The Effect of Mass Irregularities on the Response of Building

(MAJOR PROJECT REPORT)

Submitted in partial fulfillment of the requirements for the award of degree of

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Submitted by

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CERTIFICATE

This is to certify that this report entitled, **“The Effect of mass irregularities on the response of buildings”,** is report of the Major Project-II done by me. This is a bona fide record of my own work carried by me under the guidance and supervision of Mr. Alok Verma, Associate Professor, in partial fulfillment of the requirement for the award of the Degree of Master of Technology in Civil Engineering(Structural Engineering) by the Delhi Technological University, Delhi.
The matter embodied in this project has not been submitted for the award of any other degree anywhere else.

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

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ABSTRACT

Response and behavior of regular buildings has been specified to be better in most of the circumstances compare irregular ones. Different codes of practices for design and analysis of civil engineering structure specify different type of irregularities.

Mass irregularity has been clearly defined by IS 1893 (Part 1) : 2002 to be a degrading influence on buildings. Apart from near presence of mass irregularity its location also is an important parameter.

Different types of buildings such as framed buildings and/or stiff masonry constructions behave differently against earthquake forces. This difference is all the more prominent if mass irregularities are present.

Considering advances in building constructions including more frequent construction of framed buildings it seems appropriate to study the effect of mass irregularity is such buildings.

In the present study mass irregularity in framed building is considered. Due to irregular variation in mass the dynamic characteristics of the buildings are studied. In this study an aluminium framed model of a three storey building is taken. The mass irregularity is created by putting additional masses at different floor level. The eigenvalue analysis is undertaken for the framed model. The calculated eigenvalues are compared with values obtained from Etabs software analysis results. It is found that the results obtained in the cases of theoretical analysis, experimental programme and in the case of use of Etabs software are comparable.

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CHAPTER 1

INTRODUCTION

During an earthquake the building may suffer damages at the point of weakness. The points of weakness are termed as the irregularity in the structure. The irregularity in the structure is due to mass variation, strength variation, and stiffness variation. The irregularity in mass, stiffness, and strength may be in plan or in elevation. To give better performance in an earthquake, the building should satisfy four main parameters that are simple and regular shape configuration and adequate lateral strength, stiffness and ductility. The structures with regular and simple configuration may undergo considerably much less damages during seismic excitation than the irregular structures. The irregular building does not perform well during an earthquake. Still, the irregular buildings are designed. This is because of their functional use and architectural appearances.

The building is classified as irregular due many reasons, like mass irregularity, stiffness irregularity and strength irregularity. As per is IS 1893 (Part 1):2002, the building is considered as irregular due to following type of irregularity:

1.1 Horizontal irregularity or plan irregularity

I. Torsion irregularity

A structure is considered torsion irregular when floor diaphragms are rigid in their own plan in with respect to the vertical structural elements that resist the lateral forces. When the maximum storey drift calculated with the eccentricity at one end of the building transverse to and axis is more than 1.2 times the average of the storey drift calculated at the two ends of the buildings.

II. Re-entrant corners

When the projections of the building beyond the re-entrant corner are greater than 15% of its plan dimension in the given dimensions, the buildings are considered as irregular of re entrant corners.

III. **Diaphragm Discontinuity**

The building is considered as diaphragm irregular when the cut out or openings in the diaphragm is greater than 50% of the gross enclosed diaphragm area or changes in the diaphragm stiffness is more than 50% from one storey to the next storey.

IV. **Out of plane offsets**

When the lateral resisting element is out of plane of vertical elements, the building is considered as out of plane offsets.

V. **Non parallel systems**

The building comes under this category of irregularity when the vertical members resisting the lateral force are not parallel to or symmetric about the major orthogal axes or the lateral resisting members.

1.2 Vertical irregular buildings

I. **Stiffness irregularity**

a) **Soft storey**

A soft storey is considered when the lateral strength of the storey under consideration is less than 70% of that in the storey above the considered storey or less than 80% of the average lateral stiffness of the three storeys above the considered storey.

b) **Extreme soft storey**

A storey is considered as extreme soft storey when the lateral stiffness is less than 60% of the average stiffness of the three storeys above the considered storey.

II. **Mass irregularity**

Mass irregularity is considered if any storey in the building having seismic weight more than 200% of the adjacent storey. In case of roofs, irregularity is not considered.

III. **Vertical geometric irregularity**

Vertical geometrical irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150% of that in its adjacent storey.

IV. **In plane discontinuity in vertical elements resisting lateral force**

An in plane offset of the lateral force resisting elements greater than the length of those elements.

V. **Discontinuity in capacity – weak storey**

A weak storey is one in which the storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

1.3 Dynamic characteristics of the buildings

The building vibrates during an earthquake excitation. The vibrations cause inertia force in the building. The inertia force developed is depends on the intensity and duration of oscillation. These characteristics are called dynamic characteristics of the buildings. The main parameters of the buildings are modes of vibration and damping. A mode of vibration of a building is defined by Deformed Shape and Natural Time Period in which building vibrates.

1.3.1 Natural Time Period

The time taken by the building to complete one cycle of oscillation is called natural time period. It is an important property of a building controlled by mass m and stiffness k. the natural time period is related by

$$T_n = 2\pi(m/k)^{(1/2)}$$

The building with larger mass and flexible has larger natural time period than the light and stiff building. When building vibrates, there is a deformed shape of oscillation.

The fundamental time period prescribed by IS 1893 (Part I):2002 is,

$$T_a = 0.085\ h^{0.75}$$

The reciprocal of natural time period (T_n) of a building is known as natural frequency (f_n). The building offers least resistance when it vibrate at its natural time period (or natural frequency). It large displacement when vibrate at its natural frequency than other frequencies. Generally, natural time periods of 1 to 20 the buildings are in the range of 0.05-2 sec. most of the engineers considered natural time period and not natural frequency. When the frequency of oscillation of the ground is at or near any of the natural frequency of the building, resonance occurs. There is no surety that the natural frequency of the vibrating building is same as the shaking of the ground for sustained duration. Usually the response of the building increased; when the ground shaking has frequencies near to natural frequency of the building. It is noticed during the 1985 Mexico City earthquake, where the buildings having small natural time period collapsed. While those have natural time periods outside the range performed normally. This happens almost for uniform soil strata.

1.3.2 Fundamental natural time period of the buildings

Building has many frequencies, at which it vibrates and offers least resistant to ground shaking induced during an earthquake or wind and other internal elements like vibrating machines fixed on it. The mode of oscillation with largest time period (least natural frequency) is called fundamental mode of oscillation. And the corresponding natural time period is called fundamental time period and the corresponding natural frequency is called fundamental time period. The natural modes of a building are infinitely large in number. But for designing the building the numbers of modes of oscillation are finite.

Usually, the natural time period of the buildings depends on the mass and stiffness of the buildings. Following are the parameters which affect the building's natural time period:

1. The natural time periods of the structure decreases with increase in stiffness.
2. The natural time period of the structure reduces with increase in mass.
3. The height of the buildings also affect the time periods. Taller building have larger natural time period than the smaller one.
4. Flexible buildings have larger natural time period as compared to stiff buildings.
5. The in filled wall also affect natural time period of the buildings.

1.3.3 Mode shape

Mode shape of vibration is depends on the natural time period of the buildings. The deformed shape of the building, when it vibrates at or near the natural time period is known as the mode shape. For every building there are number of mode shapes associated with natural time periods of the buildings. Corresponding to fundamental time period, the mode shape is called as fundamental mode shape. It can be first, second and third depending upon the natural time period of different modes.

Factors affecting mode shapes

1. It depends on the stiffness of the buildings.
2. It depends on mass distribution of buildings.
3. Support conditions of the buildings.
4. Height of the buildings.
5. Infill walls.

1.3.4 Damping

When the building vibrates by ground shaking due to seismic excitation comes back to rest after some time. This phenomenon occurs due to dissipation of energy into other forms (in the form of heat and sound). This conversion of earthquake energy in to some other form is called as damping.

CHEPTER 2

OBJECTIVE, SCOPE AND METHODLOGY OF THE STUDY

2.1 Objective of the study

The objective of this study is to understanding the behavior of the building under structural irregularity. The structural irregularities are mass irregularity, stiffness irregularity plan irregularity and other type of irregularities. Here vertical mass irregularity is considered.

As per IS 1893:2002 (Part 1) the expression given to calculate the fundamental time period is a function of height of the building and plan dimension of the building. But in actual the fundamental time period also affected due to change in irregularity and change in mass variation at different floor level.

In this project work the objective is to show that the natural time period is also a function of change in mass at different floor level and distribution of mass.

Following are the objectives of this study:

1. To study the mass irregularities of the building.
2. To find out the dynamics characteristics of the frame with or without mass variation.

The dynamic characteristics are: time period, frequencies and mode shapes.

2.2 Scope of the study

The scope of this work is limited to find out the change in natural frequencies, time period and mode shapes of the building frame model with or without mass variation at different floor level. The location of mass to which level is best suited is studied.

2.3 Steps of the study

Following are the steps utilized in this project work:

1) **Literature review**

Literature survey was carried out to understand the irregularity type of the structure. The response of the structure during earthquake is studied. The change in the fundamental time period of the building due to irregularity of the building is studied. The failure of the irregular building due to past earthquake was studied.

2) **Mathematical formulation**

Mathematical formulation is carried out to find the natural frequency of the Aluminium frame modal. The change in the natural frequency of the frame modal is calculated due to change in the mass at different floor level. The details about the addition of mass are presented in analytical calculation. Following cases are considered

a) Without mass variation.
b) With additional mass on 1st floor.
c) With additional mass on 2nd floor.
d) With additional mass on 3rd floor.
e) With additional mass on 1st floor and on 2nd floor.
f) With additional mass on 2nd floor and on 3rd floor.

3) **Data collection**

The data collection was mainly based on the tests conducted on the prepared model. The data collected was frequency of the signals, dimensions of the frame modal, and other relevant information related to the experimental arrangements.

4) **Experimental arrangements and model testing**

All the equipments required during the course of study were arranged. Test was conducted on the Aluminum frame modal to find out the floor displacement and corresponding natural frequency. The model is arranged and fitted to the shake table to perform the testing procedure. The frame modal is available in the Delhi Technological University, Civil Engineering Department, and Earthquake Engineering Laboratory. Testing on frame modal is carried out to study the pattern of floor displacement at guiding frequency and the fundamental frequency was found out at resonant.

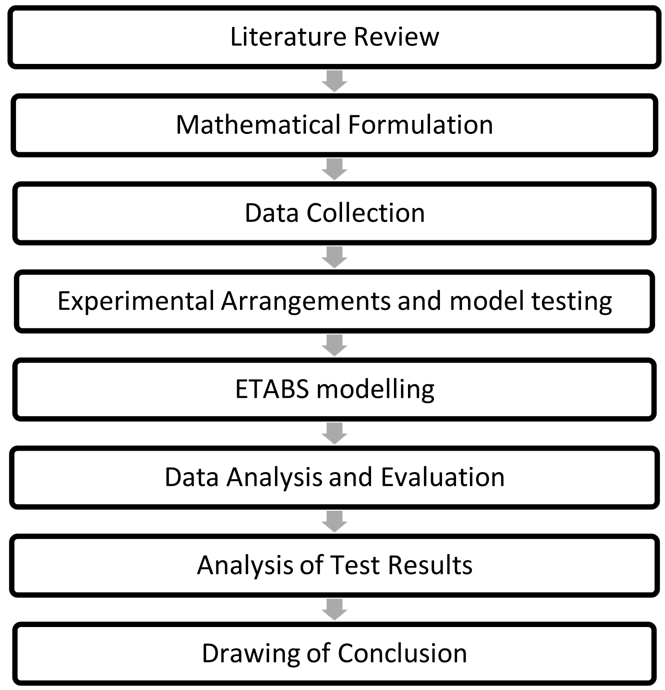
5) **ETABS modeling**

Frame is modeled in the commercial ETABS software to validate the result that is obtained from analytical and experimental procedure.

6) **Data analysis and evaluation**

The test results of the frame modal were compared with respect to calculate by the eigenvector analysis and the results were presented using table, pictures, and graphs.

2.4 Work flow chart



CHAPTER 3

LITERATURE REVIEW

The performance of the building structures during past year earthquakes give the idea that the plan irregularities are one of the most sever cause of damage during the seismic excitation. The plan irregularity can be due to irregular distribution of mass along the horizontal and vertical direction, stiffness in horizontal or vertical direction, and strength along the plan. Number of research was carried out in past years to understand the behavior of the irregular structure during past earthquake.

Past research studies on structure irregularities started in early 1980's with Tso and Sadek (1985) [1] who found out the change in ductility demand by performing inelastic seismic response of simple one storey mass eccentric frame modal and bi linear hysteric modal. The result of mathematical study showed that the time period played very important effect on the ductility demand after the elastic range of the building. The variation of 20 percent in the results obtained between Clough's and bi-linear modal. Irregularity in strength and stiffness are one of the major causes of failure during the seismic excitation. These irregularities are interdependent and to study the behavior of the building due to these irregularities on seismic response researchers like Tso and Bozorgnia (1986) [2] determined the inelastic seismic response of plan irregularity building models (as described in Table 3.1) with strength and stiffness eccentricity using curves proposed by Tso and Dempsey (1990) [3]. Result of the analytical study showed the effectiveness of the curves proposed by Tso and Dempsey (1990) [3] except for torsionally stiff buildings with low yield strength.

Table 3.1 details of different models taken by Tso and Sedek (1989) [1]

Serial No.	Model Name	Description
1	M_e	Model with mass eccentricity with all three resistant elements having equal yield deformation.
2	S_{e1}	Model with stiffness eccentricity with identical yield strength.
3	S_{e2}	Model with stiffness eccentricity with identical yield deformation.

Sarkar et al. (2010) [4] gave a new method of quantifying irregularity in vertical irregular structure frames, accounting for the dynamic characteristics (mass and stiffness). The following are the results are obtained:

a) Calculation of vertical mass irregularity suitable for stepped building, known as irregular index is proposed, accounting for the variation in mass and stiffness in the direction of the height of the structure.
b) An empirical formula was given to calculate the fundamental time period of the stepped building as a function of irregularity index.

Karavasilis et al. (2008) [5] has studied the inelastic seismic response of steel frame with mass irregularity at different floor level. The analysis of the test showed that the strength of beam column, the number of storey in the building and the location of the heavier mass influence the inelastic deformation demands, while the response does not much vary with the mass ratio.

Devesh et al. (2006) [6] studied on the increase in the drift demand in the building portion of set back structure and on the increase in the seismic demand for buildings with irregular distribution in mass, strength and the stiffness. The results showed that the more seismic demand was obtained for the combined stiffness and strength irregularity. It was obtained the behavior of the building is influenced by the type of the frame model.

Poonam et al. (2012) [7] studied the vertical irregular buildings behaves different one than the regular buildings. The study showed that any storey, either first or any other storey, should not be weak or soft storey than the storey above or below. It was obtained that the irregular distribution in mass also adds to the increased response of the buildings. The irregularity in mass is to be provided at appropriate floor and at appropriate location so that the response of the structure does not affect so much.

Moghadam and Aziminejad (2005) [8**]** analyzed PBD (Performance Based Design) of irregular buildings. The study obtained the response of single storey building (code design) with asymmetric configuration for optimizing mass, stiffness and strength center configurations corresponding to variation in levels of the plastic hinge formation. The authors adopted the concept of balanced center of location and center of rigidity given by Tso and Myslimaj (2003) to find out the best performance level of the structure. On the basis of analytical study it was obtained that the best location for the center of stiffness and center of rigidity depended upon the required performance level of the building and also on damage indices.

Humar and Wright (1977) [9] was take a multistory steel building frames with or without irregularity. He has found out that the storey drift greater at upper portion of setback. Again Aranda (1984) [10] continue this study and result is calculated with or without setback of the frame model. He found out that the ductility demand increases on upper setback.

Fernandez (1983) [11] calculated the elastic and inelastic seismic response of frames. He is considered multistory building frame. Study is carried out on frame with irregular distribution of stiffness and mass. He concluded that there was increases in storey drift due to decrease in stiffness. The structures perform well under seismic excitation due to regular variation in mass and stiffness. The better the regularity, the better will be performance of the buildings. The uneven or abrupt variation in mass affect the building time period and storey drift.

Moelhe (1984) [12] studied the seismic response of reinforced concrete structures with or without irregularities. For the study he has considered a nine storey building frames. The structure consists of wall. The irregularity is due to discontinuous of structural walls at different level of the storey. He has found out that the response of the reinforced building does not only depend on the irregularity but also on the distribution of irregularity along the height. Moehle and Alarcon (1986) [13] performed the experimental analysis to validate the analytical results. He considered a small prototype reinforced concrete building frame given a ground motion. The testing is done with help of shake table. The irregularity is created in the mode frame by discontinuous shear wall at the first floor level. They noted down the displacement of the top floor. He concluded that the ductility demand increases drastically due to abruptly change in irregularity in the building.

Barialoa and Brokken (1991) [14] found out the effect of stiffness and strength irregularity on seismic response of multistory structure frames. He has considered 8 storey building frame for analytical study. Three different time periods are considered for the analysis as low medium and high. They have considered weak and strong type of buildings. The base shear was 15 percent in weak storey of the total seismic weight of the building. The base shear for strong storey was 30 percent of total seismic weight of the building. The results were concluded. They have found that the natural time period of the building increases during the ground shaking and this was more predominant in case of weaker structures.

Ruiz and Diederich (1989) [15] determined analytical modeling on 5 and 12 storey building frame models with or without strength irregularities. The irregularity was introduced with or without infill walls. First of all they have considered first storey as weak storey. And then the infill walls are provided at the top of the model. In the third situation they have modeled infill walls ductile. They have found out that the performance of the building affected by time period of seisminc excitation with infill walls.

Shahrooz and Moehle (1900) [16] found out the performance of the building with vertical setbacks. They calculated dynamic characteristics of vertical setback building. For this they considered a 6 storey reinforced concrete building with setback at half of the storey level. The model was prepared. They obtained displacement with the experimental analysis. The displacement were obtained to be largest with more damage drastically reduction in the inertia forace at the level of setback. This change in displacement and lateral force with the height gives the idea about the damage due to fundamental mode of excitation. The ductility demand is increased with increase in setback at the level of storey height.

Nassar and Krawinkler (1991) [17] determined the dynamical characteristics of multistory buildings. They have considered 5, 10,20,30,40 storey building. The buildings were single degree of freedom system and multidgree freedom system. They have taken different natural time periods of the seismic excitation ranging from 0.217-2.051. they concluded that presence of weak storey will produced ductility and more overturning moments.

Esteva (1992) [18] determined the seismic response of the structure frames with strength irregularity. He performed non-linear analysis. The strength irregularity is created with first storey as weak storey. The study deals with the bilinear behavior of the building. He does not take into account the p-delta effect. He concluded that the dynamical characteristic of the building did not affected by the setback irregularity.

Wong and Tso (1994) [19] taken elastic response spectrum analysis to find out dynamic characteristics of the building. They have considered setback irregularity and it was observed that the structure with this irregularity had larger modal mass abruptly distribution of seismic load as compared to static codal procedure.

Duan and Chandler (1995) [20] determined analytical modeling of buildings. They have considered setback irregularity. Both static and modal spectrum analysis was carried out. The results showed that there was inaccurate data to find out the extent of damage in the building members at the level of setbacks.

Valmudsson and Nau (1997) [21] found out the dynamic characteristic of multistory buildings. They have considered vertical irregularity in the building. They have developed two dimensional shear beam building models with different number of storeys. They have considered 5, 10,and 20 storey. The irregularity is created in the modal by changing the mass, stiffness and strength. The result of analytical study shows that there is minor variation seismic response of the building with mass and stiffness irregularity. The storey drift were affected. By reducing the strength of about20 percent can approximately increase the ductility two times.

Al-Ali and Krawinkler (1998) [22] determined the irregularity of mass, strength and stiffness. They considered individual irregularity. In first case they have considered the mass irregularity. In second case they have considered the stiffness irregularity. In third case they have considered the strength irregularity. They have taken a 10 story building model to study the behavior of building. They obtained that when irregularity is considered as separately, the effect of strength irregularity is more compared to other two irregularities. The displacement of the top floor was maximum due to strength irregularity. The effect of mass irregularity on the floor displacement is much less than other irregularity. They have combined the two irregularities and the results shows that the impact of strength and stiffness on roof displacement was more severe to that of other irregularity combinations.

