isn't taken into account in the model due to the examined structure is completely abandoned. Not only were the self-weight of the structural (columns, beams and slabs) and non-structural (partition walls) elements but also floor coverings (2 kN/m^2) included in the model as dead load.

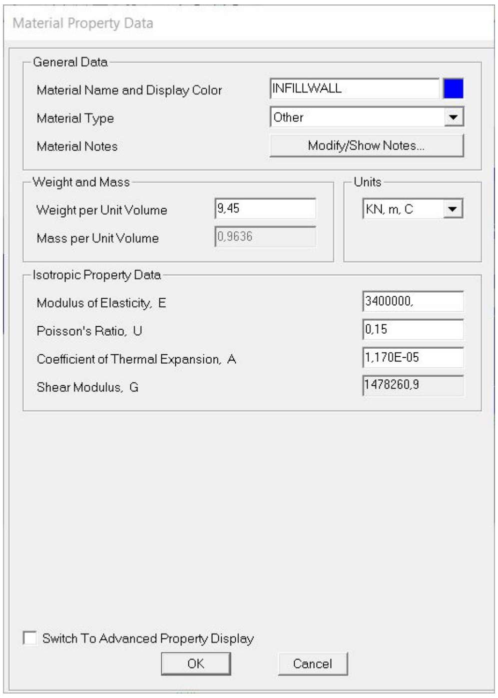

Figure 5.8. Partition wall material properties used in 3D Finite Element Model.

Table 5.3. Dynamic properties of studied building obtained from Shell Model.

Mode Number	**Period (sec)**	**Frequency (Hz)**	**Mass Participation**		**Total Mass Participation**	
			X-dir.	**Y-dir.**	**X-dir.**	**Y-dir.**
1	0.245	4.081	0.001	0.765	0.001	0.765
2	0.218	4.586	0.729	0.001	0.730	0.766
3	0.189	5.270	0.013	0.004	0.744	0.770
4	0.083	12.098	0.000	0.089	0.744	0.859
5	0.081	12.416	0.001	0.079	0.745	0.938
6	0.078	12.825	0.000	0.003	0.745	0.941
7	0.077	12.933	0.000	0.000	0.745	0.941
8	0.077	12.971	0.001	0.000	0.745	0.941
9	0.077	12.981	0.000	0.000	0.745	0.941
10	0.077	13.070	0.001	0.000	0.745	0.941

According to the AVT results given in detail in Chapter 3, the x directional modal frequency was found to be 4.60 Hz, the y directional modal frequency was found to be 4.02 Hz and the torsional modal frequency was found to be 5.44 Hz. Dynamic properties of the shell model are shown in Table 5.3. Based on the performed analysis results, the x directional modal frequency (the second mode of the building) was obtained as 4.586 Hz; y directional modal frequency (the first mode of the building) was obtained as 4.081 Hz and torsional modal frequency (the third mode of the building) was obtained as 5.270 Hz. Also, the mode shapes obtained from mathematical model of the first three modes are given in Figure 5.9, Figure 5.10 and Figure 5.11, respectively.

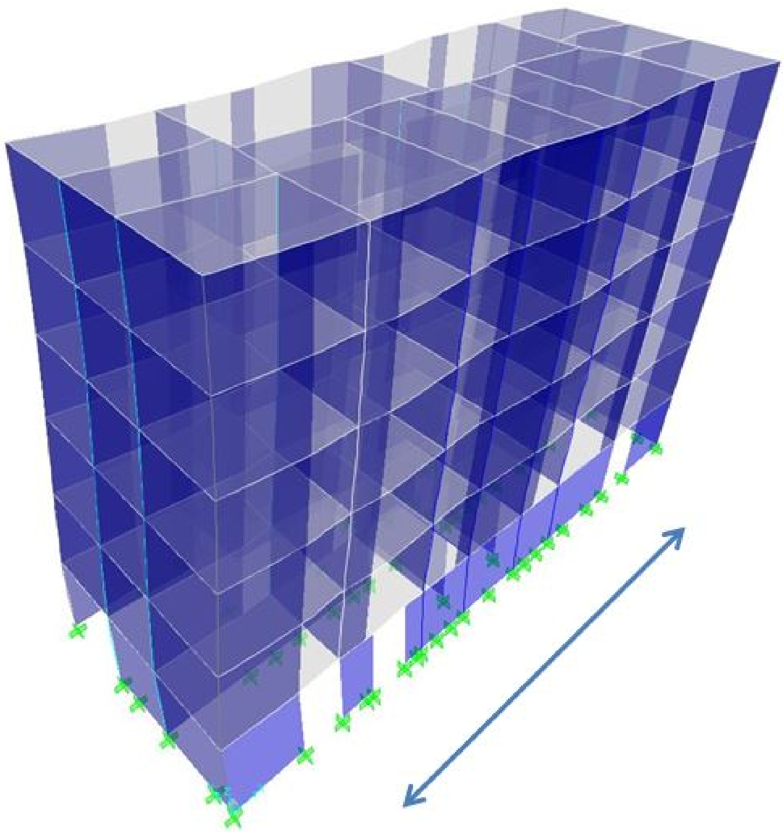

Figure 5.9. The illustration of the X directional movement: f=4.586 Hz; T=0.218 sec.

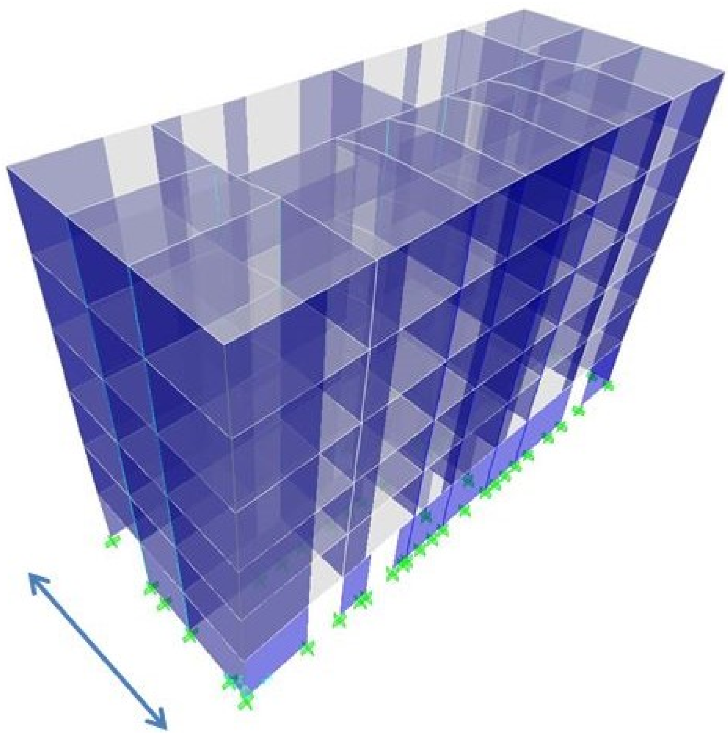

Figure 5.10. The illustration of the Y directional movement: f=4.081 Hz; T=0.245 sec.

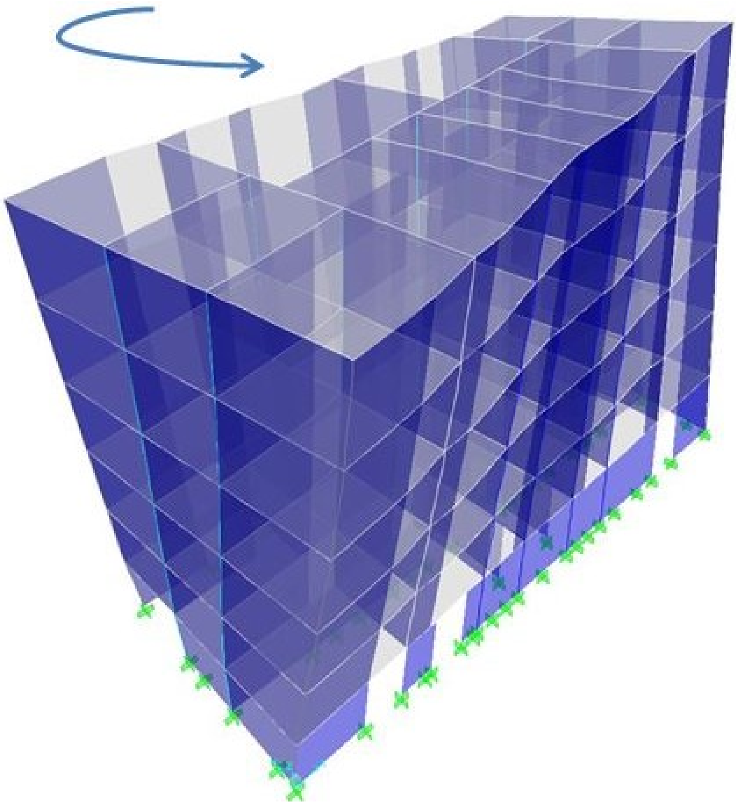

Figure 5.11. The illustration of the torsional movement: f=5.270 Hz; T=0.189 sec.

Table 5.4. Comparison of Shell Model and AVT results in terms of dynamic properties.

Mode Number (Direction)	Shell Model Analysis Results		Vibration Analysis Results		Difference
	T(s)	***f*(Hz)**	**T(s)**	***f*(Hz)**	**(%)**
Mode 1 (Y)	0.245	4.081	0.249	4.02	1.52
Mode 2 (X)	0.218	4.586	0.217	4.60	0.30
Mode 3 (Torsion)	0.189	5.270	0.184	5.44	3.13

As can be seen from the results given in Table 5.4, the results of the vibration analysis and the shell model are very close to each other at very small error rates. Acceptable results were obtained when compared the shell model with the vibration analysis:

- The difference was found to be 0.30% in the x direction,
- The difference was found to be 1.52% in the y direction,
- The difference was found to be 3.13% in the torsional direction.

Numerical analysis results satisfied the expectations and this model revealed the best matched results with the ambient vibration tests. As a result of obtained data, it is an exact approach to consider the non-structural walls in reinforced concrete buildings as shell elements in the finite element model.

The mode shapes of the shell model were drawn as shown in Figure 5.12. Normalized modal displacements were calculated by scaling the displacement values obtained from corner B of the shell model in SAP2000.

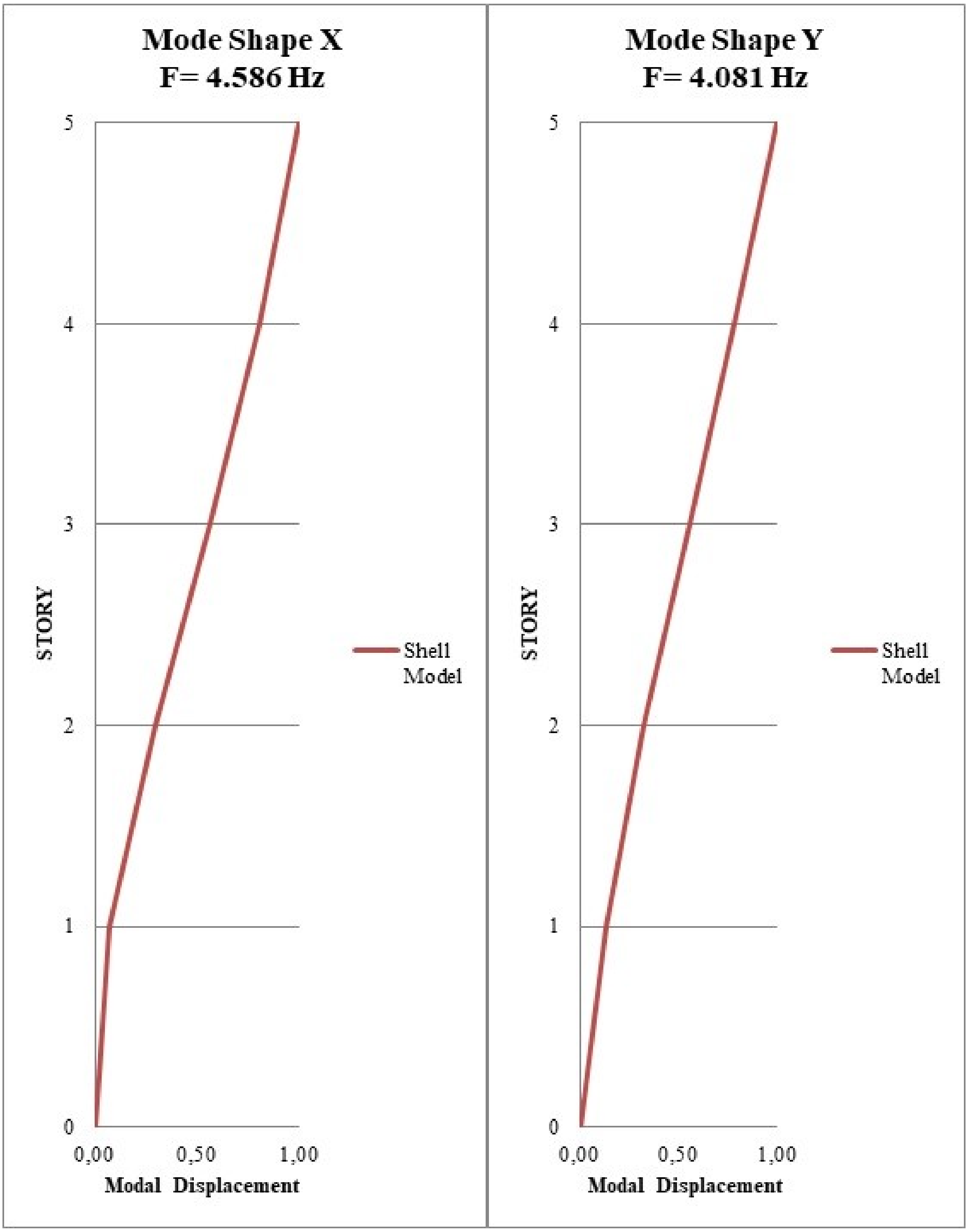

Figure 5.12. Normalized modal displacements obtained from Shell Model.

5.3. Single Strut Model - 1

In the third mathematical model designed to examine the effect of partition walls on structural behavior, partition walls are considered as the one – way equivalent single strut elements (Figure 5.13). The material properties of the equivalent single strut are the same as the material properties of the wall. To model the mechanical properties of non-structural brick walls accurately, the data obtained from the experimental surveys were used. The widths of the equivalent single struts were calculated according to the Turkish Earthquake Code 2018. When modelling the equivalent single struts in SAP2000, it is taken into consideration that the single strut element has two ends with hinged and it is connected diagonally to the corner nodes of the framework of the subjected partition wall. In this way, the equivalent single struts represent the rigidity of the partition walls and meet only axial stresses. Thus, such struts are not subjected to moment stresses.

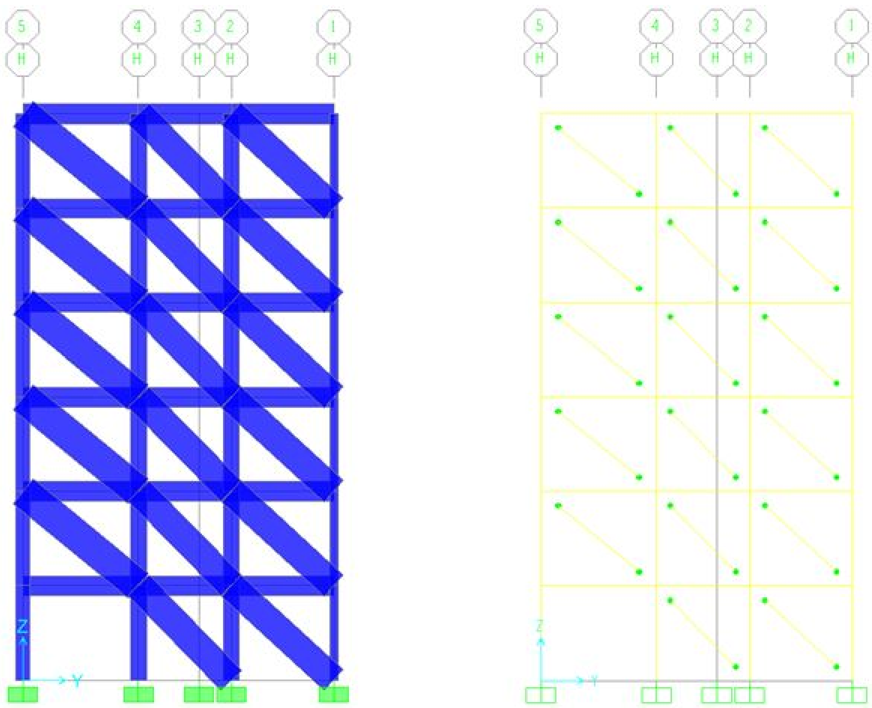

Figure 5.13. Representation of One – way Equivalent Single Struts and Releases on 2D frame.