Kappos and Scott (1998) [23] compared the result of static and dynamic analysis for determining the seismic response of the reinforced concrete building frames. They have considered setback irregularity. After comparison of two results it was found out that the result of both the analysis was different. The dynamic analysis yields more than static analysis. In the mathematical formulation and analytical study they have considered other type of irregularities.

Manfliulo et al. (2002) [24] determined the dynamic characteristics of the multistory buildings. He has considered 5 and 9 storey building model. He has considered the mass irregularity, strength irregularity and stiffness irregularity. By the analytical study he has found that the variation in mass does not affect the plastic demands. The irregularity due to strength enhanced the seismic demand of the building. However the seismic demand not affected by irregular strength in columns. Finally he made conclusion that the parameter of storey strength given in EC8 and IBC codes was inadequate to predict strength irregularity due to strength variation.

Das and Nau (2003) [25] determined the impact of mass, stiffness and strength on inelastic seismic response of taller building with having large number of storeys. For analytical sturdy they have considered different storeys with number of storeys ranging from 5-20 were modeled as shown in fig. 3.1. The structural irregularity in these model was considered. The irregularities were mass irregularity, stiffness irregularity, strength irregularity and masonry infills. The modeling was done as per ACI 1999 and UBC 97. The mode shape and fundamental time period was calculated. The result of regular and irregular building was compared. The code gives the increase in storey drift of 2 percent. But the result was just abruptly showing the increase in strorey drift. The storey drift changes unevenly due the combination of irregularity. They have concluded that mass irregularity affect less damage of the building than the other irregularities.

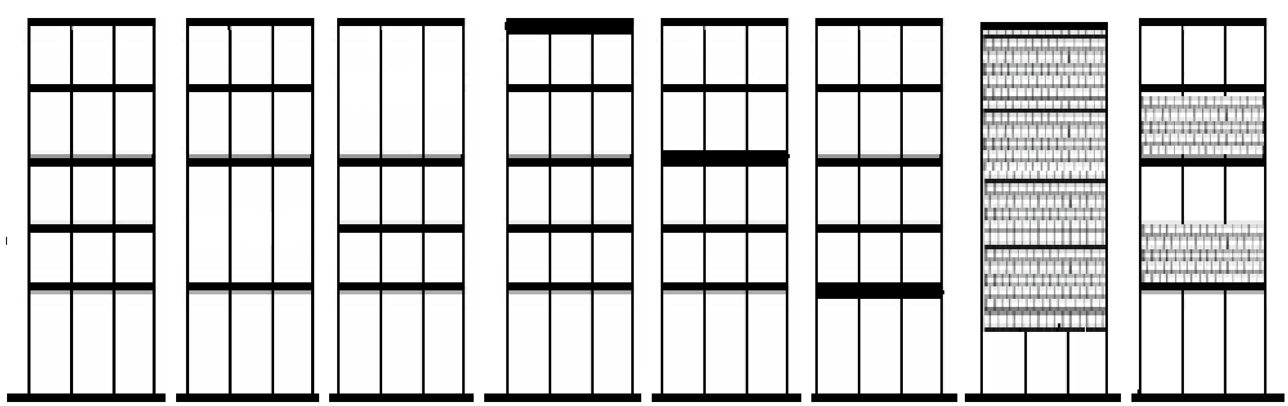


Fig. 3.1 different type of vertically irregular building models as per Das and Nau (2003)

Chintanpakde and Chopra (2004) [26] determined the effects of combination of stiffness and strength irregularities. They had considered 12 storey frame model. The theory of strong column and weak beam theory were applied. The irregularity was created at different level of floor. Time history analysis was carried out. It was obtained that when the stiffness and strength irregularity apply in combination, the effect on response of the building was more severe. For deformation, the irregularity must be present at lower level of storey.

Tremblay and Poncet (2005) [27] determined the dynamic characteristic of the building frames with vertical mass irregularity. The analysis was done as per NBCC guidelines. The static and dynamic analysis were carried out it was observed that the codes provision were ineffective in mass irregularity of buildings.

Tremblay and Poncet (2005) [27] determined the dynamic parameters of the building modeled frames. The mass irregularity was considered. The analysis was done as per NBCC code. Ayidin (2007) [28] determined the seismic response of building with mass variation. He was consider the ELF procedure to analyse the frame. The buildings were modeled as 5 to 20 storey buildings. The mass irregularity was introduced by variation in mass of one storey by keeping the mass of other storey constant. The result was obtained and concluded. It was found that the code processors ineffective in calculating the seismic response. The result from the code was over estimated.

CHAPTER 4

MATHEMATICAL FORMULATION AND ANALYSIS

4.1 Introduction

Modal analysis is the study of the dynamic properties of structures under vibration excitation in structural engineering, modal analysis uses the overall mass and stiffness of a structure to find the various periods at which it will naturally resonate. A normal mode of an oscillating system is pattern of motion in which al parts of the system move sinusoidally with the same frequency and with a fixed phase relation. Eigenvector analysis determines the undamped free vibration mode shapes and frequencies of the system. These natural modes provide an excellent insight into the behavior of the structure. Ritz vector analysis seeks to find modes that are excited by a particular loading. Ritz vectors can provide a better basis than do eigenvectors when used for response-spectrum or time- history analyses that are based on modal superposition. Thus, modal analysis is done by following methods,

1. Eigenvector analysis.
2. Ritzs vector analysis

4.2 Eigen value analysis

The aim of this eigenvector analysis is to find out the natural frequency of the Aluminium frame model. The frame in figure-1 can be approximately modeled as a three degree of freedom shear beam as shown in Figure 4.1. Following is the free body diagram shown in Fig. 4.2.

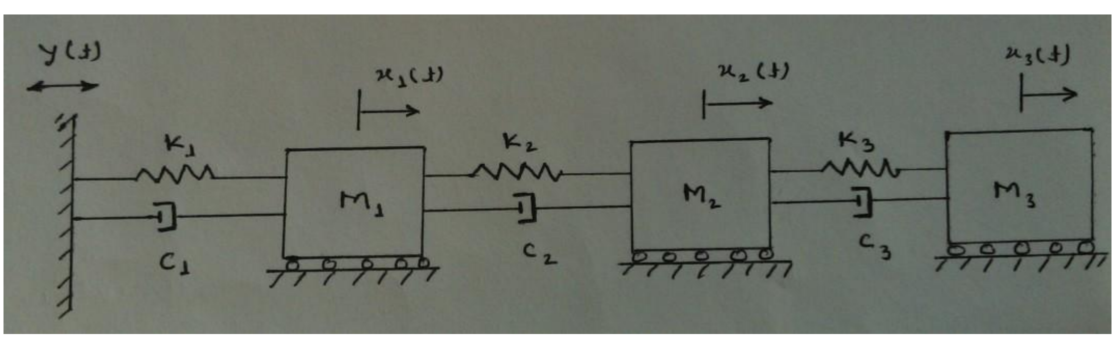


Fig. 4.1 three degree of freedom beam model of frame

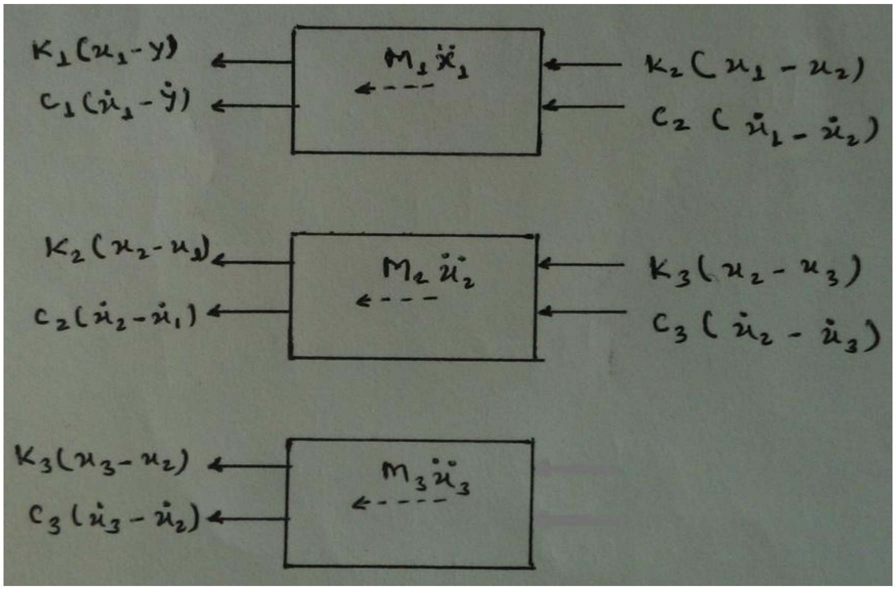


Fig. 4.2

Displacement imposed at the base of the structure = y

Expressing the floor displacements relative to the base motion:

$$x_{r1} = x_1 - y$$
$$x_{r2} = x_2 - y$$
$$x_{r3} = x_3 - y$$

_____ (1)

Thus, we can set up the equation of motion for the total displacement of the three masses as follows:

$$m_1\ddot{x}_{r1} + c_1\dot{x}_{r1} + c_2(\dot{x}_{r1} - \dot{x}_{r2}) + k_1x_{r1} + k_2(x_{r1} - x_{r2}) = -m_1\ddot{y}$$

$$m_2\ddot{x}_{r2} + c_2(\dot{x}_{r2} - \dot{x}_{r1}) + c_3(\dot{x}_{r2} - \dot{x}_{r3}) + k_2(x_{r2} - x_{r1}) + k_3(x_{r2} - x_{r3}) = -m_2\ddot{y}$$

$$m_3\ddot{x}_{r3} + c_3(\dot{x}_{r3} - \dot{x}_{r2}) + k_3(x_{r3} - x_{r2}) = -m_3\ddot{y}$$

_____ (2)

Rewriting this in matrix form:

$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \begin{pmatrix} \ddot{x}_{r1} \\ \ddot{x}_{r2} \\ \ddot{x}_{r3} \end{pmatrix} + \begin{bmatrix} c_1 + c_2 & -c_2 & 0 \\ -c_2 & c_2 + c_3 & -c_3 \\ 0 & -c_3 & c_3 \end{bmatrix} \begin{pmatrix} \dot{x}_{r1} \\ \dot{x}_{r2} \\ \dot{x}_{r3} \end{pmatrix} + \begin{bmatrix} k_1 + k_2 & -k_2 & 0 \\ -k_2 & k_2 + k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix} \begin{pmatrix} x_{r1} \\ x_{r2} \\ x_{r3} \end{pmatrix}$$
$$= (-) \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix} y$$

_____ (3)

Thus,

$$[m]\{\ddot{x}_r\} + [c]\{\dot{x}_r\} + [k]\{x_r\} = -[m]\{\Gamma\}y$$

_____ (4)

Initial condition:

$x_r = 0 \; and \; \dot{x}_r = 0 \;\; at \; t = 0$

Here, {Γ} is 3×1 vector of ones.

Eigenvector analysis

Eigenvector analysis determines the undamped free-vibration mode shapes and frequencies of the system. These natural modes provide an excellent insight into the behavior of the structure.

$$[m]\{\ddot{x}_r\} + [k]\{x_r\} = 0$$

_____ (5)

When floors of a frame reach there extreme displacement at the same time and pass through the equilibrium position at the same time, then each characteristic defected shape is called as natural mode of vibration of an MDF system.

During the natural mode of vibration of an MDF system there is a point of zero displacement tha does not move at all. The point of zero displacement is called as node. As the number of mode increases, number of node increases accordingly.

In free vibration, the system will oscillate in a steady-state harmonic fashion-

$$x = (a \sin wt + b \cos wt)$$

Where a and b are constants.

Thus,

$$\ddot{x} = -w^2(a \sin wt + b \cos wt)$$

Such that,

$$\ddot{x} = -w^2 x$$

_____ (6)

Substituting equation (6) in equation (5), we will get:

$$(-mw^2 + k)x = 0$$

This equation is called as matrix eigenvalue problem.

Trivial solution of above equation is x=0, means the system is at rest.

For non- trivial solution,

$$|-mw^2 + k| = 0$$

_____ (7)

Where, m and k have their usual meaning, i.e., mass matrix and stiffness matrix respectively. Rewriting in that form:

$$\left| -\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} w^2 + \begin{bmatrix} k_1 + k_2 & -k_2 & 0 \\ -k_2 & k_2 + k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix} \right| = 0$$

_____ (8)

Since all the columns at each storey have same dimensions, hence stiffness will be same for all floors. Hence assuming, $k_1 = k_2 = k_3 = k$

Then by solving equation (8), we will get

$$-m_1 m_2 m_3 (w^2)^3 + k(m_1 m_2 + 2m_2 m_3 + 2m_3 m_1)(w^2)^2 - k^2(m_1 + 2m_2 + 3m_3)w^2 + k^3 = 0$$

______ (9)

This is a cubic equation in the form of: $a\,x^3 + b\,x^2 + c\,x + d = 0$

Where, w^2 is the variable and,

$a = -m_1 m_2 m_3$

$b = k(m_1 m_2 + 2m_2 m_3 + 2m_3 m_1)$

$c = -k^2(m_1 + 2m_2 + 3m_3)$

$d = k^3$

4.3 Dimension Parameters:

Plate length (x-direction) = 300 mm

Plate width (y-direction) = 150 mm

Plate thickness (z-direction) = 12.44 mm

Column thickness (x-direction) = 2.92 mm

Column width (y-direction) = 24.95 mm

Column height of each storey (z-direction) = 400 mm

Moment of inertia of column area about y-axis, $\mathrm{I}_{yy} = \frac{bd^3}{12}$

(Here, b = 24.95 mm and d = 2.92 mm)

Stiffness of each column, $k_c = \frac{12EI}{l^3}$

(Here, l = 400 mm)

Modulus of Elasticity of aluminium, $E = 0.69 \times 10^5\ N/mm^2$

Thus, stiffness of each storey,

$$k = 4k_c = 4 \times \frac{12 \times (0.69 \times 10^5) \times \frac{24.95 \times 2.92^3}{12}}{400^3}$$

$$= 2.67885 \ N/mm$$

(Or, 2678.85 N/m)

4.4 Weight Parameters:

Mass of each slab = 1.54 kg

Mass of column 1 of all three storey = 0.2338 kg

Mass of column 2 of all three storey = 0.2341 kg

Mass of column 3 of all three storey = 0.2335 kg

Mass of column 4 of all three storey = 0.2322 kg

Hence, mass of all columns at each storey level = $\frac{0.2338 + 0.2341 + 0.2335 + 0.2322}{3}$

= 0.3112 kg

4.5 Calculation natural frequency

4.5.1 Case-1: without mass variation on floor levels.

$m_1 = 1.54 + 0.3112 = 1.8512 \ kg$

$m_2 = 1.54 + 0.3112 = 1.8512 \ kg$

$m_3 = 1.54 + \frac{0.3112}{2} = 1.6956 \ kg$

And $k = 2678.85$ N/m

On substituting these values in equation (9), we will get,

$$w_1{}^2 = \ \ 300.25 \ \ (rad/s)^2$$

$$w_2{}^2 = 2321.39 \ (rad/s)^2$$

$$w_3{}^2 = 4746.58 \ (rad/s)^2$$

As, $f = \frac{w}{2\pi}$

Hence,

$$f_1 = \ \ 2.7578 \ Hz$$

$$f_2 = \ \ 7.6683 \ Hz$$

$$f_3 = 10.9651 \ Hz$$

4.5.2 Case-2: with mass variation on floor levels.

4.5.2.1 Additional mass on slab of 1st floor = 2.0 kg

Then,

$m_1 = 1.54 + 0.3112 + 2.0 = 3.8512 \ kg$

$m_2 = 1.54 + 0.3112 \ = 1.8512 \ kg$

$m_3 = 1.54 + \dfrac{0.3112}{2} = 1.6956 \ kg$

And $k = 2678.85$ N/m

On substituting these values in equation (9), we will get,

$$w_1{}^2 = \ \ 263.922 \ \ (rad/s)^2$$

$$w_2{}^2 = 1452.276 \ (rad/s)^2$$

$$w_3{}^2 = 4149.045 \ (rad/s)^2$$

As, $f = \frac{w}{2\pi}$

Hence,

$$f_1 = 2.590\ Hz$$

$$f_2 = 6.070\ Hz$$

$$f_3 = 10.25\ Hz$$

4.5.2.2 Additional mass on 2^nd floor = 2 kg

Then,

$m_1 = 1.54 + 0.3112 = 1.8512\ kg$

$m_2 = 1.54 + 0.3112 + 2 = 3.8512\ kg$

$m_3 = 1.54 + \dfrac{0.3112}{2} = 1.6956\ kg$

And $k = 2678.85$ N/m

On substituting these values in equation (9), we will get,

$${w_1}^2 = 212.289\ (rad/s)^2$$

$${w_2}^2 = 2120.871\ (rad/s)^2$$

$${w_3}^2 = 3532.082\ (rad/s)^2$$

As, $f = \frac{w}{2\pi}$

Hence,

$$f_1 = 2.32\ Hz$$

$$f_2 = 7.34\ Hz$$

$$f_3 = 9.46\ Hz$$

4.5.2.3Additional mass on floor 3 = 2 kg

Then,

$m_1 = 1.54 + 0.3112 = 3.8512\ kg$

$m_2 = 1.54 + 0.3112\ = 1.8512\ kg$

$m_3 = 1.54 + \frac{0.3112}{2} + 2 = 1.6956\ kg$

And $k = 2678.85$ N/m

On substituting these values in equation (9), we will get,

$$w_1{}^2 = \ 183.617\ \ (rad/s)^2$$

$$w_2{}^2 = 1842.25\ (rad/s)^2$$

$$w_3{}^2 = 4487.364\ (rad/s)^2$$

As, $f = \frac{w}{2\pi}$

Hence,

$$f_1 = 2.157\ Hz$$

$$f_2 = 6.83\ Hz$$

$$f_3 = 10.66\ Hz$$

4.5.2.4 Additional mass on 1st floor = 2 kg and on 2nd floor = 1 kg

Then,

$m_1 = 1.54 + 0.3112 + 2.0 = 3.8512\ kg$

$m_2 = 1.54 + 0.3112 + 1\ = 2.8512\ kg$

$m_3 = 1.54 + \frac{0.3112}{2} = 1.6956\ kg$

And $k = 2678.85$ N/m

On substituting these values in equation (9), we will get,

$$w_1{}^2 = 224.17 \ (rad/s)^2$$

$$w_2{}^2 = 1450.51 \ (rad/s)^2$$

$$w_3{}^2 = 3175.12 \ (rad/s)^2$$

As, $f = \frac{w}{2\pi}$

Hence,

$$f_1 = 2.380 \ Hz$$

$$f_2 = 6.060 \ Hz$$

$$f_3 = 8.97 \ Hz$$

4.5.2.5 Additional mass on 2nd floor =2 kg and on 3rd floor =1 kg

Then,

$m_1 = 1.54 + 0.3112 = 1.8512 \ kg$

$m_2 = 1.54 + 0.3112 + 3 = 3.8512 \ kg$

$m_3 = 1.54 + \frac{0.3112}{2} + 1 = 2.6956 \ kg$

And $k = 2678.85$ N/m

On substituting these values in equation (9), we will get,

$$w_1{}^2 = 175.80 \ (rad/s)^2$$

$$w_2{}^2 = 1645.76 \ (rad/s)^2$$

$$w_3{}^2 = 3456.98 \ (rad/s)^2$$

As, $f = \frac{w}{2\pi}$

Hence,

$$f_1 = 2.110\ Hz$$

$$f_2 = 6.450\ Hz$$

$$f_3 = 9.36\ Hz$$

4.6 fundamental time period as per IS 1893(Part I):2002

$$T_a = 0.085\ h^{0.75}$$

$$= 0.085\ (1.2)^{0.75}$$

$$= 0.097 \text{ sec.}$$

CHAPTER 5

EXPERIMENTAL PROGRAM

5.1 Introduction

Aluminium frame model is used for the experimental analysis. The frame is considered three storey shear building model. Model available in Delhi Technological University, Earthquake Laboratory is used. The test is conducted to find out the mode shape and frequencies of the model. Testing was done with symmetric frame. Then apply additional mass at different floor level to see the behavior of the building model due to this mass irregularity. The variation in mass at different floor level is done as described in analytical procedure. The dimension parameter is same as described in analytical procedure. Fig. 5.1 and 5.2 show the details of the frame model.