When creating the equivalent single struts in numerical model, openings were taken into account to obtain more realistic results. Therefore, if there is any opening due to doors or windows these parts are left blank, physically.

For each individual equivalent single strut, the width of the strut (mm) is calculated using the formula as shown below.

$$a_{wall} = 0.175(\lambda_{wall} h_k)^{-0.4} r_{wall} \tag{5.1}$$

where h_k is the height of the column (mm) replaced to the two side of the subjected partition wall, r_{wall} is the diagonal length of the partition wall (mm). λ_{wall} can be calculated using equation 5.2.

$$\lambda_{wall} = \left[\frac{E_{wall} t_{wall} \sin 2\Theta}{4 E_c I_k h_{wall}}\right]^{\frac{1}{4}} \tag{5.2}$$

where E_{wall} is the elasticity modulus of the partition wall (MPa), t_{wall} is the thickness of the partition wall (mm), Θ is the angle between the diagonal and the horizontal axis, E_c is the elasticity modulus of the frame concrete (MPa), I_k represents the inertia momentum of the frame column (mm^4) and h_{wall} is the height of the partition wall (mm).

Also, the axial rigidity of a diagonal compression strut element could be obtained using formula given in 5.3.

$$k_{wall} = \frac{a_{wall} t_{wall} E_{wall}}{r_{wall}} \tag{5.3}$$

Table 5.5. Dynamic properties of studied building obtained from Single Strut Model-1.

Mode Number	Period (sec)	Frequency (Hz)	Mass Participation X-dir.	Mass Participation Y-dir.	Total Mass Participation X-dir.	Total Mass Participation Y-dir.
1	0.403	2.482	0.037	0.782	0.037	0.782
2	0.369	2.711	0.642	0.053	0.679	0.835
3	0.321	3.111	0.057	0.004	0.737	0.839
4	0.137	7.279	0.005	0.110	0.742	0.950
5	0.125	8.005	0.077	0.009	0.819	0.958
6	0.110	9.096	0.011	0.000	0.830	0.959
7	0.081	12.378	0.000	0.004	0.830	0.963
8	0.080	12.539	0.005	0.018	0.834	0.981
9	0.077	13.034	0.003	0.002	0.838	0.983
10	0.075	13.277	0.021	0.001	0.859	0.985

According to the AVT results given in detail in Chapter 3, the x directional modal frequency was found to be 4.60 Hz, the y directional modal frequency was found to be 4.02 Hz and the torsional modal frequency was found to be 5.44 Hz for the examined building. In the finite element model that the partition walls in the building are considered as one – way equivalent single strut elements, modal analysis was performed using SAP2000.

Based on the completed analysis results given in Table 5.5, the x directional modal frequency (the second mode of the building) was obtained as 2.711 Hz, y directional modal frequency (the first mode of the building) was obtained as 2.482 Hz and torsional modal frequency (the third mode of the building) was obtained as 3.111 Hz.

Also, the mode shapes obtained from finite element model of the first three modes are shown in Figure 5.14, Figure 5.15 and Figure 5.16, respectively.

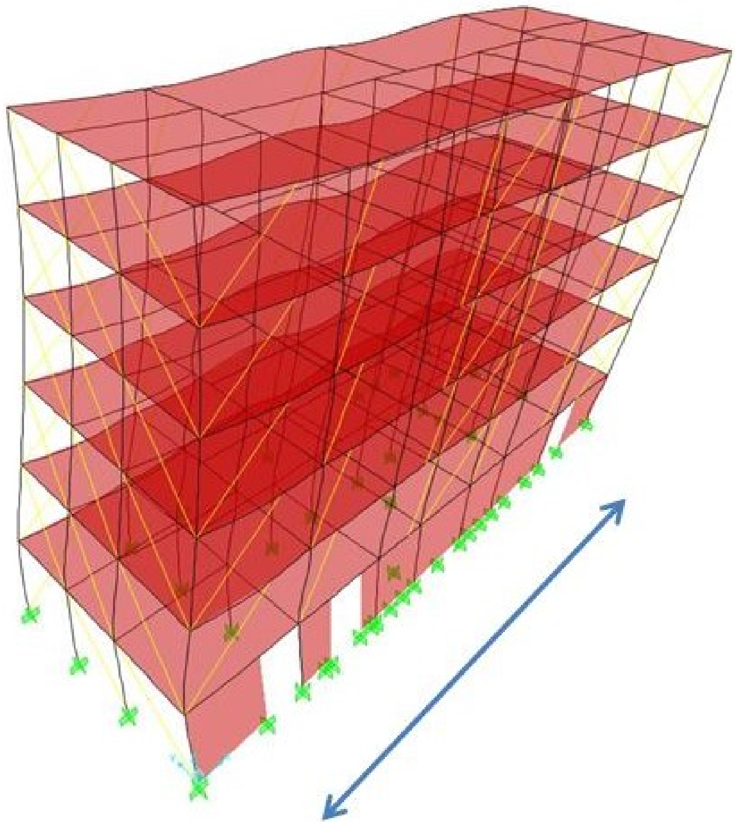

Figure 5.14. The illustration of the X directional movement: f=2.711 Hz; T=0.369 sec.

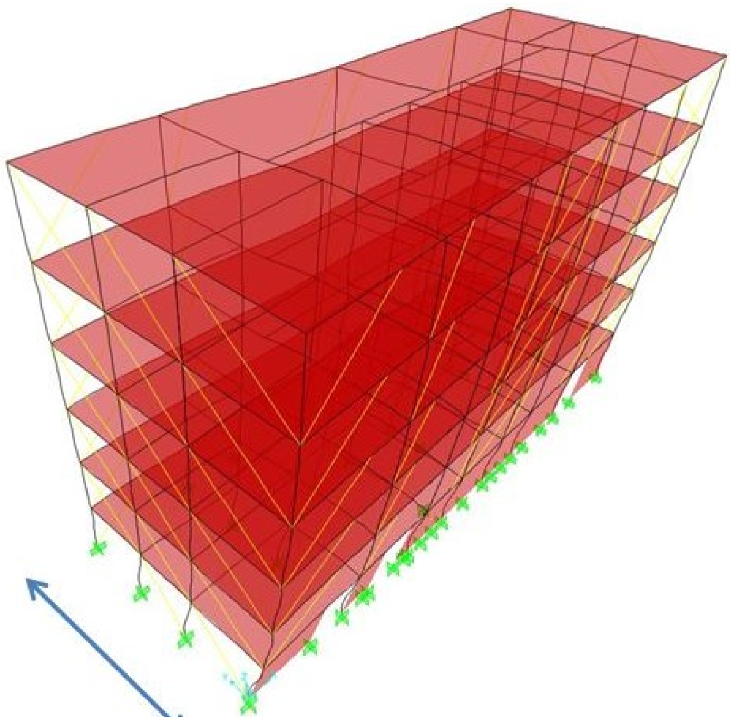

Figure 5.15. The illustration of the Y directional movement: f=2.482 Hz; T=0.403 sec.