Following cases are adopted while carrying out the experimental analysis:

Case 1: Without mass variation (there is additional mass at different floor level

Case 2: With mass variation (additional mass is applied at different floor level. addition of mass at different floor level is as follows:

a) With additional mass on 1st floor =2 kg
b) With additional mass on 2nd floor = 2 kg
c) With additional mass on 3rd floor = 2 kg
d) With additional mass on 1st floor = 2 kg and on 2nd floor = 1 kg
e) With additional mass on 2nd floor = 2 kg and on 3rd floor = 1 kg

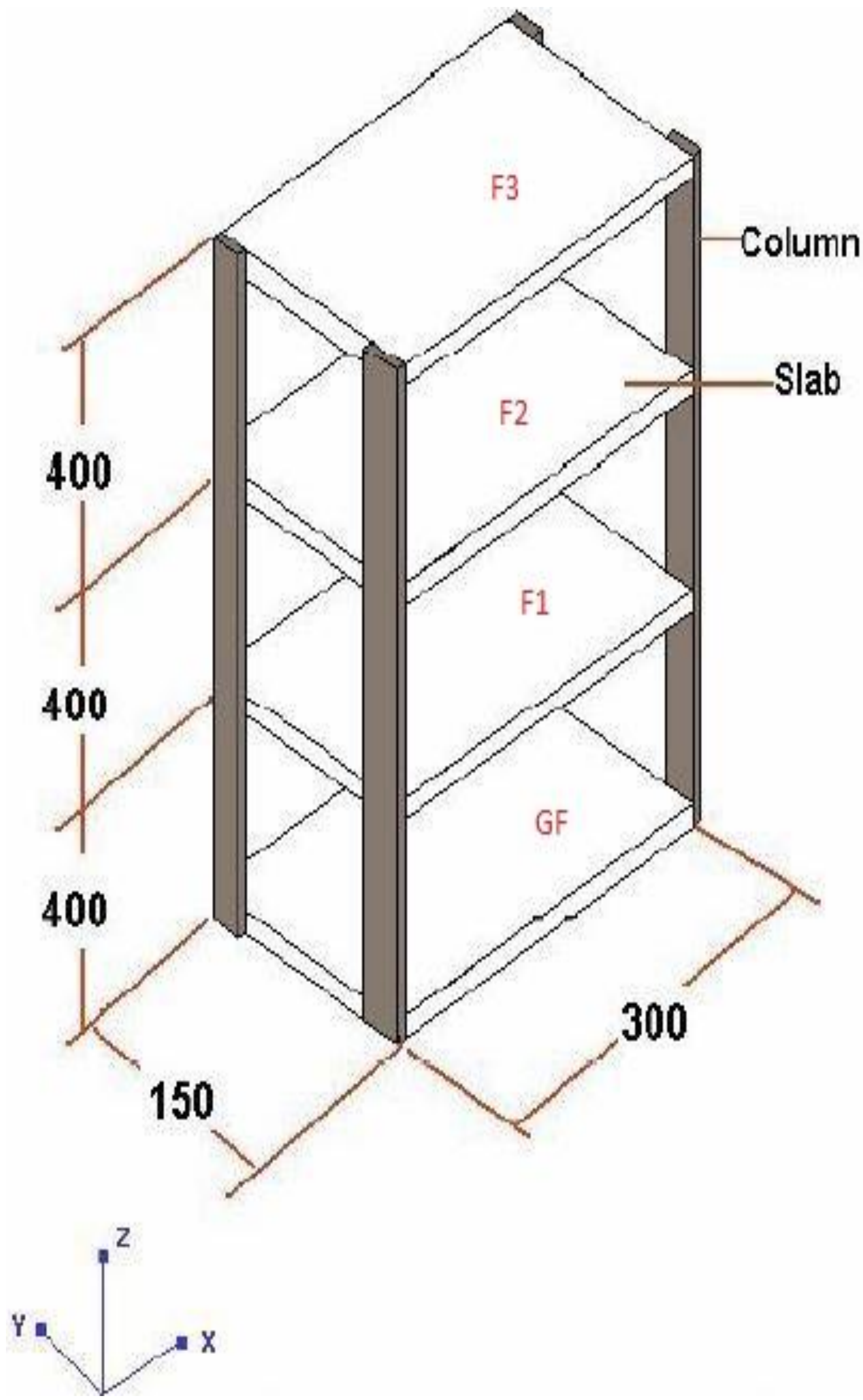


Fig. 5.1 experimental model (NPEEE)

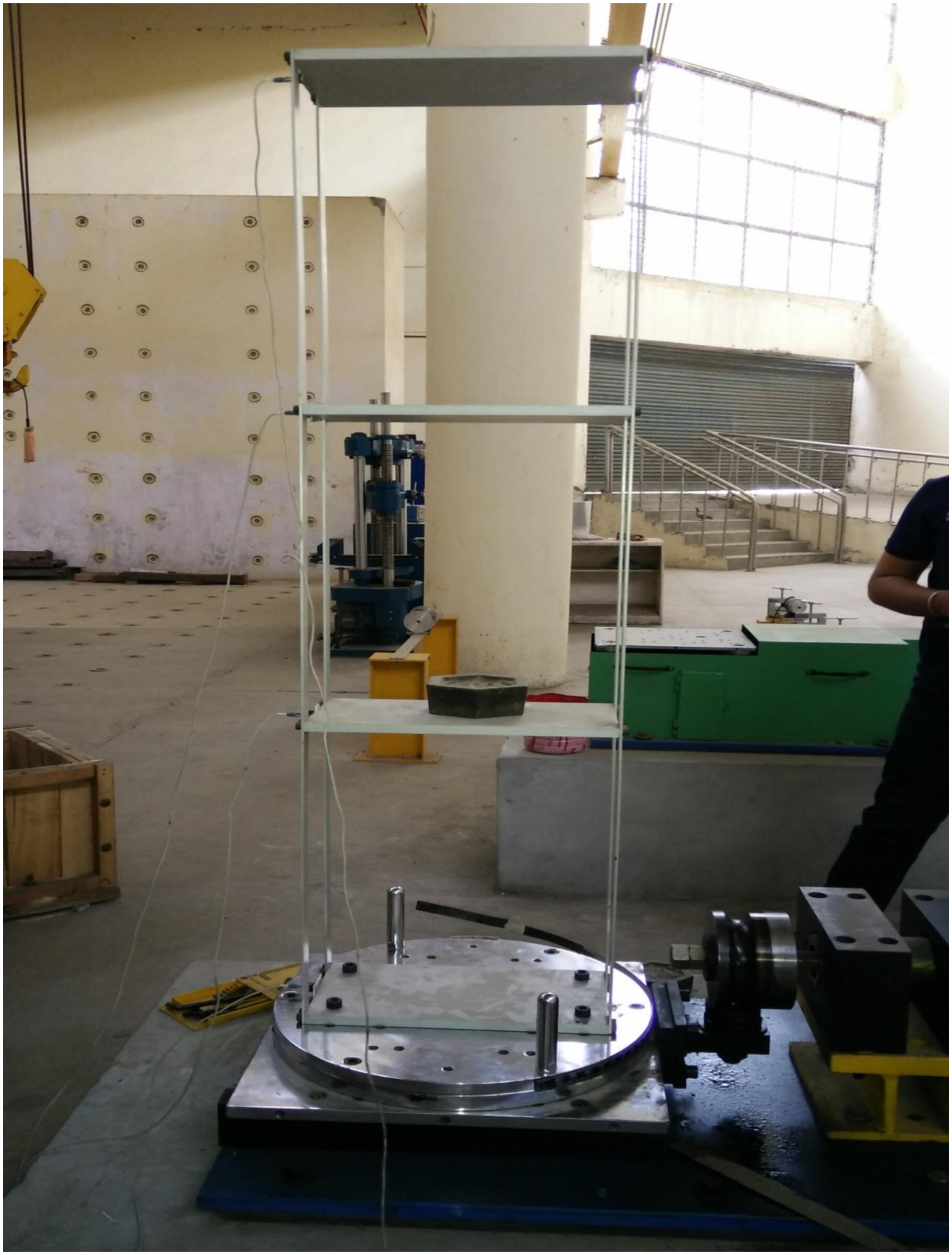


Fig. 5.2 experimental model

5.2 Equipment used in the experimental analysis

The equipments used in the experimental analysis are given in the table 5.1

Table 5.1 details of instruments to be used in the experimental study.

No.	Equipments	Quantity
1	Oscilloscope	1
2	Accelerometers	3
3	Display unit	1
4	Transducers conditioning amplifiers	1
5	Shake table	1

1) Oscilloscope

OROS 3-Series/NVGate analyzer system type OR36 software analysis was used to measure the free vibration for the modal. It can be used for both free vibration as well as forced vibration study. This has the channels to connect the cables for analyzing both input and output signals. The FFT analyzer is shown in Fig. 5.3.

Fig. 5.3 oscilloscope

2) Accelerometer

Accelerometer combines high sensitivity, low and small physical dimensions making them ideally suited for modal analysis. It can be easily fitted to different test objects using section mounting clips. The accelerometer which is used in the present experimental analysis of free vibration test is presented in fig. 5.4.

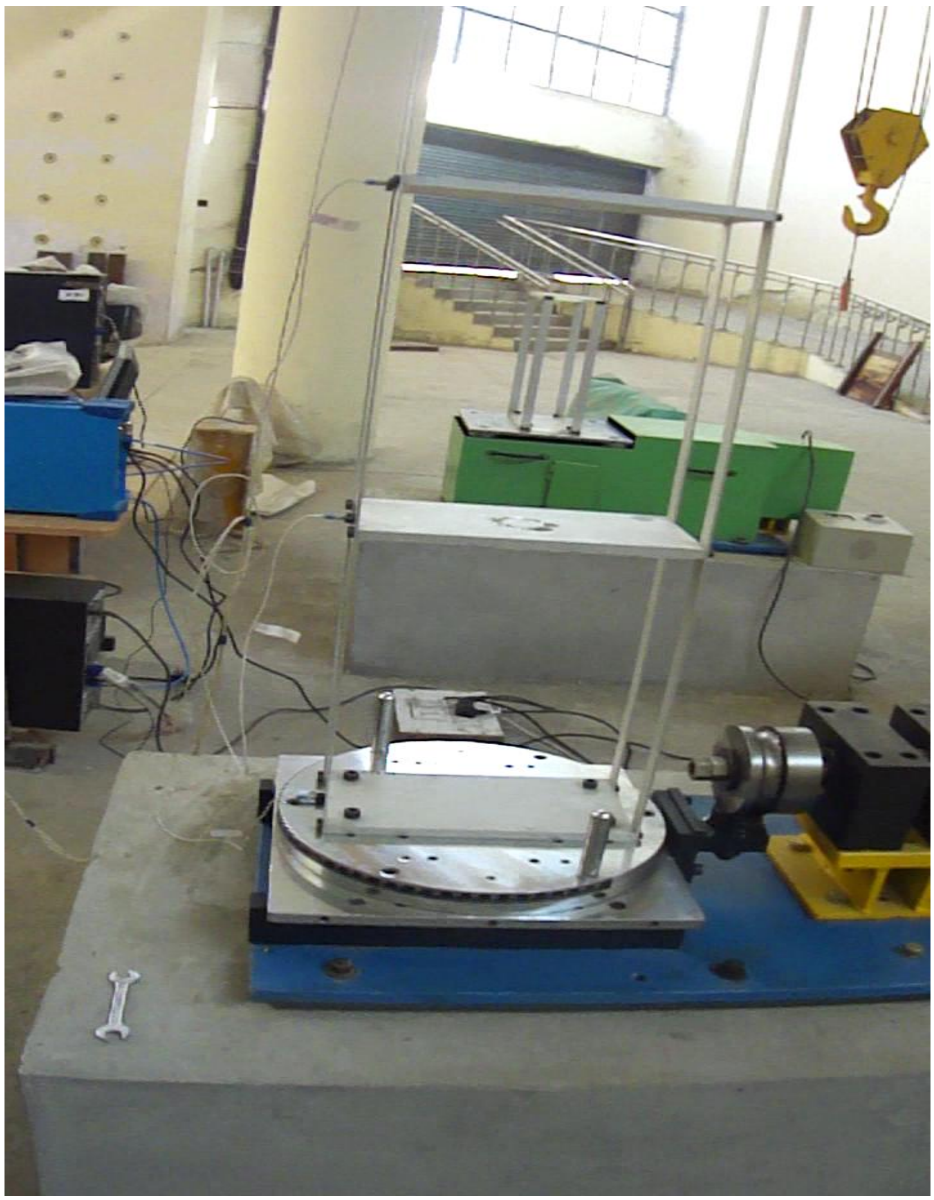


Fig. 5.4

3) Display unit

This is mainly in the form of PC. When the excitation occurs to the structure, the signals transfer to the portable PUSLE and after conversion this comes in graphical form through the software and display on the screen of PC. Mainly the data includes graphs of force Vs time, frequency vs. time, frequency vs displacement, frequency vs. acceleration. The display unit is show below in Fig. 5.5.

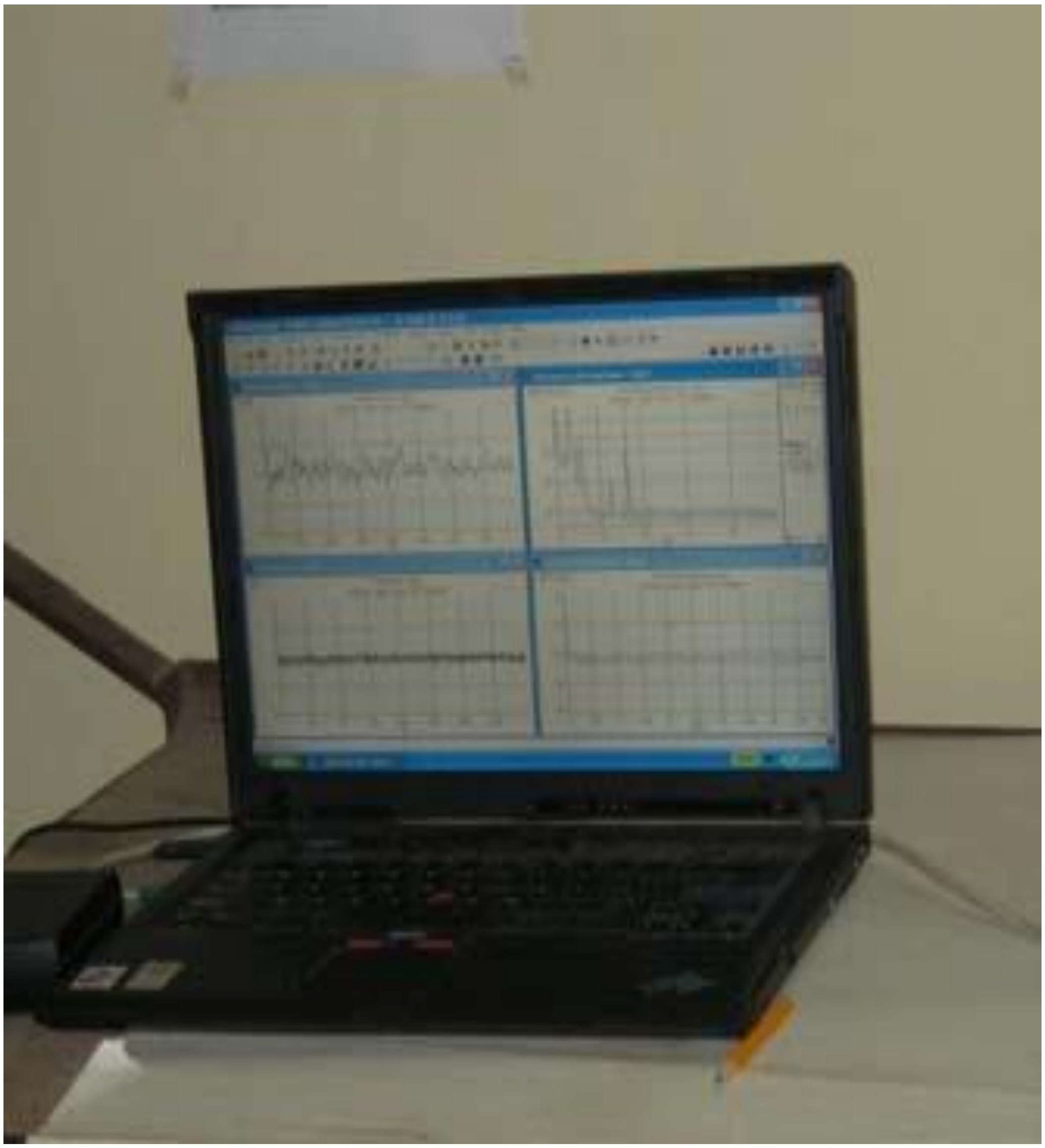


Fig. 5.5 display unit

3) **Details of the sensors used**

Table 5.2 details of sensors

No.	Sensor	Sensitivity m(V)/g
1	1	91.72
2	2	92.72
3	3	96.02

5.3 Preliminary measurements

a) The geometric and material properties of the testing system are collected. The particular related to the experiment is given in table

b) Taking the mass and stiffness properties of the model, eigen value analysis is carried out to calculate the natural frequencies.

c) The data related to the sensors. The sensitivities of sensors are noted down.

d) The initial displacement of the base motion is noted down by running the electric motor a particular frequency.

e) This base motion displacement should be kept constant.

f) The experimental setup is so arranged so that the displacement in the x direction is obtained. The experimental setup is shown in figure

g) The frame is arranged into free vibration by giving some initial displacement. This could be done by slightly pulling the frame at about at the top of the slab and releasing it. The free vibration decay on the oscilloscope is observed and the reading is noted down. The damping ratio of the frame is calculated by logarithmic decrement. It is assumed that the damping ratio is obtained should be remain constant for all the modes.

h) Now the frame is set to run and the frequency of the motor changed. By gradually increasing the frequency of the motor, floor displacement is recorded. When the resonance occurs the floor displacement is drastically changes. At the resonant condition the floor displacement is very high. Note that the resonance frequency should not be missed.

5.4 Testing and observations

5.4.1 Damping Ratio of aluminum frame model

Damping ratio is calculated by logarithmic decrement method. The free vibration observations are presented in table 5.2.

Table 5.2 observation reading of free vibration to find out damping ratio

No	Peak value	Damping ratio %
1	0.00123	
2	0.001219	0.142974
3	0.00115	0.927379
4	0.001133	0.237029
5	0.001106	0.383867
6	0.001146	-0.56544
7	0.001105	0.579837
8	0.001049	0.827734
9	0.00104	0.137138
10	0.001007	0.513197
11	0.001036	-0.54187
12	0.000949	1.396007
13	0.000939	0.168598
14	0.000919	0.342651
15	0.000902	0.305993
16	0.00089	0.209698
17	0.000864	0.468351
18	0.000843	0.395437
19	0.000869	-0.49095
20	0.000845	0.455105
21	0.000805	0.764276
22	0.000797	0.162952
23	0.000814	-0.34577
24	0.000776	0.768802
25	0.000786	-0.19976
26	0.000741	0.927946
27	0.000702	0.86707
28	0.000676	0.605454
29	0.00071	-0.79255
30	0.000697	0.289463
31	0.000638	1.426794
32	0.000636	0.052506
33	0.00062	0.385296
34	0.000612	0.203998
35	0.000587	0.671483
36	0.000592	-0.12152

Logarithmic decrement $\delta = \ln (y1/y2)$

Where y1 and y2 two consecutive peak values

Damping ratio $\xi = (\delta/(2\pi))*100\%$

Taking average by leaving first and last value:

Then, damping ratio $\xi = 0.35$ %

5.4.2 Determination of natural frequencies of frame

5.4.2.1 Without mass variation (there is additional mass at different floor level

In this case the frame is analyzed with no additional mass on the different floor level. The frame is put on the shaking table and fixed along the X direction. Then motor is run at a particular frequency. The floor displacement corresponding to the running frequency is measured from the display unit. Then the frequency of the motor keep on changing gradually and the floor displacement is recorded. During the changing the frequency it is kept in mind that the frequency at resonance should not missed. The record of frequency vs. floor displacement is presented in table 5.3 and the frequency vs. floor displacement graph is plotted in fig. 5.6. F1, F2 and F3 indicate floor 1, floor 2 and floor 3 respectively.

Table 5.3 Floor displacement corresponding to guiding frequency without mass variation

Frequency Hz	Floor Displacement (μm)		
	F1	F2	F3
1.2	583	836	1387
1.5	937	1183	1683
1.8	1034	1375	1876
2	1375	1675	2987
2.2	1694	1924	3876
2.5	2188	2965	4978
2.6	4638	8828	8945
2.7	6493	11748	40375
2.9	1765	3356	17843
3.2	1432	2943	5340
3.4	482	1843	3865
3.6	173	976	2786
3.8	93	914	1654

4	99	837	1786
4.2	219	830	1861
4.4	392	736	1564
4.6	440	604	1689
4.8	582	584	1643
5	579	421	1543
5.2	643	314	1458
5.6	832	221	1264
5.8	893	201	1498
6	953	196	1578
6.2	1193	121	1594
6.4	1286	65	1675
6.6	1738	189	1987
6.8	2043	483	2158
7	2785	738	2583
7.2	5743	1438	4387
7.4	7484	1983	5568
7.6	12653	4937	9948
7.8	19476	8378	15843
8	8386	4427	7493
8.2	5746	2832	5846
8.4	3856	1865	2568
8.8	1563	975	1946
9.1	1178	1023	1325
9.7	563	856	1105
10.2	274	1145	1167
10.6	167	1387	1254
10.9	638	1746	1387
11.1	1837	3387	2458
11.2	4829	5684	3420
11.3	4833	8576	5478
11.6	4254	6854	3387
11.8	3185	2987	2054

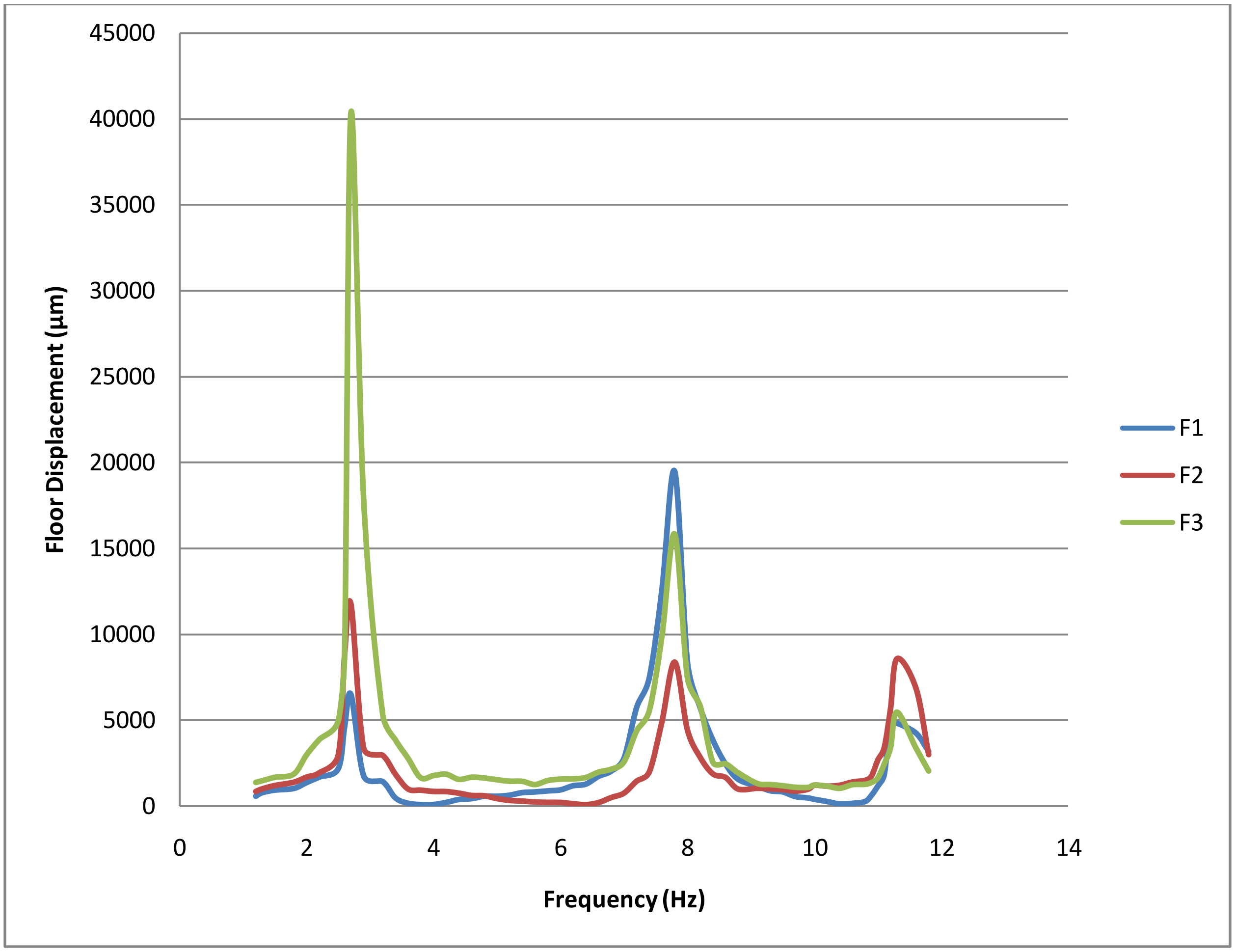


Fig. 5.6 floor displacement corresponding to guiding frequency without mass variation

The graph shows above in figure 5.6 gives the details of displacement of floor with increase in the frequency. The floor displacement is maximum for the top floor in fundamental mode of vibration. The displacement of first floor is observed maximum when the frame vibrates in second mode and the displacement of the second floor is found out maximum when the frame vibrates in third mode.

5.4.2.2 With mass variation (additional mass is putted at different floor levels).

Case 1: In this case the additional mass of 2 kg is putted at the first floor. . Then motor is run at a particular frequency. The floor displacement corresponding to the running frequency is measured from the display unit. Then the frequency of the motor keep on changing gradually and the floor displacement is recorded. During the changing the frequency it is kept in mind that the frequency at resonance should not missed. The record of frequency vs. floor displacement is presented in table 5.4 and the frequency vs. floor displacement graph is plotted in fig. 5.9.

Table 5.4 floor displacement corresponding to guiding frequency with additional mass on 1st floor

Frequency (Hz)	Floor displacement (μm)		
	F1	**F2**	**F3**
1.4	664	1254	777
1.6	605	844	467
2.0	330	2438	2075
2.2	102	1607	2386
2.4	234	4190	5660
2.6	909	14960	21890
2.8	134	3790	6940
3.0	1430	1365	3117
3.6	563	543	748
4.0	464	287	276
4.4	974	654	654
4.8	3865	984	956
5.2	4732	2897	2054
5.5	5876	3054	3870
5.7	8743	3287	8976
5.8	12745	7852	15753
5.9	7843	4065	7054
6.0	4976	2643	4727
6.8	2765	1378	2054
7.4	1543	964	1483
8.9	276	386	372
9.4	950	754	754
10.2	3785	1865	1276
10.6	4764	2476	2053
11.0	8759	3548	4956
11.1	11534	3876	5863
11.2	4987	2454	3876
11.4	2432	1054	986

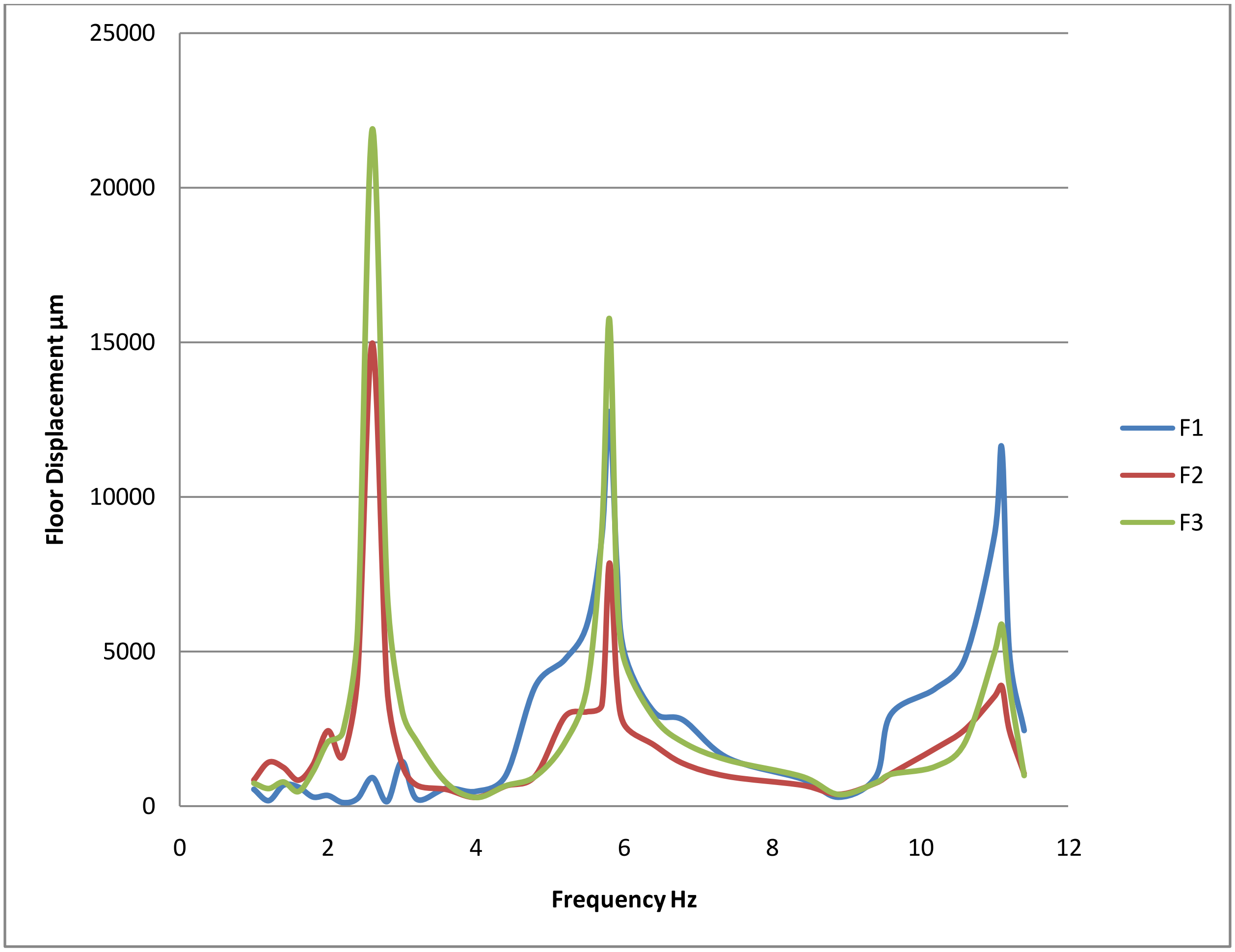


Fig. 5.7 floor displacement corresponding to guiding frequency

The graph provides above in fig. 5.7 gives details of floor displacement vs. frequency for the case of mass variation. The mass irregularity is created by applying additional mass of 2 kg on first floor. The graph shows that the displacement of top floor is observed maximum as compared to other floor displacements when the frame vibrates in fundamental mode. The displacement of second floor is observed maximum when the frame vibrates in second mode and the displacement of first floor is obtained maximum when the frame vibrates in third mode as compared to other floor displacements.

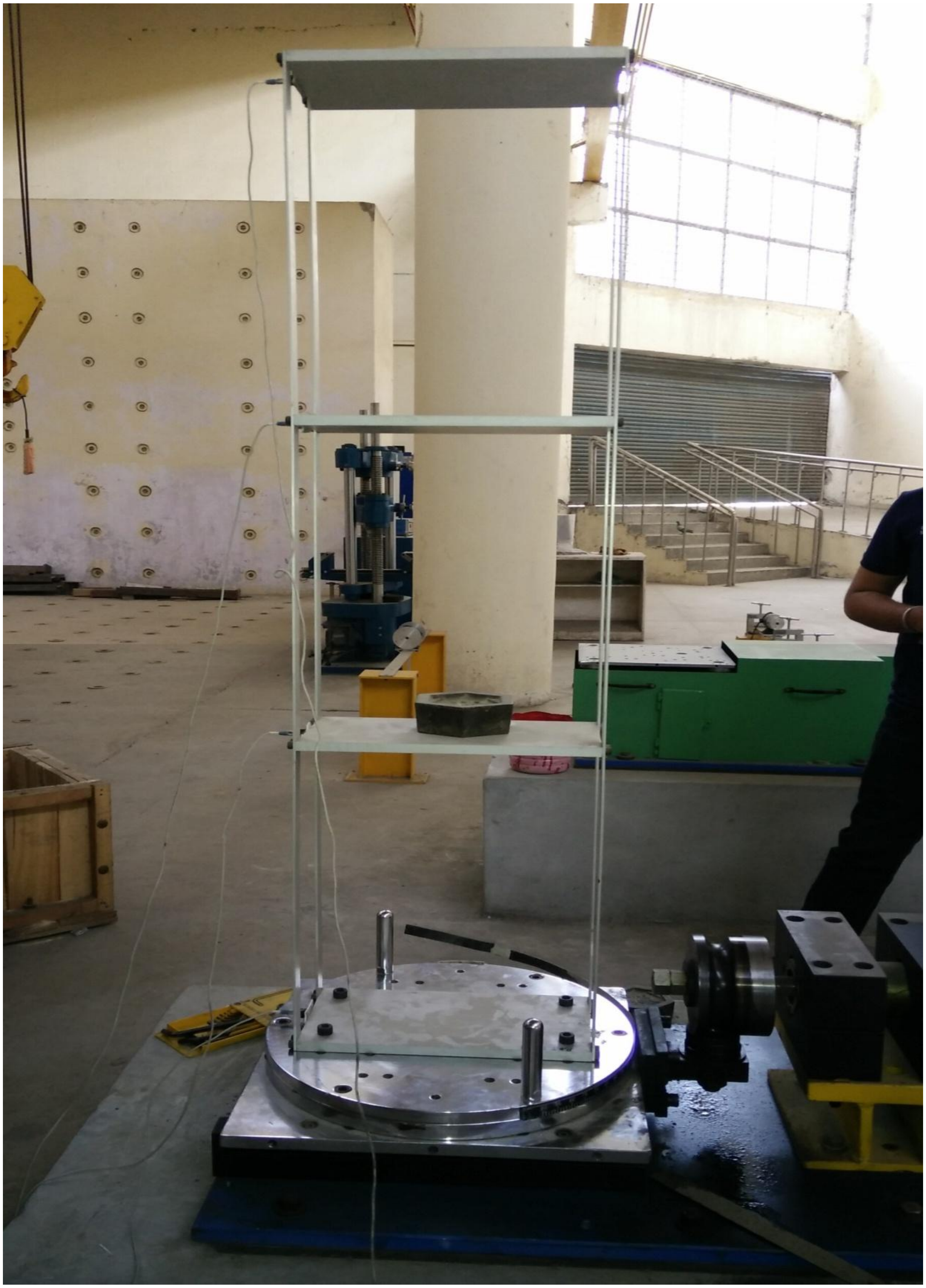


Fig. 5.8 model with additional load on 1st floor

Case 2: In this case the additional mass of 2 kg is putted at the second floor. Then motor is run at a particular frequency. The floor displacement corresponding to the running frequency is measured from the display unit. Then the frequency of the motor keep on changing gradually and the floor displacement is recorded. During the changing the frequency it is kept in mind that the frequency at resonance should not missed. The record of frequency vs. floor displacement is presented in table 5.5 and the frequency vs. floor displacement graph is plotted in fig. 5.9.

Table 5.5 floor displacement corresponding to guiding frequency with additional mass on 2nd floor

Frequency (Hz)	Floor displacement (μm)		
	F1	**F2**	**F3**
1	602	996	506
1.3	182	1533	522
1.6	622	1258	777
2.0	471	1612	2097
2.2	250	2567	3725
2.4	900	8220	15260
2.6	123	645	1836
3.0	202	1080	1964
3.4	384	2664	1678
4.0	243	1054	1298
4.8	95	542	1287
5.4	353	105	324
6.2	543	738	874
6.8	1054	2043	1879
7.0	2865	2176	4098
7.1	4054	3087	10654
7.2	2053	1654	5987
8.0	1674	987	2976
8.5	975	287	253
8.9	876	969	543
9.0	1252	956	1754
9.2	4324	1764	3645
9.4	2054	756	2065
9.8	654	784	754

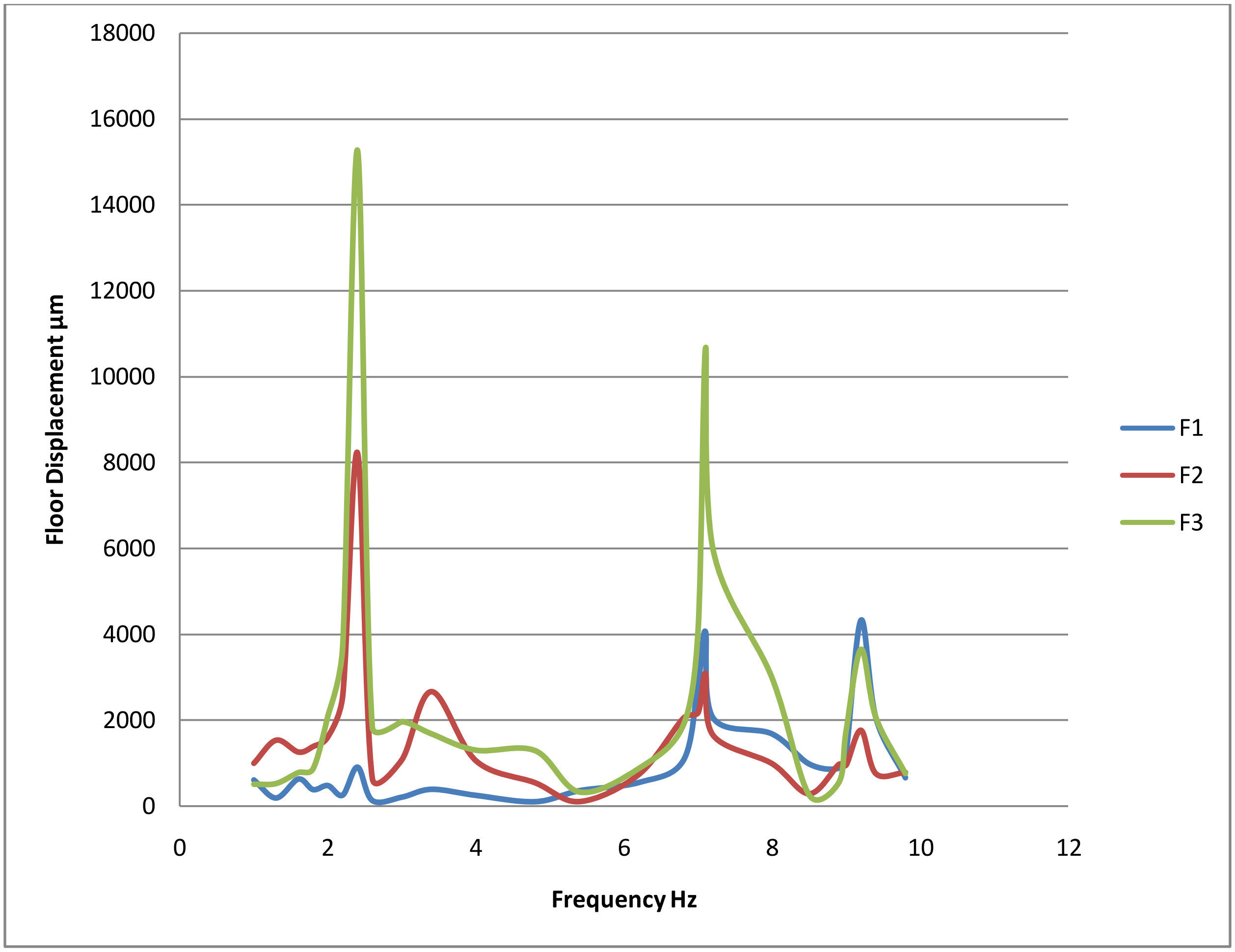


Fig. 5.9 floor displacement corresponding to guiding frequency with additional mass on 1st floor

The graph provides above in fig. 5.9 gives details of floor displacement vs. frequency for the case of mass variation. The mass irregularity is created by applying additional mass of 2 kg on second floor. The graph shows that the displacement of top floor is observed maximum as compared to other floor displacements when the frame vibrates in fundamental mode. The displacement of third floor is observed maximum when the frame vibrates in second mode and the displacement of first floor is obtained maximum when the frame vibrates in third mode as compared to other floor displacements.