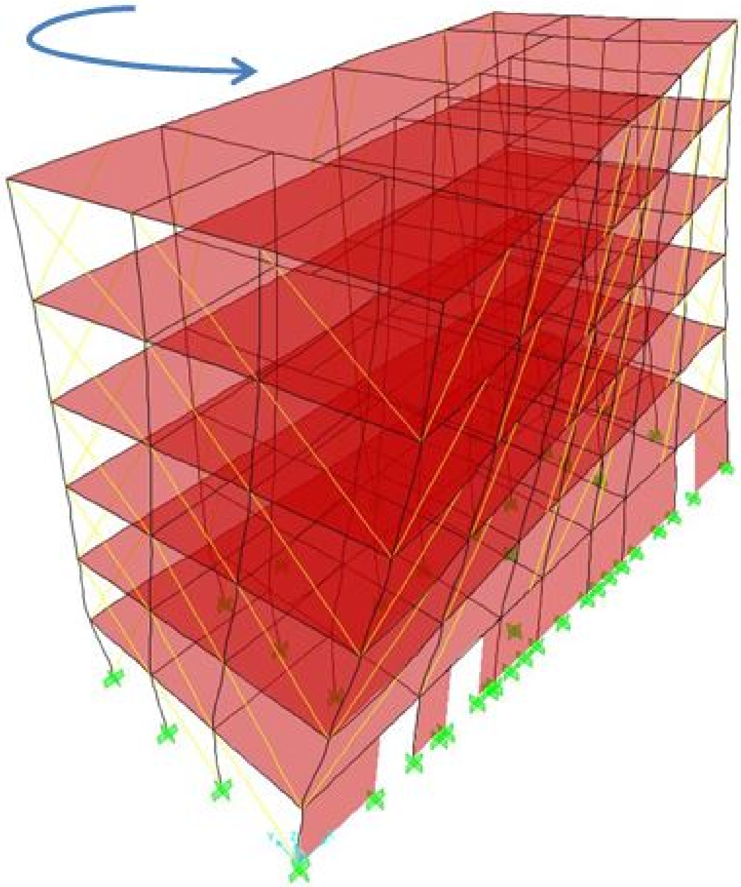

Figure 5.16. The illustration of the torsional movement: f=3.111 Hz; T=0.321 sec.

Table 5.6. Comparison of Single Strut Model – 1 and AVT results in terms of dynamic properties.

Mode Number (Direction)	**Single Strut Model – 1 Analysis Results**		**Vibration Analysis Results**		**Difference (%)**
	T(s)	**f(Hz)**	**T(s)**	**f(Hz)**	
Mode 1 (Y)	0.403	2.482	0.249	4.02	38.26
Mode 2 (X)	0.369	2.711	0.217	4.60	41.07
Mode 3 (Torsion)	0.321	3.111	0.184	5.44	42.81

As can be seen from the results given in Table 5.6, the results of the vibration analysis and the single strut model – 1 are not close enough to each other. The differences were found to be 41.07% in the x direction, 38.26% in the y direction and 42.81% in the torsional direction. The frequency values of this model were expected to be between the frequency values of the bare frame model and shell model.

As a result of obtained data, modeling the partition walls as one – way equivalent single strut is a partially correct approach.

The mode shapes of the single strut model – 1 were drawn as demonstrated in Figure 5.17. Normalized modal displacements were figured out by scaling the displacement values obtained from corner B of the single strut model – 1 in SAP2000.

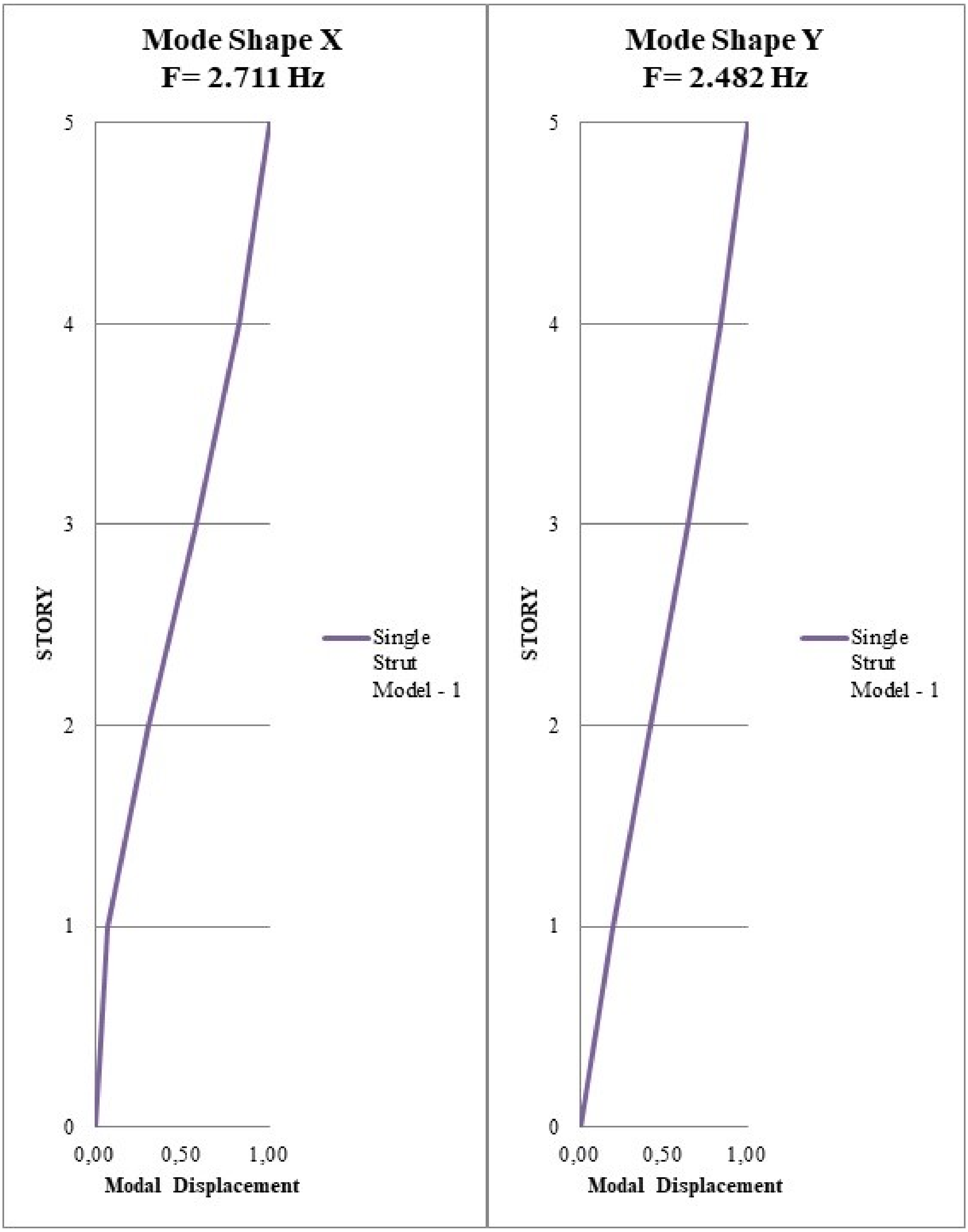

Figure 5.17. Normalized modal displacements obtained from Single Strut Model – 1.

5.4. Single Strut Model - 2

In the fourth mathematical model designed to examine the effect of partition walls on structural behavior, partition walls are considered as two – way equivalent single strut elements (Figure 5.18). The conditions mentioned in single strut model – 1 are also valid for the single strut model – 2. The only difference is that diagonal struts are used in both directions instead of one direction considered as in previous analytical model. Since the building under investigation is only exposed to environmental influences like wind or human activities, there is no earthquake effect on the current situation of the building. In the scope of this study, dynamic parameters of the building were examined under very small vibrations. In this respect, using two – way equivalent single strut is a more accurate approach because there won't be any detachment between frames and infill walls due to the small displacements.