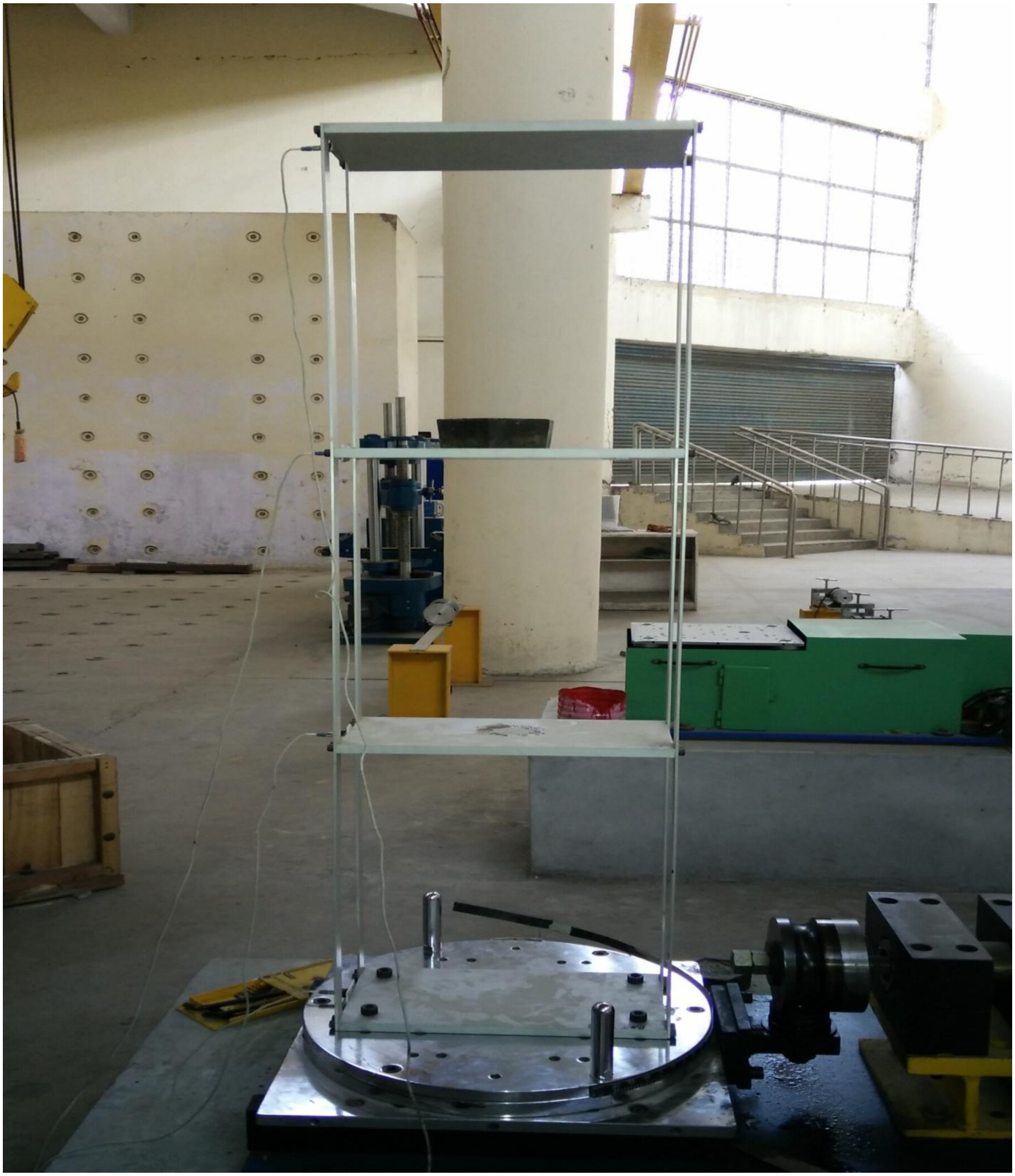


Fig. 5.10 model with additional load on 2^{nd} floor

Case 3: In this case the additional mass of 2 kg is putted at the first floor. . Then motor is run at a particular frequency. The floor displacement corresponding to the running frequency is measured from the display unit. Then the frequency of the motor keep on changing gradually and the floor displacement is recorded. During the changing the frequency it is kept in mind that the frequency at resonance should not missed. The record of frequency vs. floor displacement is presented in table 5.6 and the frequency vs. floor displacement graph is plotted in fig. 5.11.

Table 5.6 floor displacement corresponding to guiding frequency with additional mass on 3rd floor

Frequency Hz	Floor displacement (μm)		
	F1	F2	F3
1.1	664	872	502
1.3	994	617	555
1.5	771	721	618
1.7	845	1299	1097
1.9	398	2266	1975
2.2	4765	10380	16830
2.4	608	1676	4175
2.6	608	234	1357
2.8	208	1000	1061
3.4	105	876	275
4.8	95	948	432
5.6	376	387	976
6.0	657	765	2786
6.4	1654	1077	5837
6.9	3876	2065	8964
7.2	1187	986	3876
7.8	1048	765	1754
8.4	765	`654	974
8.9	276	`127	876
9.8	574	374	276
10.7	876	987	326
10.9	1987	967	1065
11.1	2436	1563	1974
11.3	1054	867	897

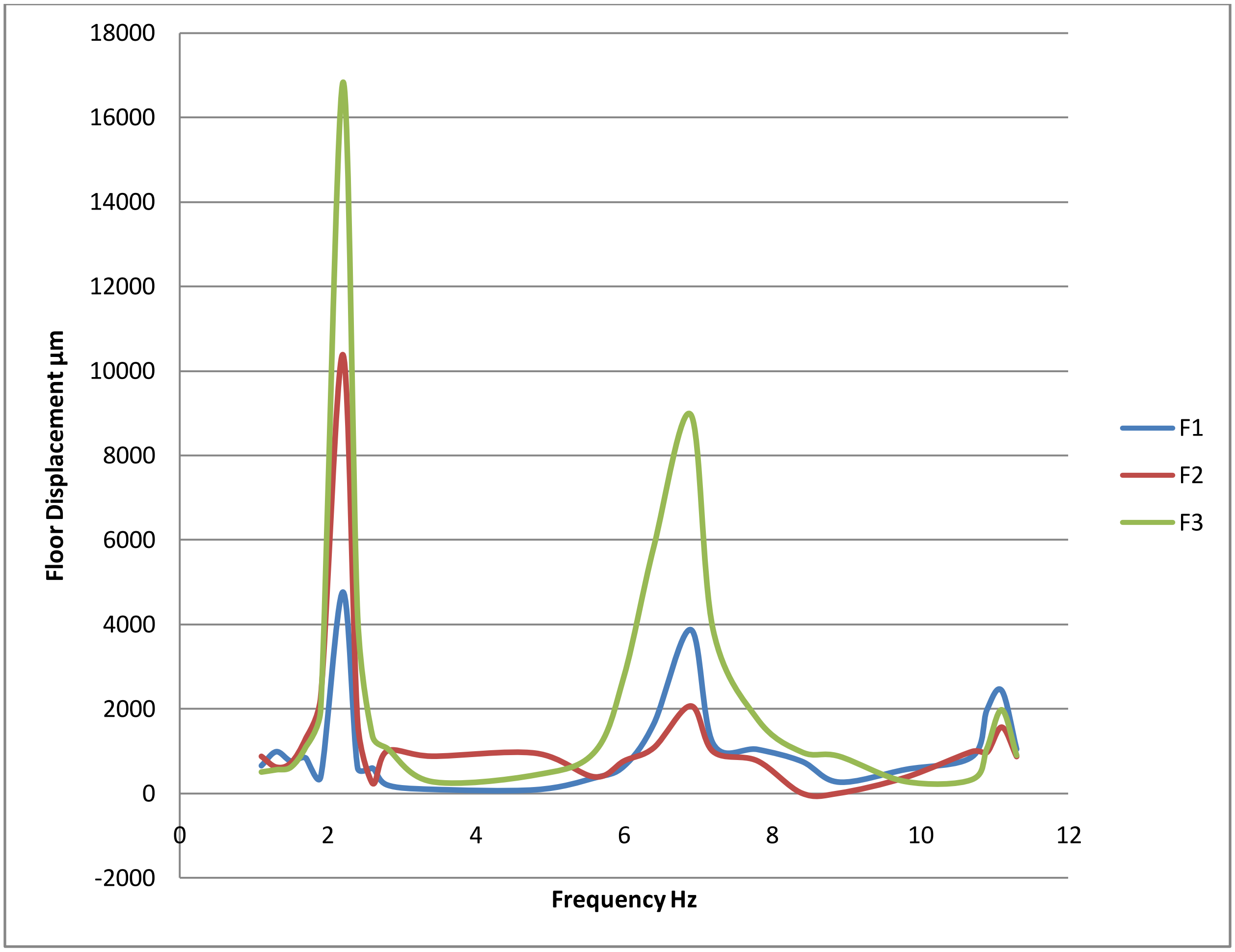


Fig. 5.11 floor displacement corresponding to guiding frequency with additional mass on 3^{rd} floor

The graph provides above in fig. 5.11 gives details of floor displacement vs. frequency for the case of mass variation. The mass irregularity is created by applying additional mass of 2 kg on first floor. The graph shows that the displacement of top floor is observed maximum as compared to other floor displacements when the frame vibrates in fundamental mode. The displacement of top floor is observed maximum when the frame vibrates in second mode and the displacement of first floor is obtained maximum when the frame vibrates in third mode as compared to other floor displacements.

Fig. 5.12 model with additional mass on 3rd floor

Case 4: In this case the additional mass of 2 kg is putted at the first floor and additional mass of 1 kg is applied on second floor. Then motor is run at a particular frequency. The floor displacement corresponding to the running frequency is measured from the display unit. Then the frequency of the motor keep on changing gradually and the floor displacement is recorded. During the changing the frequency it is kept in mind that the frequency at resonance should not missed. The record of frequency vs. floor displacement is presented in table 5.6 and the frequency vs. floor displacement graph is plotted in fig. 5.13.

Table 5.7 floor displacement corresponding to guiding frequency with additional mass on 1st floor and on 2nd floor

Frequency (Hz)	Floor displacement (μm)		
	F1	F2	F3
1	354	1702	766
1.4	835	620	658
1.8	901	1534	950
2.0	162	1494	1891
2.2	124	2655	4203
2.4	1645	3171	13880
2.5	2974	3286	17560
2.6	987	1178	3996
2.8	280	1595	2777
3.0	291	2547	1706
3.8	756	987	97
4.8	765	453	543
5.4	57	87	874
6.0	785	875	1745
6.2	897	975	3985
6.5	2634	1985	9745
6.7	1065	1054	4875
7.9	574	986	1765
8.0	373	768	873
8.4	265	86	67
8.8	865	365	473
9.0	976	564	785
9.2	2084	965	1784
9.4	876	657	876
9.8	765	165	567

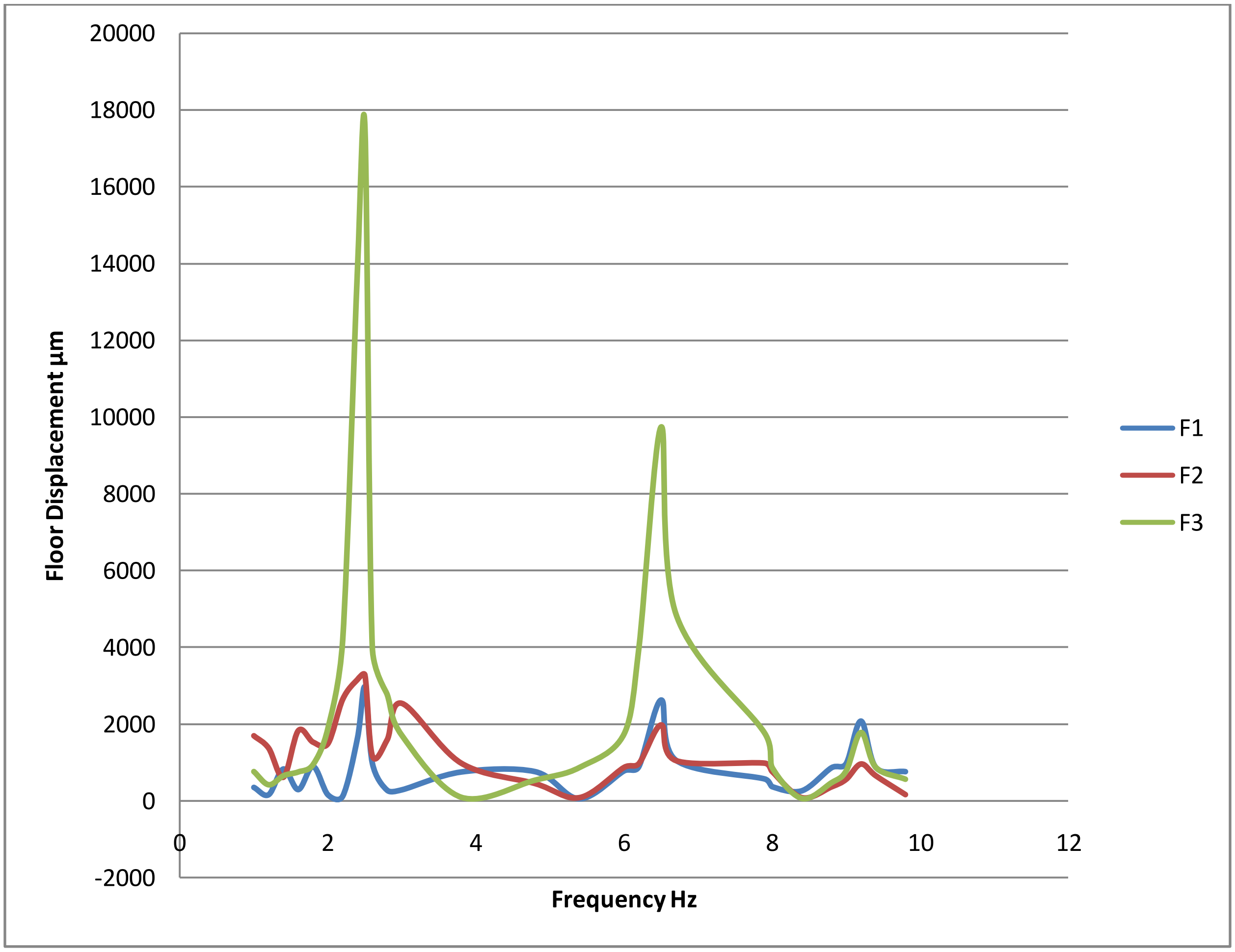


Fig. 5.13 floor displacement corresponding to guiding frequency with additional mass on 1st floor and on 2nd floor

The graph provides above in fig. 5.3 gives details of floor displacement vs. frequency for the case of mass variation. The mass irregularity is created by applying additional mass of 2 kg on first floor and additional mass of 1 kg on second floor. The graph shows that the displacement of top floor is observed maximum as compared to other floor displacements when the frame vibrates in fundamental mode. The displacement of top floor is observed maximum when the frame vibrates in second mode and the displacement of first floor is obtained maximum when the frame vibrates in third mode as compared to other floor displacements.

Fig 5.14 model with additional mass on 1st and 2nd floor

Case 5: In this case the additional mass of 2 kg is putted at the second floor and additional mass of 1 kg on third floor. Then motor is run at a particular frequency. The floor displacement corresponding to the running frequency is measured from the display unit. Then the frequency of the motor keep on changing gradually and the floor displacement is recorded. During the changing the frequency it is kept in mind that the frequency at resonance should not missed. The record of frequency vs. floor displacement is presented in table 5.7 and the frequency vs. floor displacement graph is plotted in fig. 5.15.

Table 5.8 floor displacement corresponding to guiding frequency with additional mass on 2nd floor and 3rd floor

Frequency (Hz)	Floor displacement (μm)		
	F1	**F2**	**F3**
1.2	695	980	412
1.4	280	1430	474
1.8	269	318	1587
2.0	985	741	3915
2.2	2876	3291	15170
2.4	436	766	2119
2.6	149	4407	1126
3.0	754	376	865
3.8	324	128	275
4.4	654	543	876
5.0	573	675	1275
6.0	868	987	4532
6.6	2585	1968	8976
6.6	1643	785	6344
6.8	354	477	378
7.2	746	797	856
7.7	866	686	358
8.1	956	976	885
8.5	753	478	685
8.7	896	987	965
8.9	2073	1763	1975
9.2	765	876	1056
9.5	235	744	684
9.8	263	7848	654

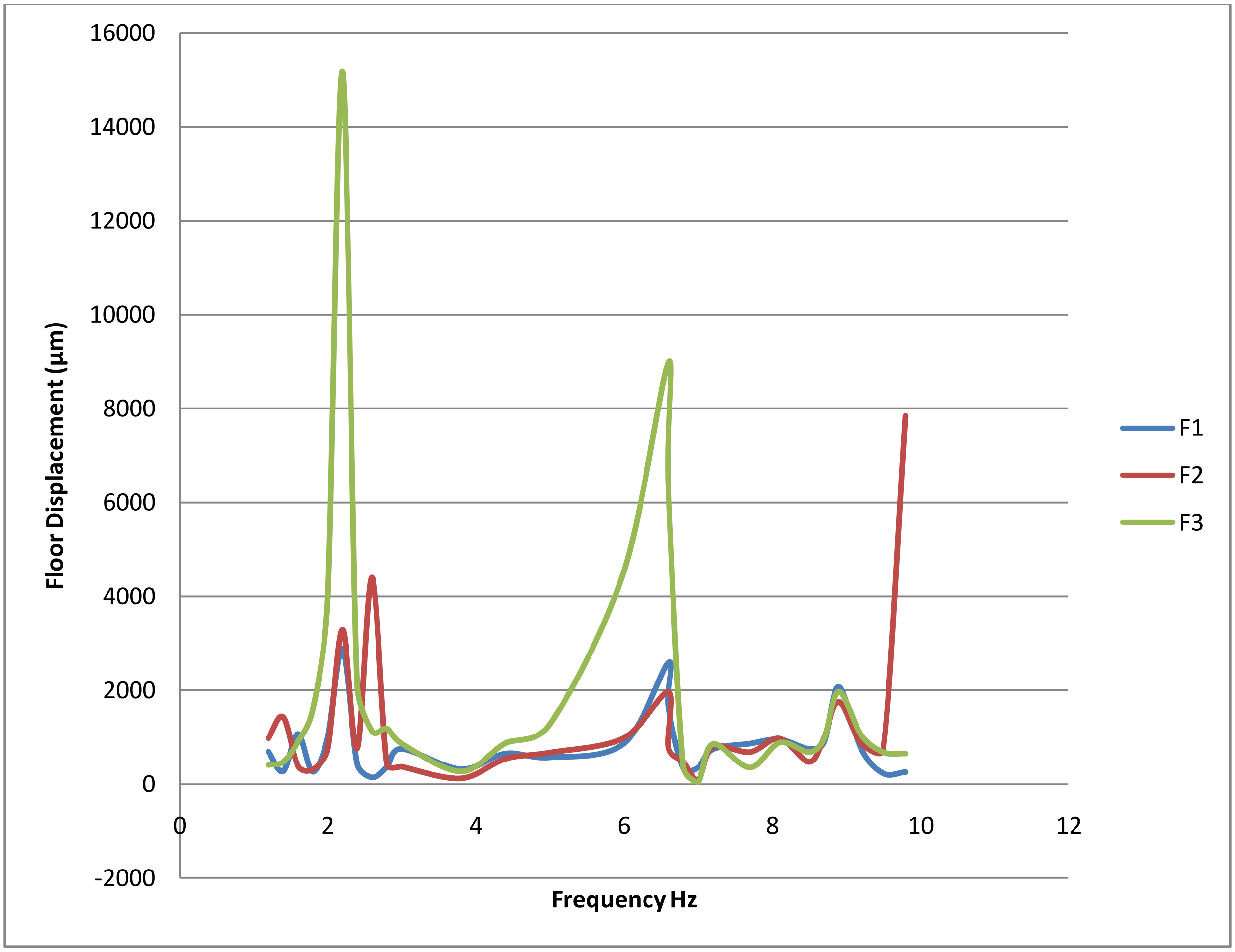


Fig. 5.15 floor displacement corresponding to guiding frequency with additional mass on 2nd floor and 3rd floor

The graph provides above in fig. 5.15 gives details of floor displacement vs. frequency for the case of mass variation. The mass irregularity is created by applying additional mass of 2 kg on second floor and additional mass of 1 kg on third floor. The graph shows that the displacement of top floor is observed maximum as compared to other floor displacements when the frame vibrates in fundamental mode. The displacement of top floor is observed maximum when the frame vibrates in second mode and the displacement of first floor is obtained maximum when the frame vibrates in third mode as compared to other floor displacements.

Fig. 5.16 model with additional mass on 2nd floor and 3rd floor

CHAPTER 6

ETABS MODELLING

6.1 Introduction

The frame is modeled in Etabs software to validate the result that I have obtained from the analytical and experimental analysis. The model is analyzed with no additional mass on the floor level and with additional mass on different floor level. The time period and frequency is calculated for every case, with or without additional mass. The mass is varied along the height of the frame. The properties of the model are taken from the analytical and experimental analysis. The damping ratio that is calculated from the experimental analysis is used for modeling. For dynamic analysis of the model zone IV is considered and the response reduction factor is taken 5. The soil strata are considered medium. The model prepared in the Etabs software is given in figure 5.1.

Following cases were adopted while carrying out analysis in Etabs:

Case 1: without mass variation.

Case 2: with mass variation.

The mass is varied at different floor level as follows:

f) With additional mass on 1^{st} floor =2 kg
g) With additional mass on 2^{nd} floor = 2 kg
h) With additional mass on 3^{rd} floor = 2 kg
i) With additional mass on 1^{st} floor = 2 kg and on 2^{nd} floor = 1 kg
j) With additional mass on 2^{nd} floor = 2 kg and on 3^{rd} floor = 1 kg

The dynamical analysis is carried out for the above cases. The time periods and frequencies are obtained for different modes. The frequencies and time periods obtained are plotted in tables 6.1, 6.2, 6.3, 6.4, 6.5 and 6.6. The mode shapes are plotted in fig. 6.1 (a, b and c), 6.2 (a, b and c), 6.3 (a, b and c), 6.4 (a, b and c), 6.5 (a, b and c) and 6.6 (a, b and c).

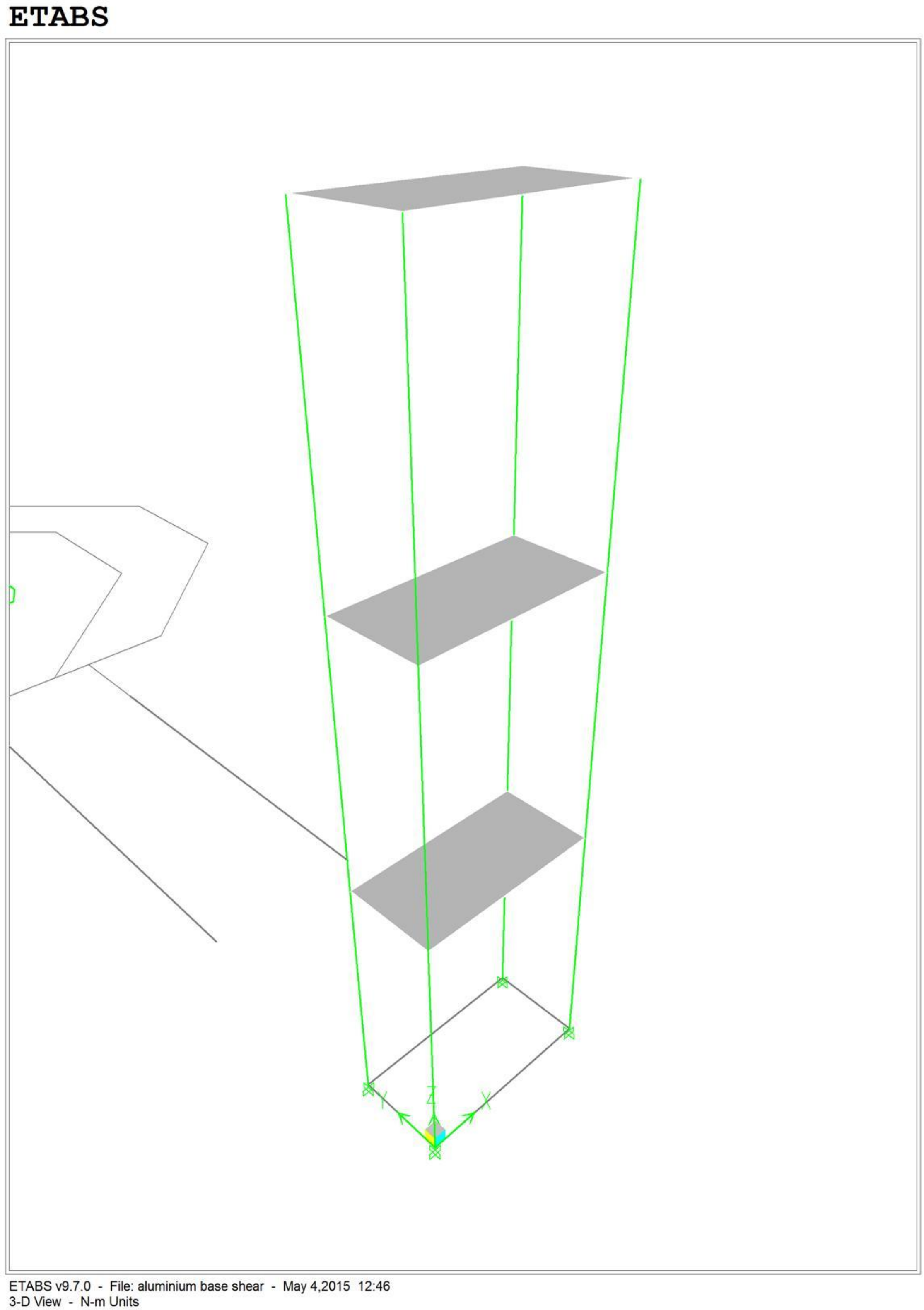
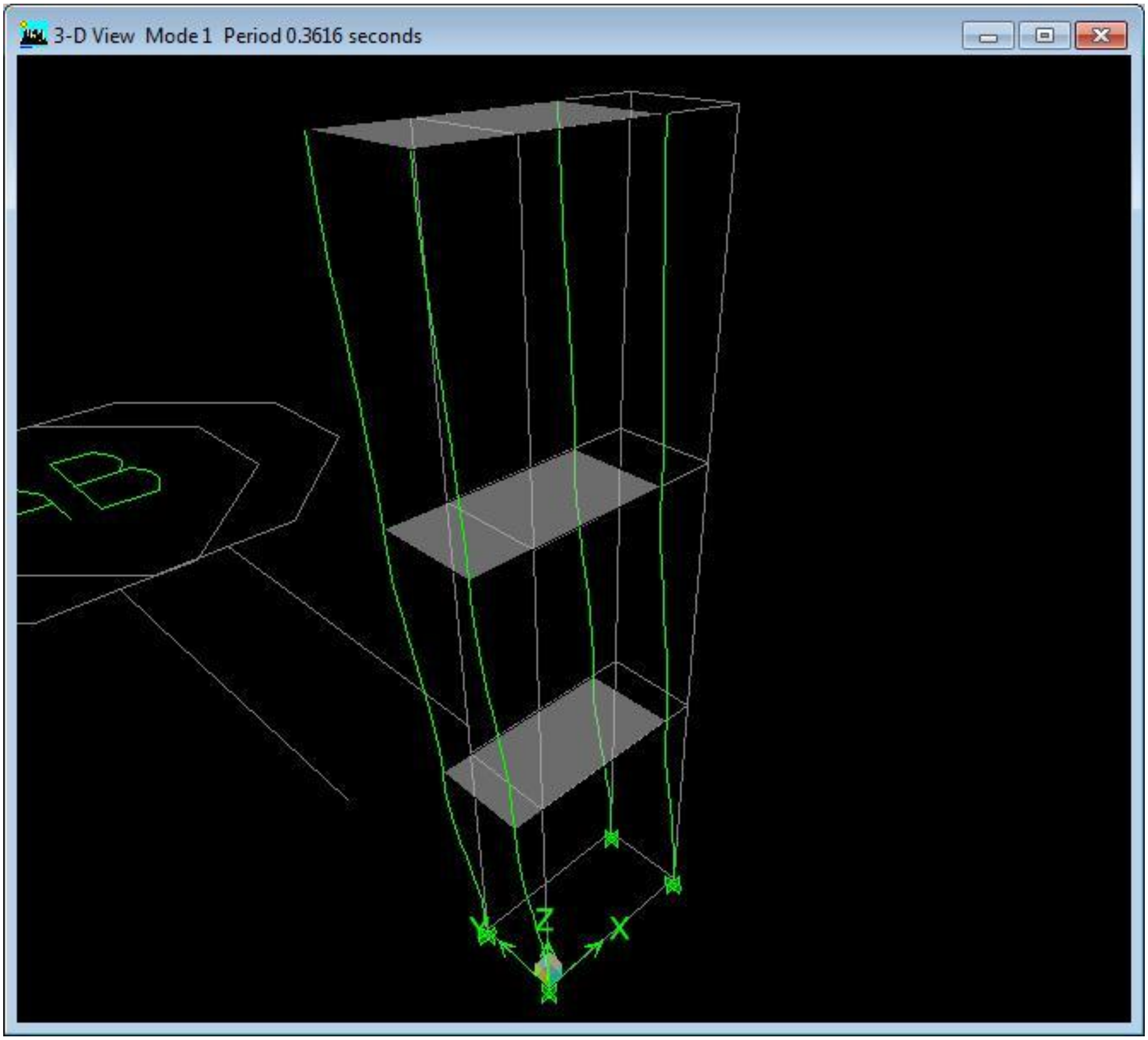


Fig. 6.1 Etabs model

Table 6.1 Frequencies and time periods from Etabs analysis without mass variation

Modes	1	2	3
Time period (sec)	0.36	0.13	0.09
Frequency (Hz)	2.76	7.69	10.98
Angular frequency (rad/sec)	13.78	48.33	69.04



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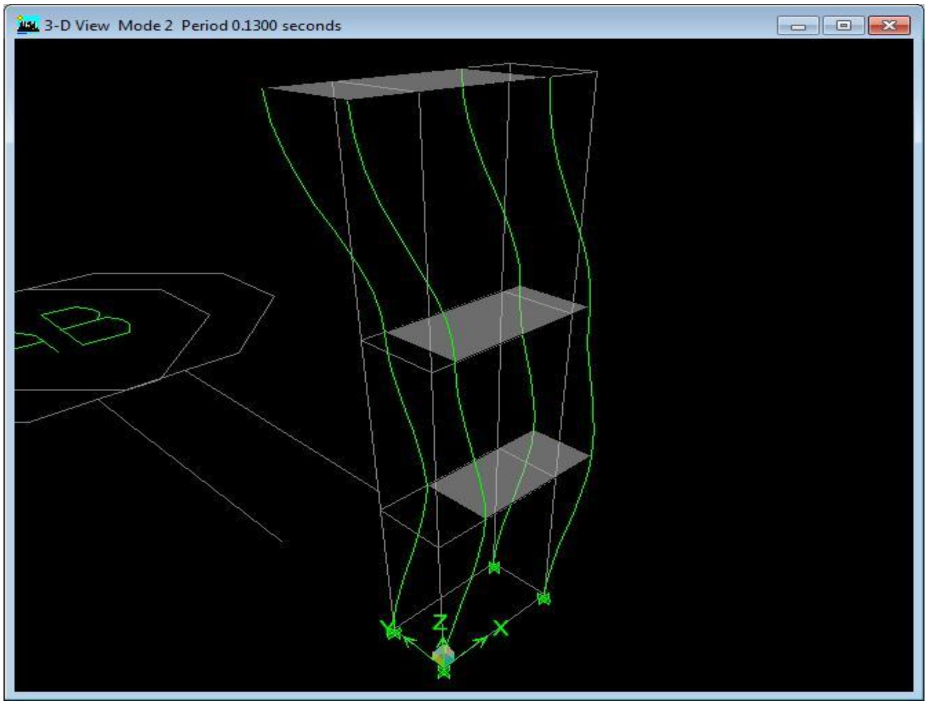

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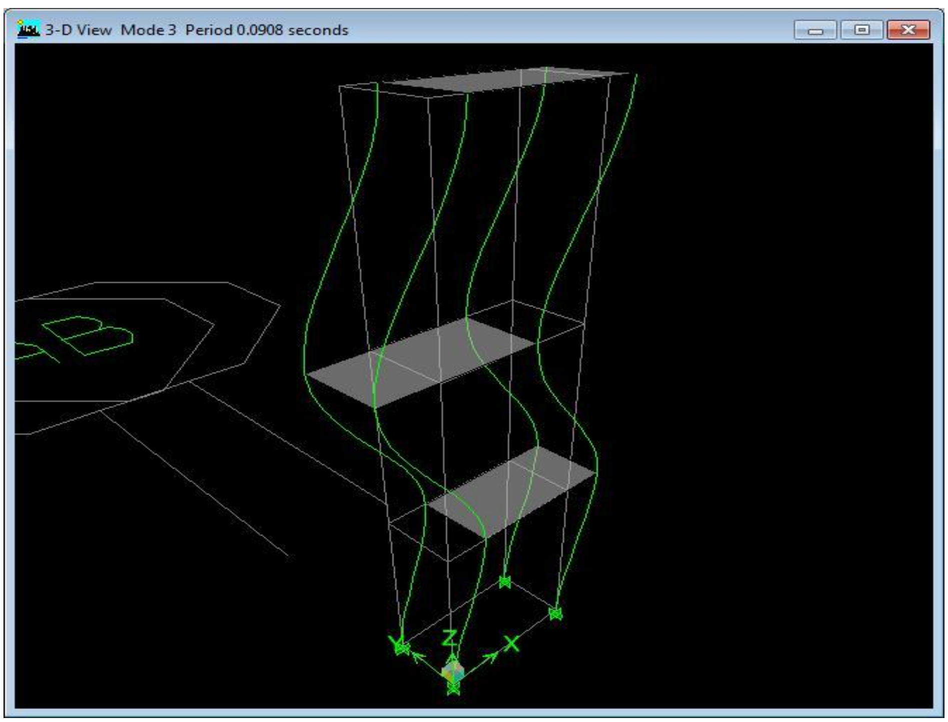
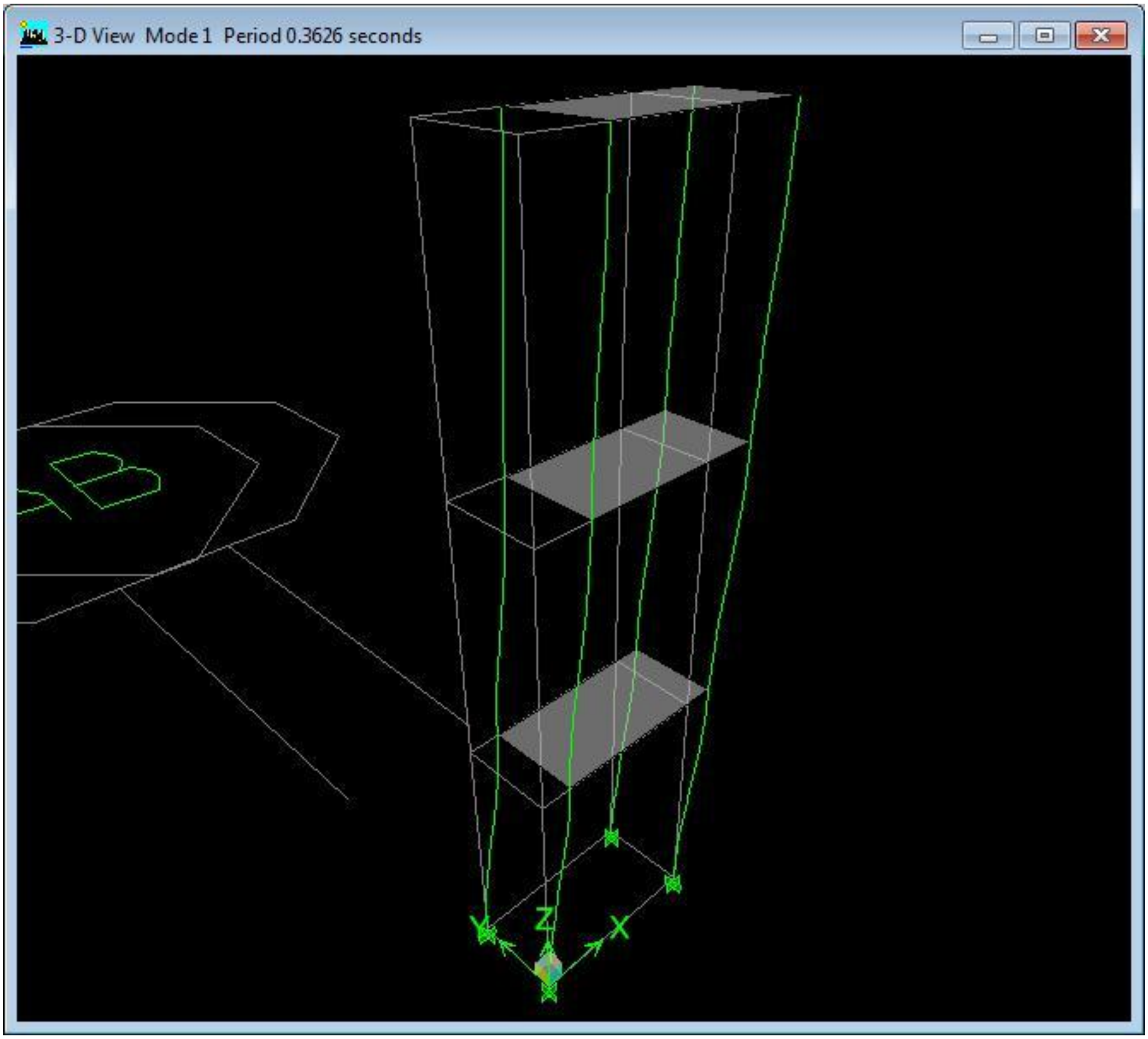


(c)

Fig.6.2 (a, b and c) mode shapes from Etabs analysis without mass variation

Table 6.2 Frequencies and time periods from Etabs analysis with additional mass on 1st floor

Modes	1	2	3
Time period (sec)	0.36	0.13	0.09
Frequency (Hz)	2.75	7.57	10.87
Angular frequency (rad/sec)	17.30	47.60	68.29



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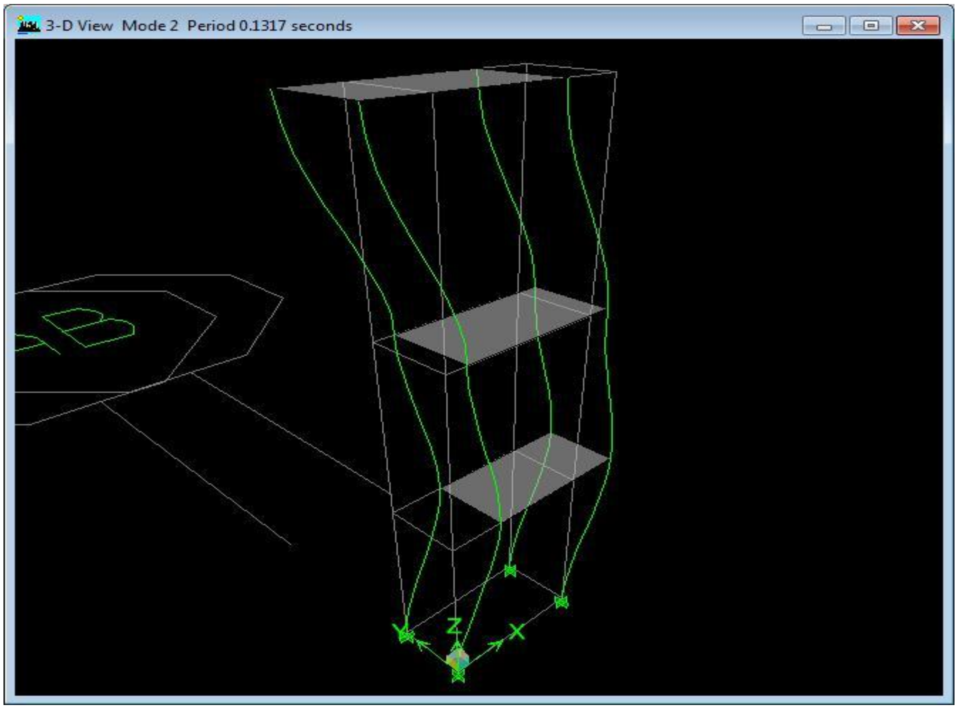