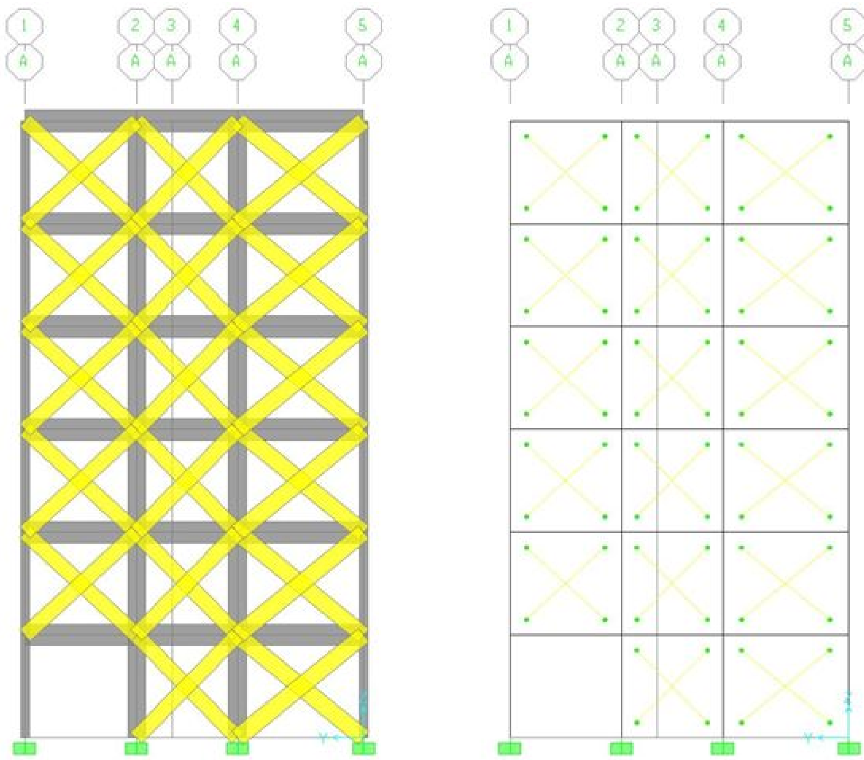

Figure 5.18. Representation of Two – way Equivalent Single Struts and Releases on 2D frame.

Table 5.7. Dynamic properties of studied building obtained from Single Strut Model-2.

Mode Number	Period (sec)	Frequency (Hz)	Mass Participation X-dir.	Mass Participation Y-dir.	Total Mass Participation X-dir.	Total Mass Participation Y-dir.
1	0.332	3.008	0.013	0.802	0.013	0.802
2	0.312	3.201	0.652	0.019	0.666	0.821
3	0.262	3.810	0.072	0.001	0.737	0.822
4	0.115	8.724	0.001	0.134	0.739	0.956
5	0.103	9.664	0.082	0.004	0.821	0.960
6	0.091	11.030	0.020	0.001	0.841	0.961
7	0.078	12.804	0.000	0.000	0.841	0.961
8	0.073	13.623	0.000	0.000	0.841	0.961
9	0.067	14.836	0.000	0.004	0.841	0.965
10	0.065	15.333	0.001	0.000	0.842	0.965

According to the AVT results given in detail in Chapter 3, the x directional modal frequency was found to be 4.60 Hz, the y directional modal frequency was found to be 4.02 Hz and the torsional modal frequency was found to be 5.44 Hz for the examined building. In the finite element model that the partition walls in the building are considered as two – way equivalent single strut elements, modal analysis was performed using SAP2000.

Based on the completed analysis results given in Table 5.7, the x directional modal frequency (the second mode of the building) was obtained as 3.201 Hz, y directional modal frequency (the first mode of the building) was obtained as 3.008 Hz and torsional modal frequency (the third mode of the building) was obtained as 3.810 Hz.

Also, the mode shapes obtained from finite element model of the first three modes are shown in Figure 5.19, Figure 5.20 and Figure 5.21, respectively.

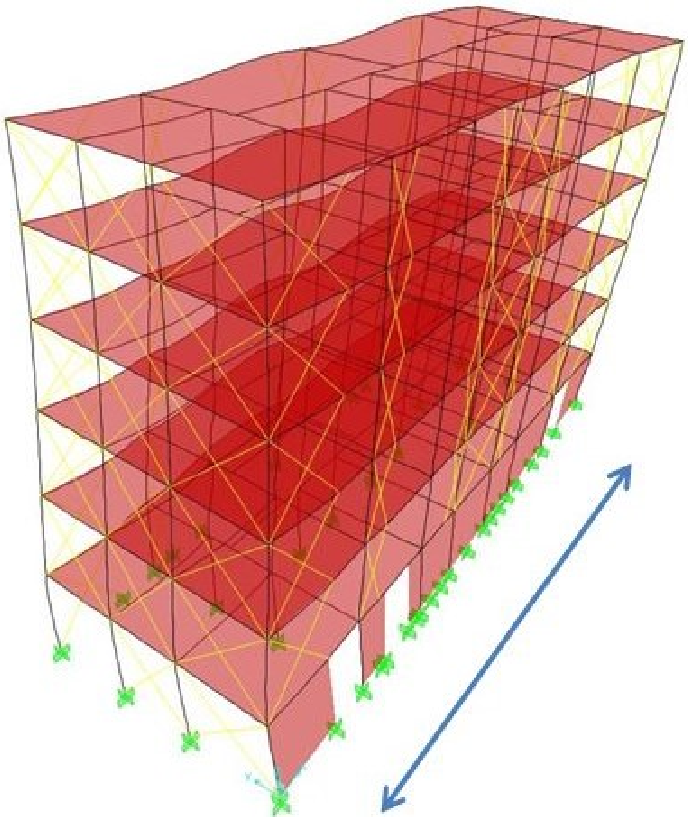

Figure 5.19. The illustration of the X directional movement: f=3.201 Hz; T=0.312 sec.

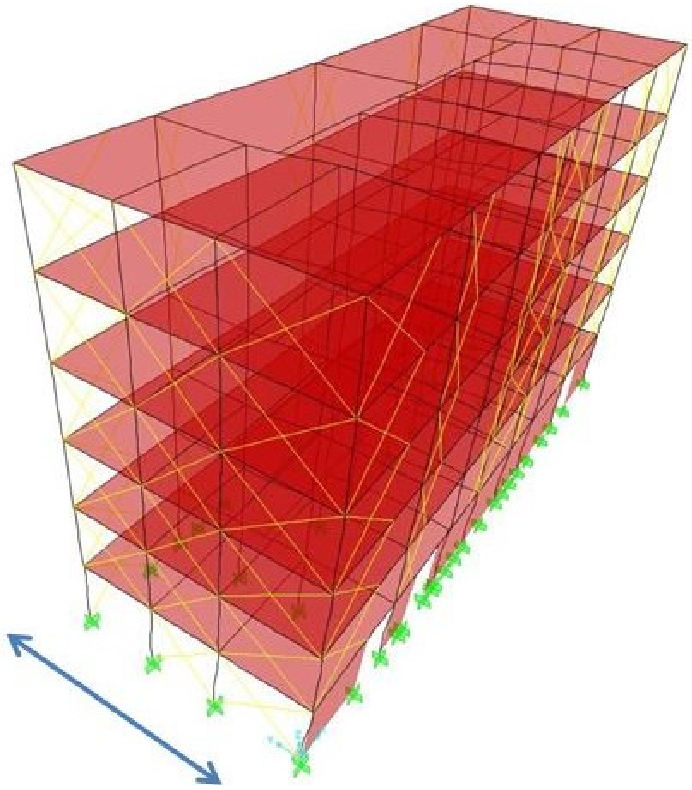

Figure 5.20. The illustration of the Y directional movement: f=3.008 Hz; T=0.332 sec.

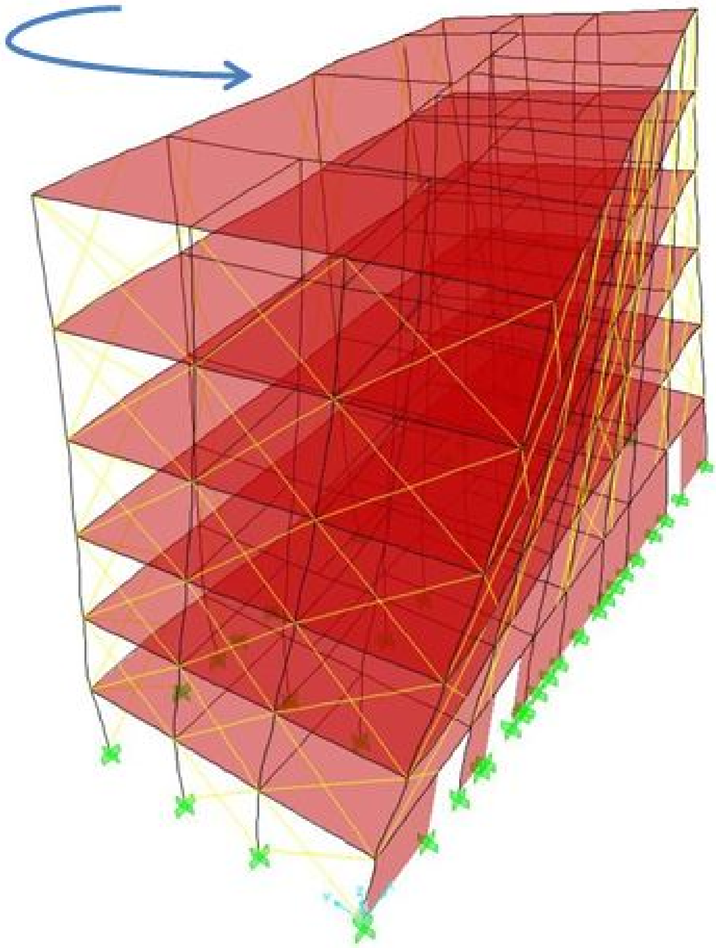

Figure 5.21. The illustration of the torsional movement: f=3.810 Hz; T=0.262 sec.

Table 5.8. Comparison of Single Strut Model – 2 and AVT results in terms of dynamic properties.

Mode Number (Direction)	**Single Strut Model – 2 Analysis Results**		**Vibration Analysis Results**		**Difference (%)**
	T(s)	**f(Hz)**	**T(s)**	**f(Hz)**	
Mode 1 (Y)	0.332	3.008	0.249	4.02	25.17
Mode 2 (X)	0.312	3.201	0.217	4.60	30.41
Mode 3 (Torsion)	0.262	3.810	0.184	5.44	29.96

As can be seen from the results given in Table 5.8, the results of the vibration analysis and the single strut model – 2 are not close enough to each other. The differences were found to be 30.41% in the x direction, 25.17% in the y direction and 29.96% in the torsional direction. In this model, differences decreased by approximately 10% in each direction compared to the single strut model – 1.

As expected, modeling the partition walls as two – way equivalent single strut is more accurate approach than the one – way equivalent single strut.

The mode shapes of the single strut model – 2 were drawn as demonstrated in Figure 5.22. Normalized modal displacements were figured out by scaling the displacement values obtained from corner B of the single strut model – 2 in SAP2000.

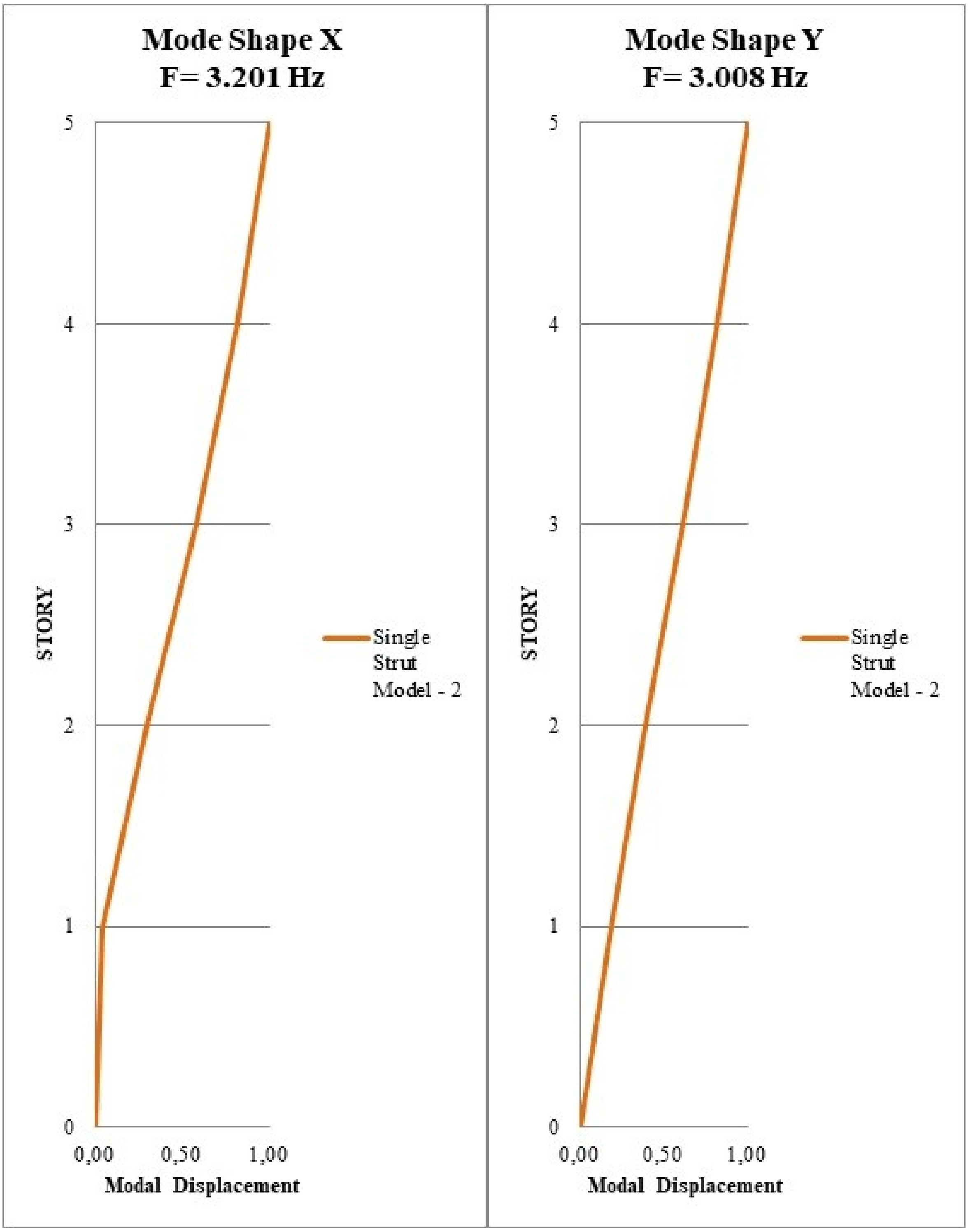

Figure 5.22. Normalized modal displacements obtained from Single Strut Model – 2.

5.5. Comparison of the Results Obtained from AVT and Mathematical Models

General comparisons of the mathematical models and AVT results are drawn in Figure 5.23. As can be seen on the table, the shell model and AVT analyses give more overlapping results with each other when compared the other models. It means that the shell model is the best model to represent the actual behavior of the building. Also, considering the bare frame model, the absence of partition walls affects the natural frequencies of the old RC buildings with almost 65-70%. Moreover, the single strut model – 1 demonstrates that the assumption of the partition walls as one – way strut elements could change the natural frequencies of the buildings with nearly 40%. If the partition walls are considered as two – way strut elements the change in the natural frequency values will be approximately 25 – 30%. It is pointed out that to create shell model with a real mechanical properties obtained from experimental tests is the best way to estimate the real dynamic properties of the RC buildings.