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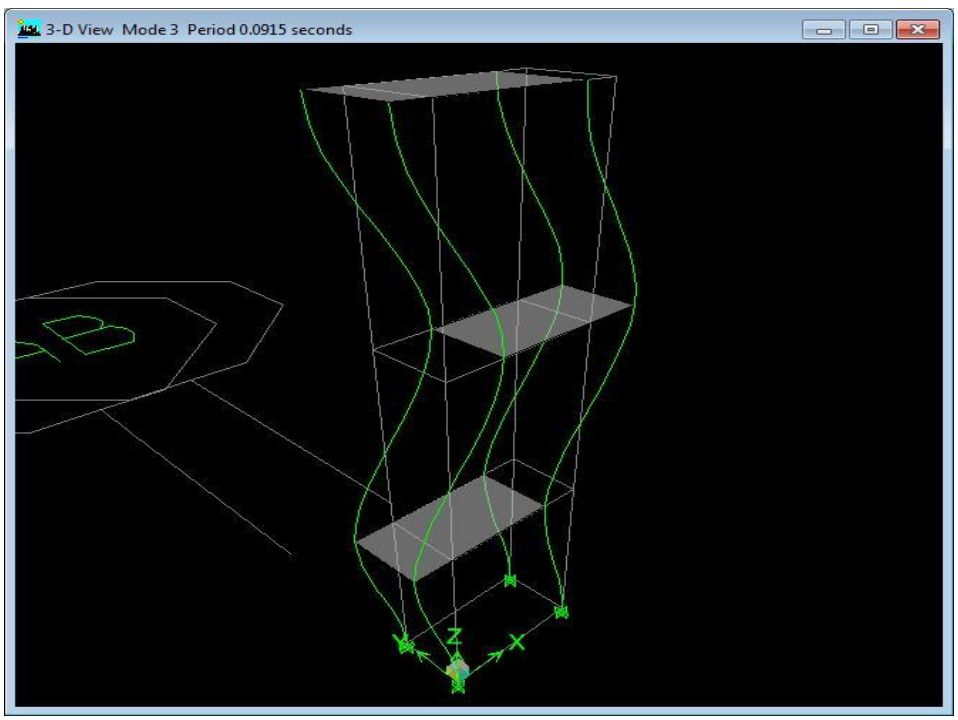
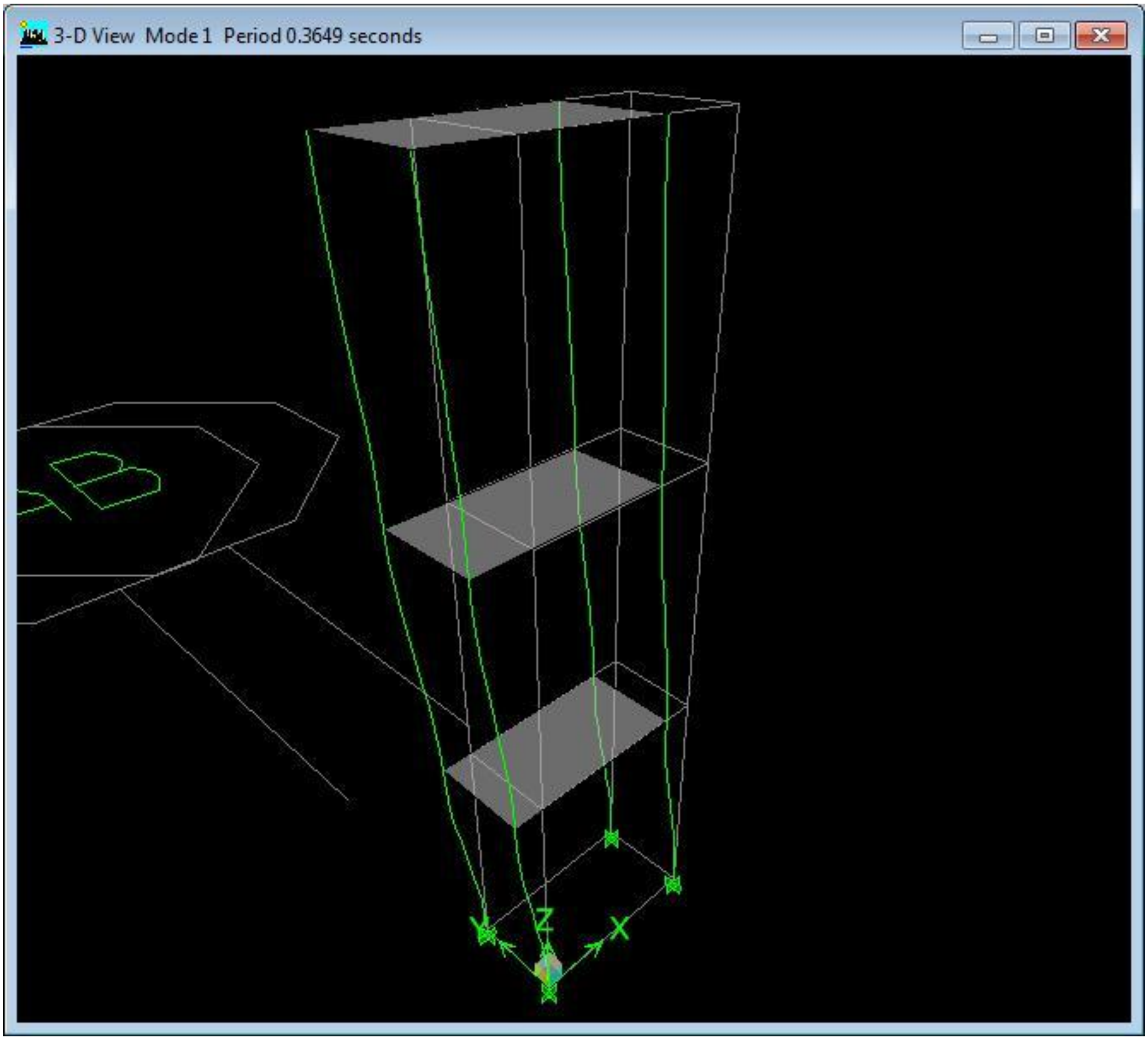


(c)

Fig.6.3 (a, b and c) mode shapes from Etabs analysis with additional mass on 1st floor

Table 6.3 Frequencies and time periods from Etabs with additional mass on 2^{nd} floor

Modes	1	2	3
Time period (sec)	0.37	0.13	0.092
Frequency (Hz)	2.74	7.69	10.87
Angular frequency (rad/sec)	17.21	48.33	68.29



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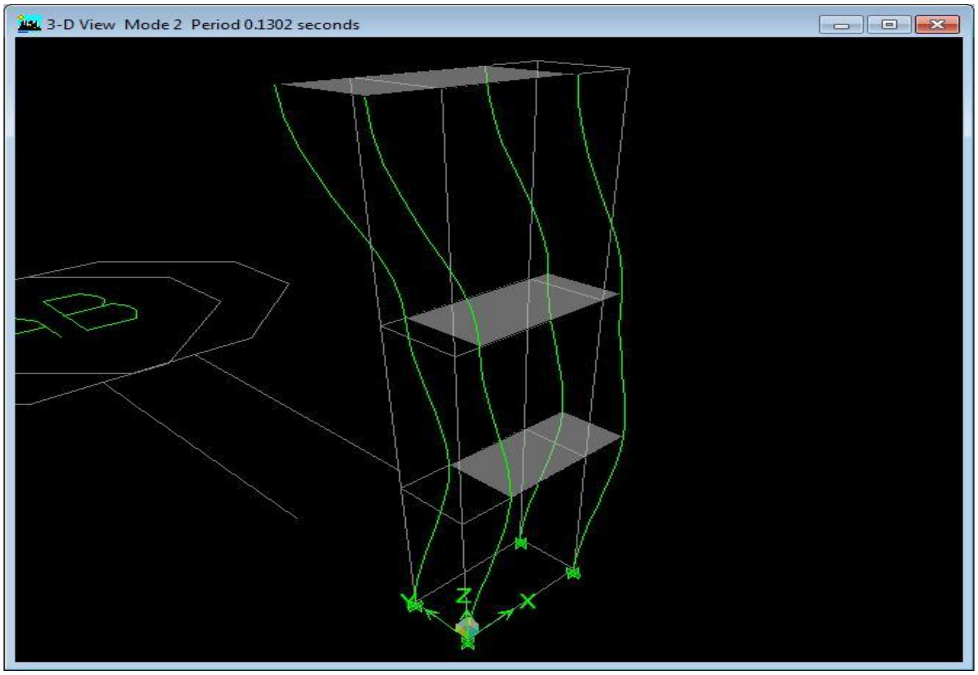

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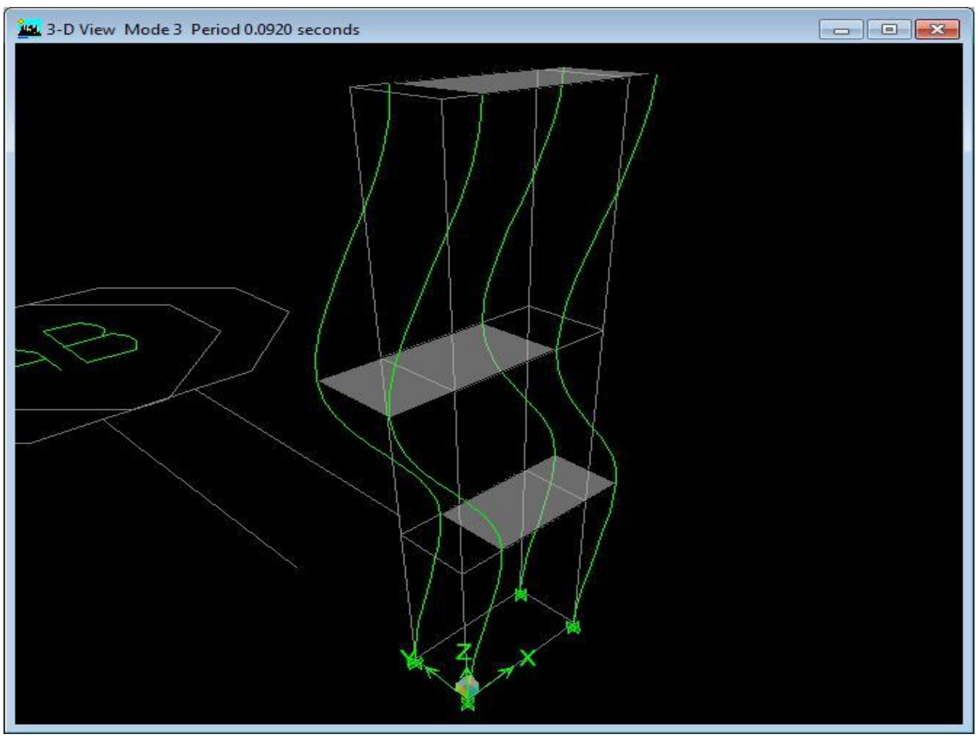
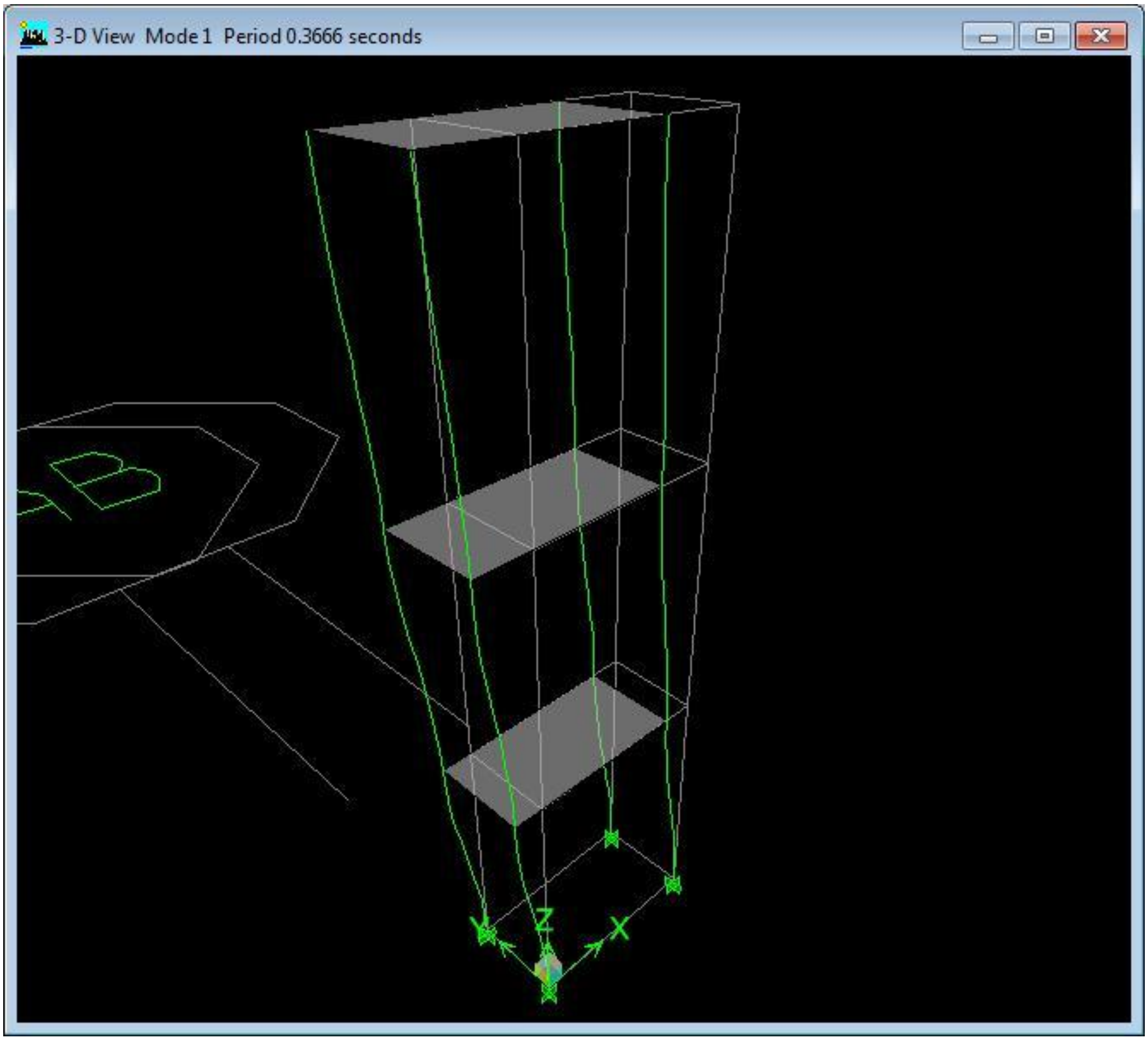


(c)

Fig. 6.4 (a, b and c) mode shapes from Etabs analysis with additional mass on 2nd floor

Table 6.4 Frequencies and time periods from Etabs analysis with additional mass on 3rd floor

Modes	1	2	3
Time period (sec)	0.37	0.13	0.091
Frequency (Hz)	2.72	7.69	10.98
Angular frequency (rad/sec)	17.12	48.33	69.04



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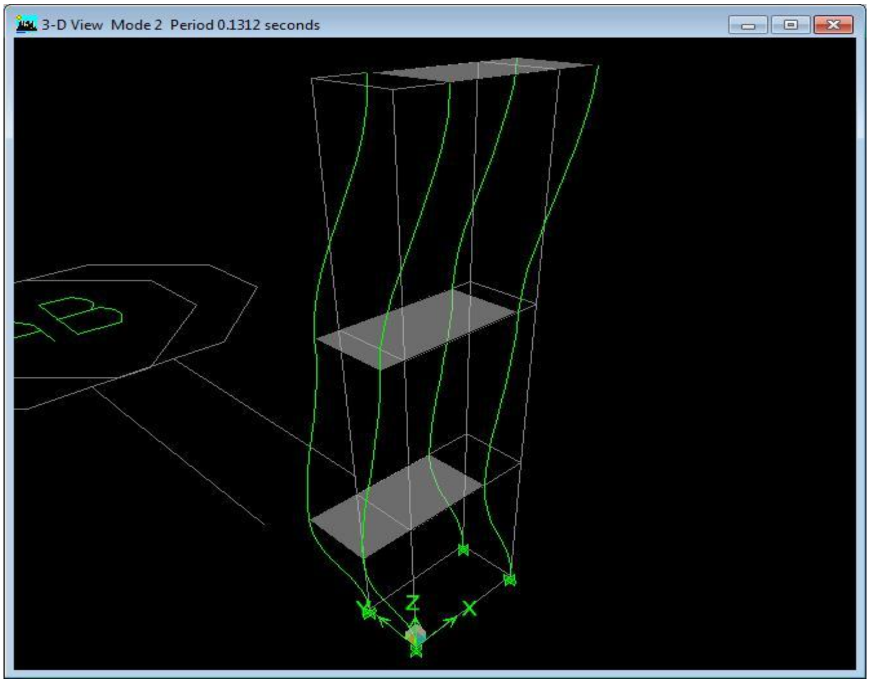