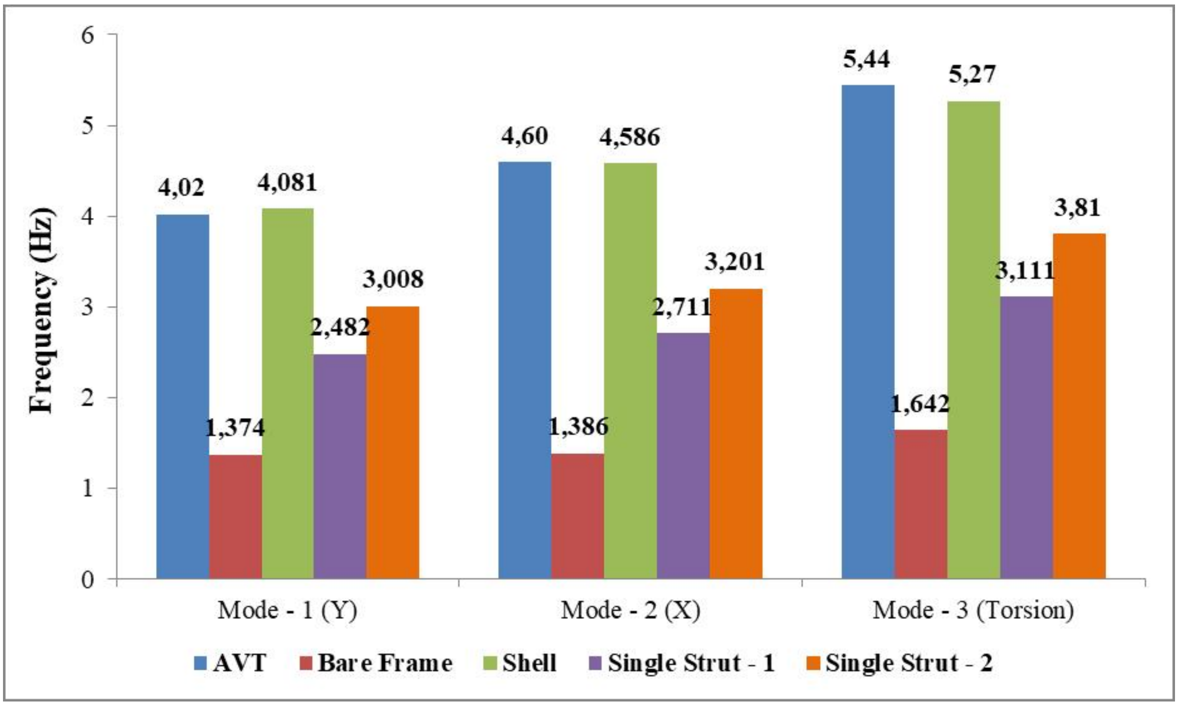

Figure 5.23. General comparison diagram of SAP2000 models and AVT results in terms of frequency values.

Modal assurance criterion (MAC) values are calculated as shown in Table 5.9. All analytical models were compared to the AVT results in terms of mode shapes. Comparison values are larger than 0.90, so analytical models are consistent.

Table 5.9. MAC values obtained to the comparison of analytical models with AVT.

MAC	**AVT - BARE FRAME**	**AVT - SHELL**	**AVT - SINGLE STRUT - 1**	**AVT - SINGLE STRUT - 2**
Direction X	0.92	0.93	0.93	0.92
Direction Y	0.96	0.94	0.97	0.97

General comparison of normalized modal displacements is summarized in Figure 5.24. To conclude, analytical models exhibit very close mode shapes to each other.

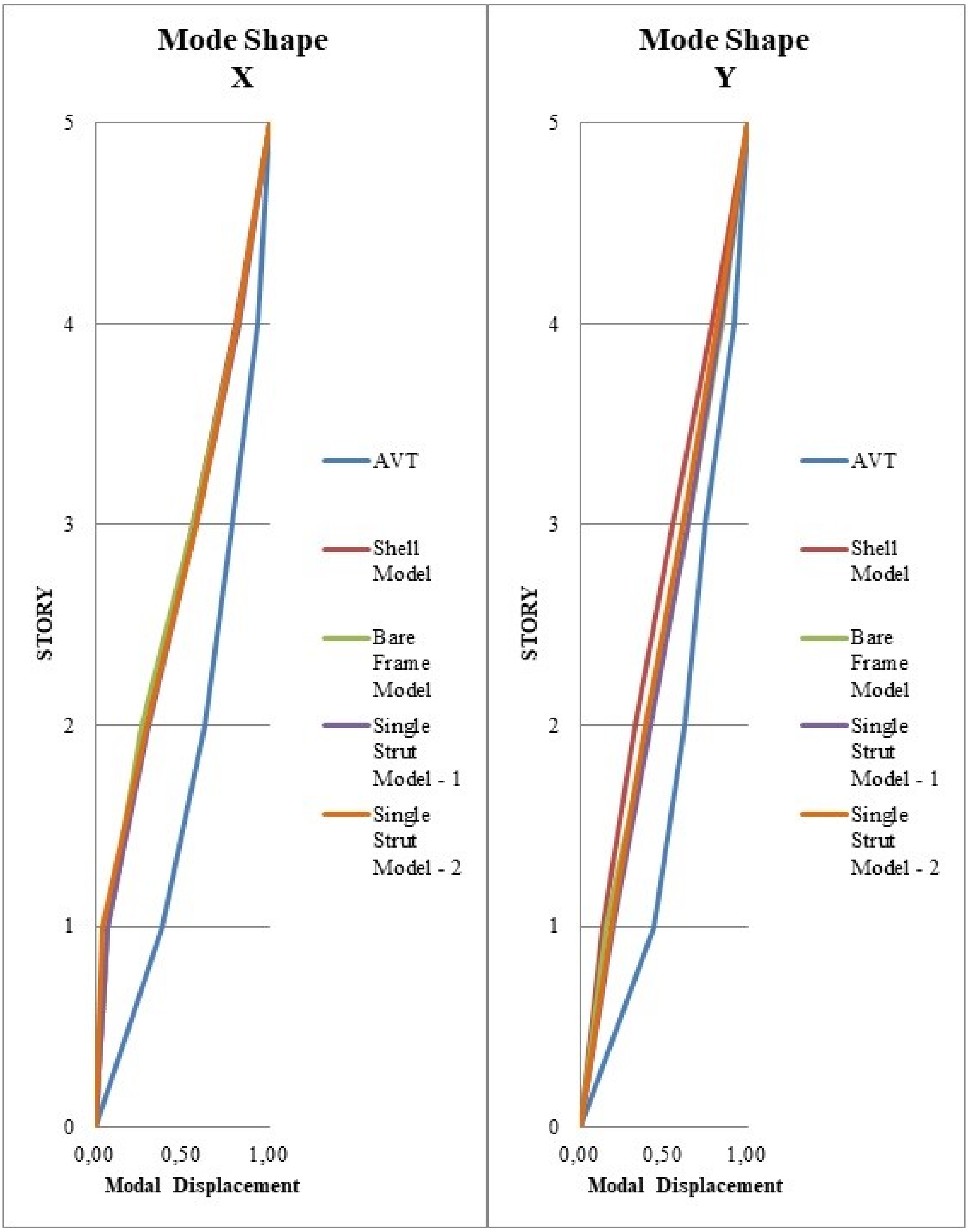

Figure 5.24. General comparison diagram of SAP2000 models and AVT results in terms of normalized modal displacements.

6. CONCLUSION

6.1. Results

In this section, principal conclusions obtained from the study carried out in the content of this thesis have been submitted. Although main results were already presented in the relevant chapters earlier, it is significant to summarize the results in this chapter in order to make the study more comprehensible. As a result, all findings obtained from the site surveys, laboratory tests and analytical models are listed below:

(i) Ambient vibration test was conducted to determine the real dynamic behavior of the studied building. AVT results showed that modal frequencies were 4.60 Hz for direction X, 4.02 Hz for direction Y and 5.44 Hz for torsion. These values were used to compare whether the mathematical models reflect the actual behavior.

(ii) In the scope of this thesis, compression and shear tests were performed in the structures laboratory of Boğaziçi University to obtain the real mechanical properties of the partition (infill) walls.

(iii) Six specimens have been extracted from the studied building to apply compression tests. As a result of such tests, the characteristic compressive strength was found to be 0.8 MPa and the modulus of elasticity was computed as 3400 MPa in accordance with the rules of EN 1052 – 1.

(iv) Eight specimens have been extracted from the studied building to apply shear tests. As a result of such tests, the characteristic shear strength was calculated as 0.21 MPa in accordance with the rules of EN 1052 – 3.

(v) Four mathematical models were created using SAP2000 as bare frame, shell, single strut – 1 and single strut – 2 models. The ultimate objective of the numerical analyses was to determine the effects of partition walls on dynamic behaviors of RC buildings. Also, these analyses helped to find the best model representing the actual behavior of the studied building when compared to the AVT results.

(vi) The bare frame model was established to reveal how partition walls affect the dynamic behavior of the RC buildings. For this reason, only beams, columns, slabs and shear walls were considered as structural elements in the model and the partition walls were ignored. The bare frame model analysis demonstrated that modal frequencies were 1.386 Hz for X direction, 1.374 Hz for Y direction and 1.642 Hz for torsion. The differences were obtained as 69.87% in X direction, 65.82% in Y direction and 69.82% in torsion, when compared to the AVT results. The fact that the differences are so high allows us to clearly see how the partition walls affect the dynamic behaviors of the structures. Briefly, the conclusion is that partition walls provide rigidity for the RC buildings and they help to decrease natural periods of the buildings.

(vii) The shell model was created to estimate the actual dynamic properties of studied building. For this purpose, partition walls were considered as shell elements and real mechanical properties obtained from laboratory tests were used in the FEM. The shell model analysis showed that modal frequencies were 4.586 Hz for X direction, 4.081 Hz for Y direction and 5.270 Hz for torsion. The differences were observed as 0.30% in X direction, 1.52% in Y direction and 3.13% in torsion, when compared to the AVT results. To conclude, findings are so close to each other and differences exist with a small error rates. The data obtained from the shell model demonstrate that created model accurately reveals the actual behavior of the building.

(viii) The single strut model – 1 was designed to determine the efficiency of the one – way equivalent single struts used in analytical models instead of partition walls. To this respect, partition walls were considered as one – way equivalent single struts in the FEM. The strut model – 1 analysis revealed that modal frequencies were 2.711 Hz for X direction, 2.482 Hz for Y direction and 3.111 Hz for torsion. The differences were pointed out as 41.07% in X direction, 38.26% in Y direction and 42.81% in torsion, when compared to the AVT results. According to the results, it is obvious that one – way equivalent single struts could partially replace the partition walls.

(ix) The single strut model – 2 was established to determine the efficiency of the two – way equivalent single struts used in analytical models instead of partition walls. In this respect, partition walls were considered as two – way equivalent single struts in the FEM. The strut model – 2 results showed that modal frequencies were 3.201 Hz for X direction, 3.008 Hz for Y direction and 3.810 Hz for torsion. The differences were found as 30.41% in X direction, 25.17% in Y direction and 29.96% in torsion, when compared to the AVT results. According to the results, it is obvious that the single strut model – 2 give closer results to the current conditions compared to the single strut model – 1.

(x) If the analyses were performed under earthquake loads it would have been correct to consider the strut model – 1 due to the frame – infill detachment on cross corner points of the partition walls. However, our assumption was that there was no earthquake loads, so the frame – infill detachment effect was ignored. For this reason, the strut model – 2 reflects our assumption better.

(xi) All in all, general comparisons of the mathematical models and AVT results showed that the shell model and AVT analyses give more overlapping results with each other when compared the other models. It means that the shell model is the best model to represent the actual behavior of the building. Also, considering the bare frame model, the absence of partition walls affects the natural frequencies of the old RC buildings with almost 65-70%. Moreover, the single strut model – 1 demonstrates that the assumption of the partition walls as one – way strut elements could change the natural frequencies of the buildings with nearly 40%. If the partition walls are considered as two – way strut elements the change in the natural frequency values will be approximately 25 – 30%. It is pointed out that to create shell model with a real mechanical properties obtained from experimental tests is the best way to estimate the real dynamic properties of the RC buildings.

(xii) Modal assurance criterion (MAC) values were calculated and all analytical models were compared to the AVT results in terms of mode shapes. Comparison values are larger than 0.90, so analytical models are consistent.

(xiii) According to the general comparison of normalized modal displacements, it was concluded that analytical models exhibit very close mode shapes to each other.

(xiv) This study aims to investigate the current state of the building, so partition walls are modeled in different ways. If a study was carried out on the performance assessment of partition walls, it would be more appropriate to consider the partition walls as strut elements, as proposed in the Turkish Earthquake Code.

6.2. Recommendations

Within the scope of the thesis, it is obvious that the uncertainties that may arise when determining the mechanical properties of partition walls such as the modulus of elasticity are not taken into consideration. Since experimental studies have been conducted to determine the mechanical properties of the partition walls and the actual values obtained from these experiments are used. However, in every study on partition walls, it may not be possible to perform experimental studies to determine the mechanical properties. To this respect, this part of study aims to give an idea about to what extent the modulus of elasticity could affect the results of analysis.

The changes in the modulus of elasticity were investigated under six different shell model cases in order to observe the variation of the dynamic properties. The modulus of elasticity was calculated as 3400 MPa according to the results of the experiment given in Chapter – 4. The fact that experimental studies have been done will allow the comparison of the data obtained from case studies with the real values. Also, the details of the six different cases created by increasing or decreasing the value of obtained elasticity modulus are as follows;

- Case – 1 ; E = 2720 MPa (20% decreased modulus of elasticity)
- Case – 2 ; E = 2000 MPa (≈40% decreased modulus of elasticity)
- Case – 3 ; E = 1020 MPa (70% decreased modulus of elasticity)
- Case – 4 ; E = 4080 MPa (20% increased modulus of elasticity)
- Case – 5 ; E = 4760 MPa (40% increased modulus of elasticity)
- Case – 6 ; E = 5780 MPa (70% increased modulus of elasticity)

The modulus of elasticity of hollow bricks is suggested as 2000 MPa in Turkish Earthquake Code 2018. This means that the given value is equal to the modulus of elasticity achieved by almost 40% reduction of the result obtained from laboratory tests. This may also be interpreted as the safety factor of the specification.

In Case – 2, the modulus of elasticity suggested by Turkish Earthquake Code 2018 were used in shell model. As can be seen on result table, the obtained modal frequency and actual modal frequency values are 15 – 20% different from each other. If laboratory tests were not performed and the modulus of elasticity proposed by the specification was used in the model, the results would have been 15 – 20% different than what were actual values. As a result of case studies, the values of frequency, period and differences (between the analysis and actual results) for each case are summarized in Table 6.1.

Table 6.1. The relationship between the variation of the modulus of elasticity and the dynamic properties of the shell model.

Modal Parameters		**Case-1 20% ↓**	**Case-2 40% ↓**	**Case-3 70% ↓**	**Case-4 20% ↑**	**Case-5 40% ↑**	**Case-6 70% ↑**
MODE - 1 (Y)	***f*** **(Hz)**	3.786	3.417	2.736	4.342	4.577	4.895
	T (sec)	0.264	0.293	0.365	0.23	0.218	0.204
	Difference %	-5.80	-14.98	-31.92	8.04	13.88	21.80
MODE - 2 (X)	***f*** **(Hz)**	4.281	3.885	3.113	4.848	5.079	5.382
	T (sec)	0.234	0.257	0.321	0.206	0.197	0.186
	Difference %	-7.84	-16.36	-32.98	4.37	9.34	15.87
MODE - 3 (Torsion)	***f*** **(Hz)**	4.864	4.36	3.452	5.631	5.957	6.397
	T (sec)	0.206	0.229	0.29	0.178	0.168	0.156
	Difference %	-10.87	-20.10	-36.74	3.19	9.16	17.23

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