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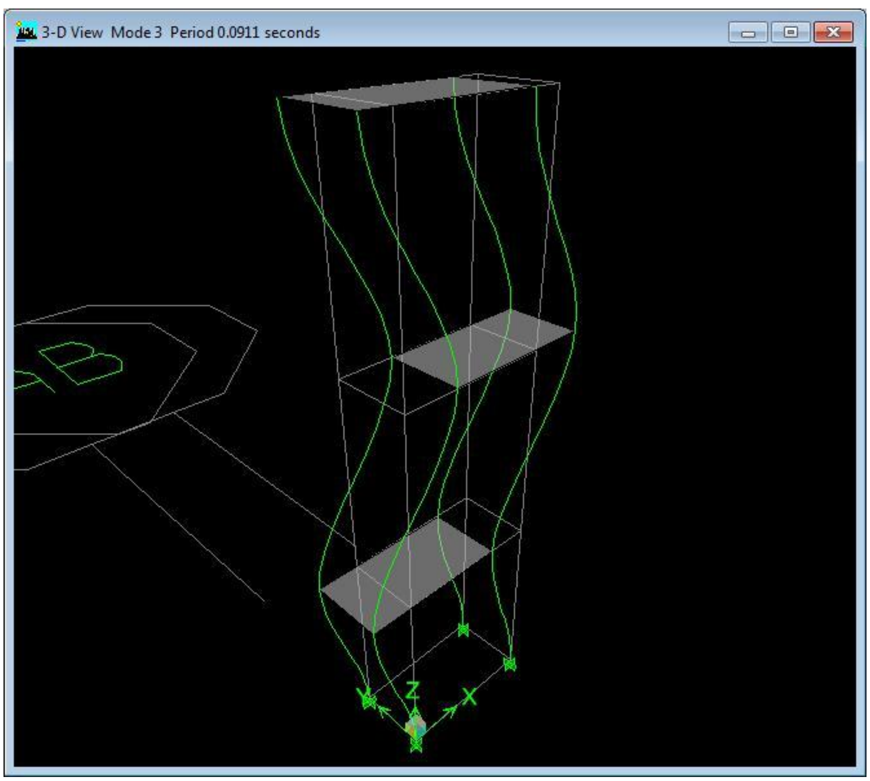
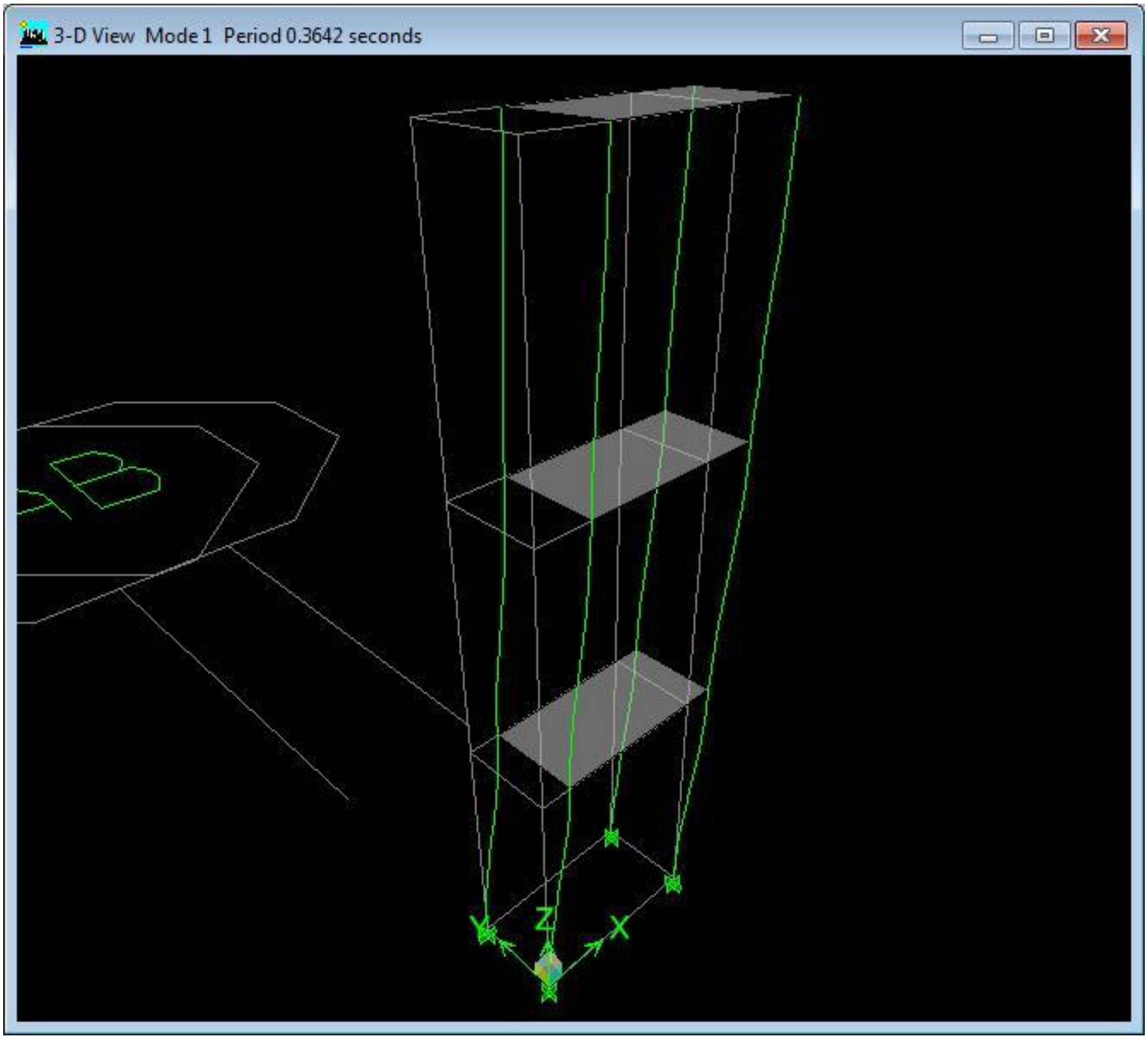


(c)

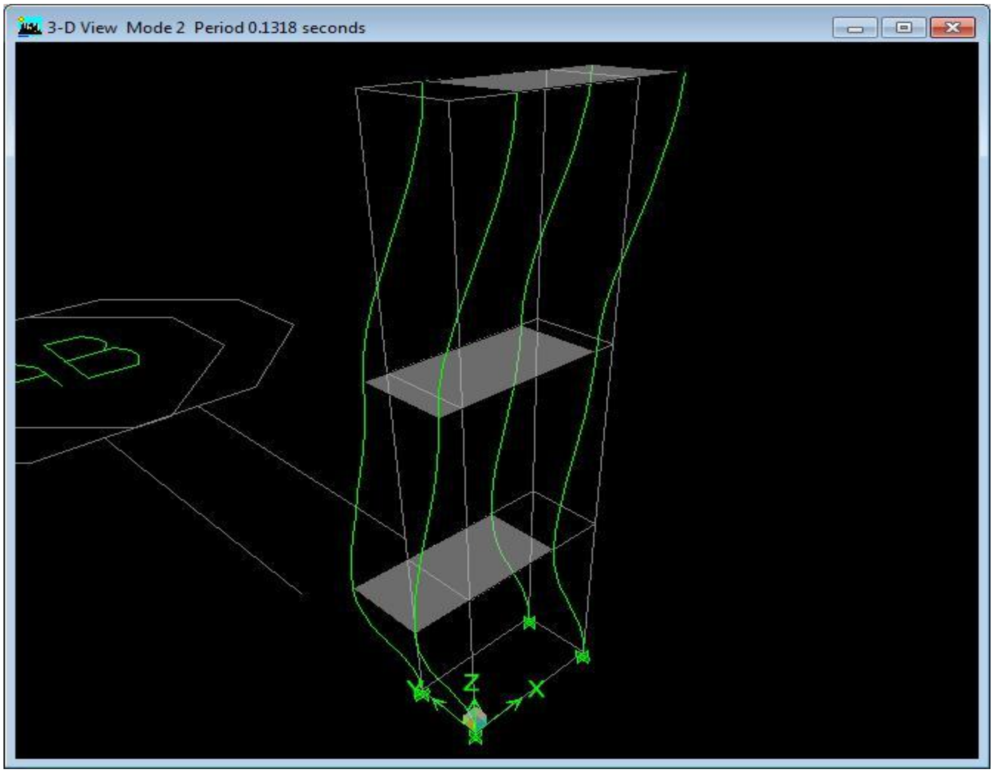
Fig. 6.5(a, b and c) mode shapes from Etabs analysis with additional mass on 3rd floor

Table 6.5 Frequencies and time period from Etabs analysis with additional mass on 1st floor on 2nd floor

Modes	1	2	3
Time period (sec)	0.36	0.13	0.09
Frequency (Hz)	2.75	7.63	10.87
Angular frequency (rad/sec)	17.26	47.96	68.29



(a)



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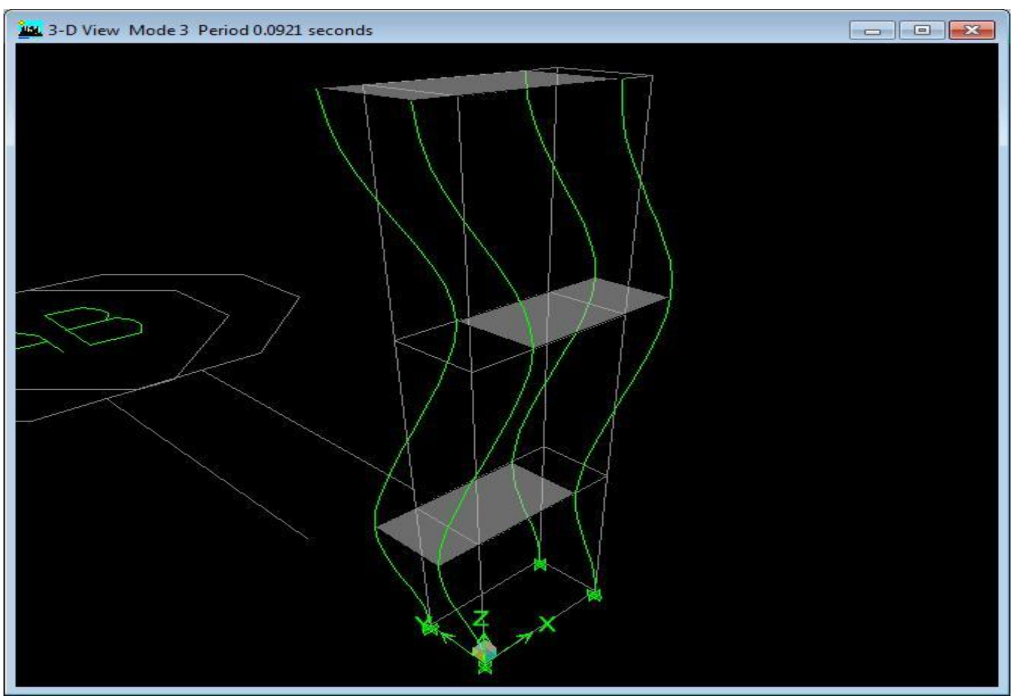
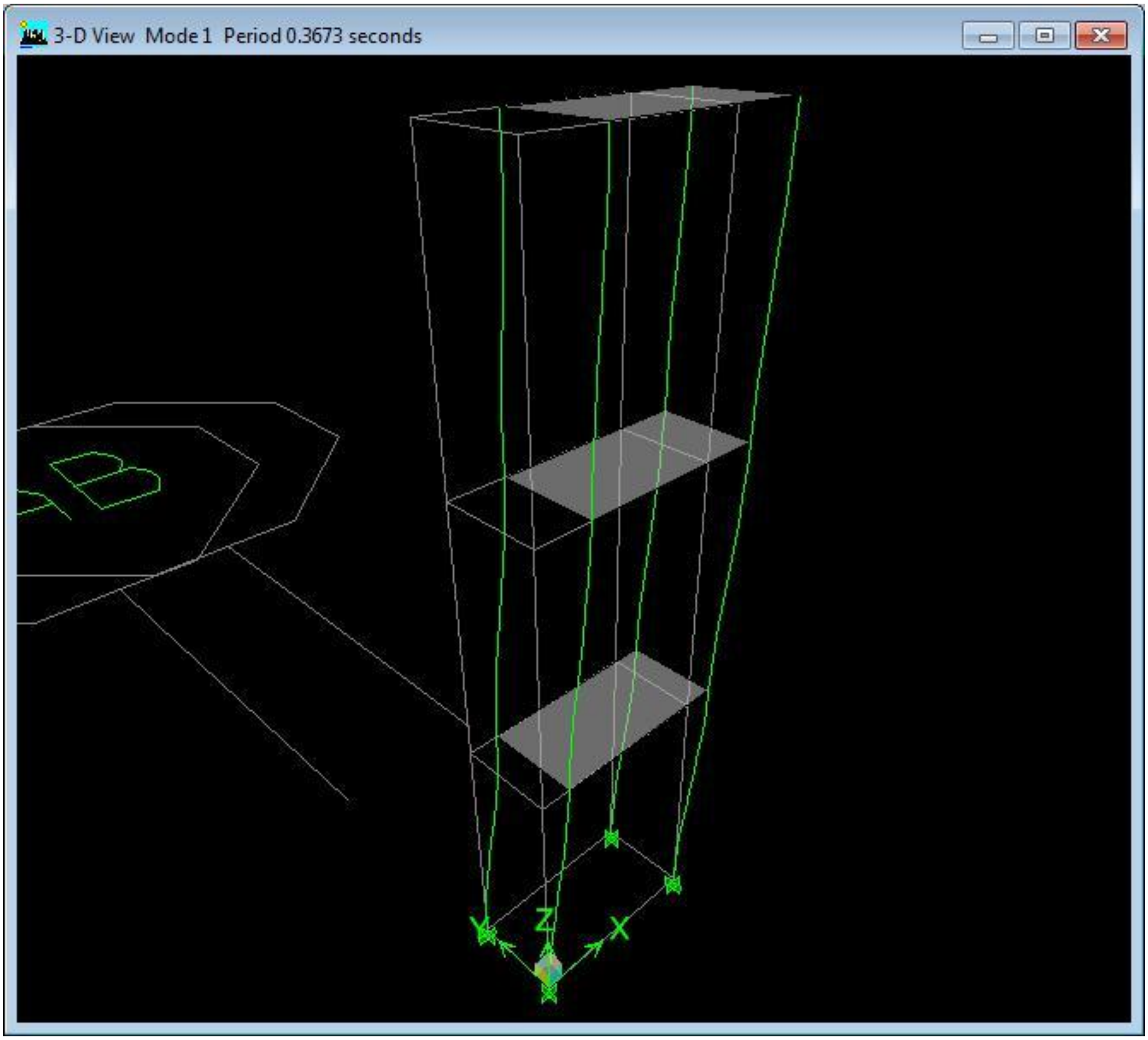


(c)

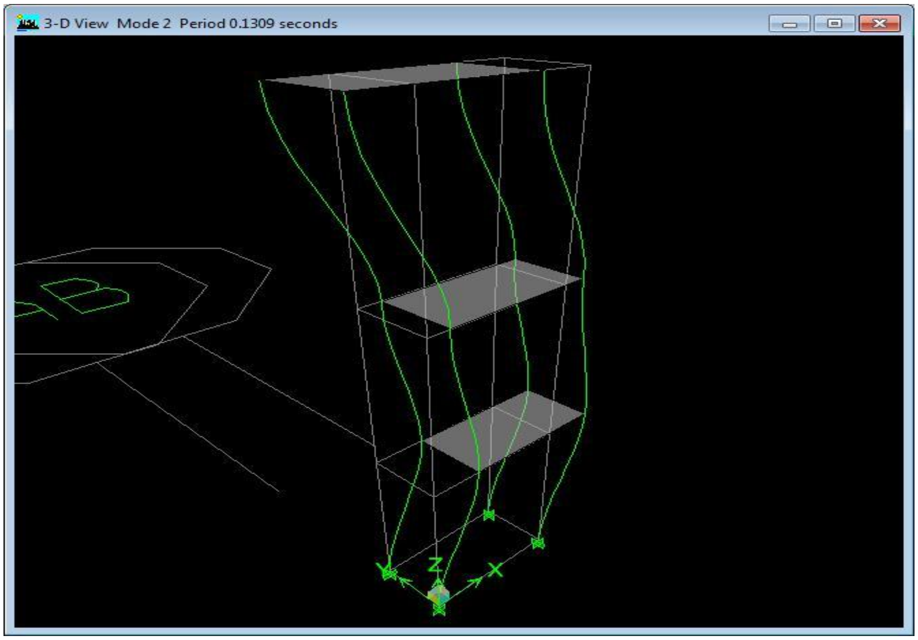
Fig. 6.6 (a, b and c) mode shapes from Etabs analysis with additional mass on 1st floor and on 2nd floor

Table 6.6 Frequencies and time period from Etabs analysis with additional mass on 2nd floor and on 3rd floor

Modes	1	2	3
Time period (sec)	0.37	0.13	0.09
Frequency (Hz)	2.72	7.69	10.67
Angular frequency (rad/sec)	17.12	48.33	68.29



(a)



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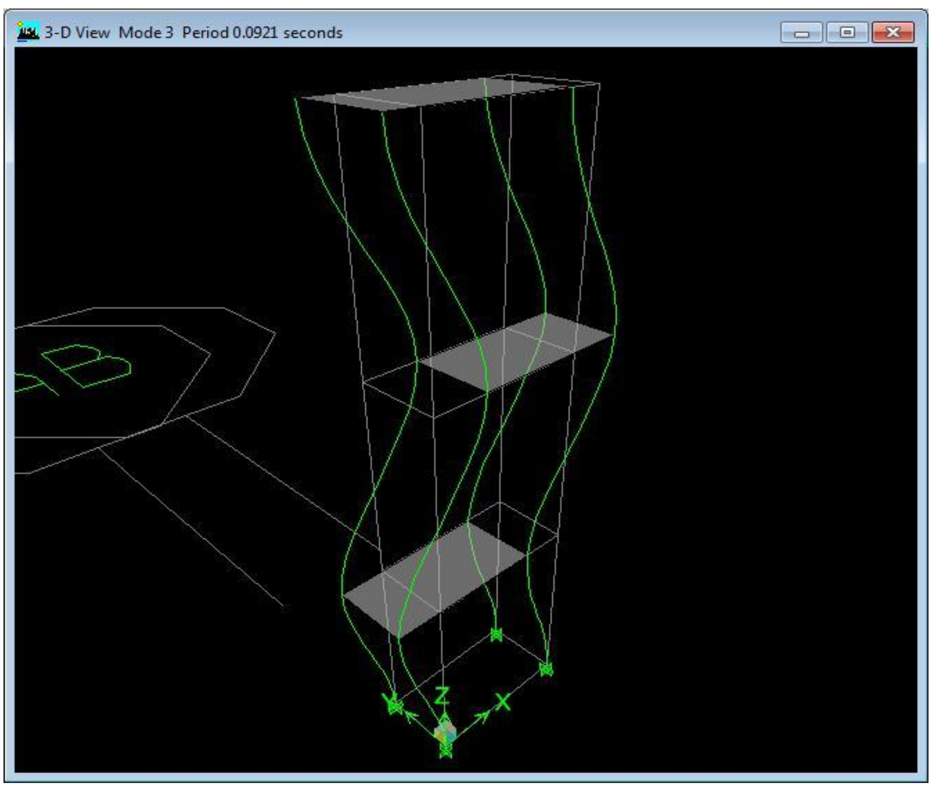

(c)

Fig. 6.7(a, b and c) mode shape from Etabs analysis with additional mass on 2nd floor and on 3rd floor

CHAPTER 7

RESULTS AND ANALYSIS

7.1Damping ratio

Damping ratio of the frame model, $\xi = 0.35$ %

7.2 Analytical results

Analytical analysis is carried out with or without mass variation.

7.2.1 without mass variation

In this case no additional mass is considered on different floor level. The results of analytical analysis are presented in table 7.1 and the mode shapes are plotted in the fig. 7.1.

Table 7.1 Frequencies, time period and mode shapes from analytical analysis without mass variation

Modes	1	2	3
Angular frequency (rad/sec)	17.33	48.18	68.90
Frequency (Hz)	2.76	7.67	10.97
Time Period (sec)	0.36	0.13	0.09
1st floor	1.00	1.00	1.00
2nd floor	1.79	0.39	-1.28
3rd floor	2.21	-0.84	0.64

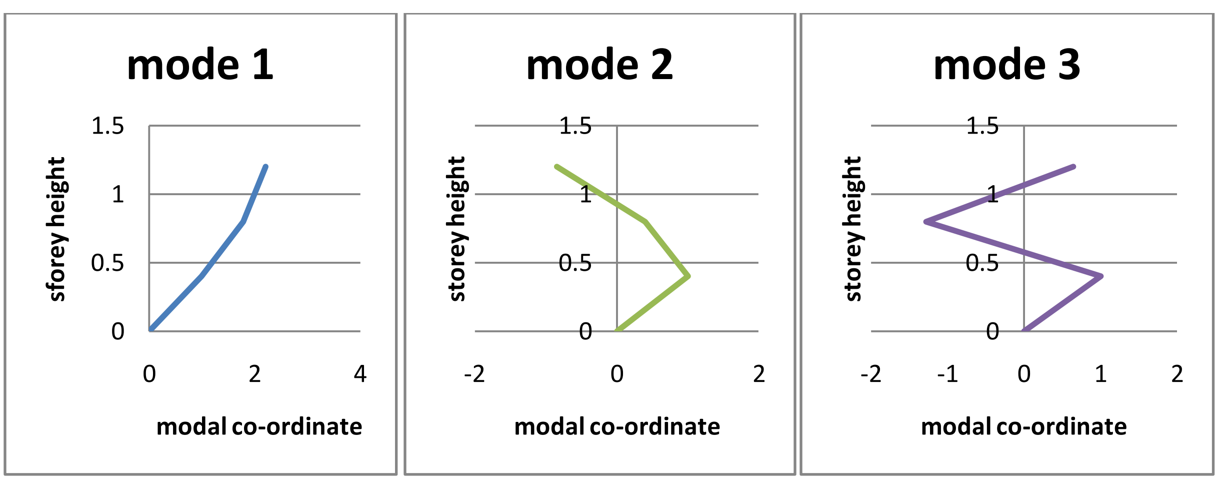

Fig. 7.1 mode shape from analytical analysis without mass variation

7.2.2 with additional mass on different floor level

In this section mass is added at different floor level. Following cases are adopted:

Case 1: In this case additional mass of 2 kg is considered on first floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.2. The mode shapes are plotted in fig. 7.2.

Table 7.2 Frequencies, time period and mode shapes from analytical analysis with additional mass on 1st floor

Modes	1	2	3
Angular frequency (rad/sec)	16.25	38.12	64.41
Frequency (Hz)	2.59	6.07	10.25
Time Period (sec)	0.38	0.17	0.10
1st floor	1.00	1.00	1.00
2nd floor	1.62	-0.08	-3.95
3rd floor	1.94	-1.08	-4.44

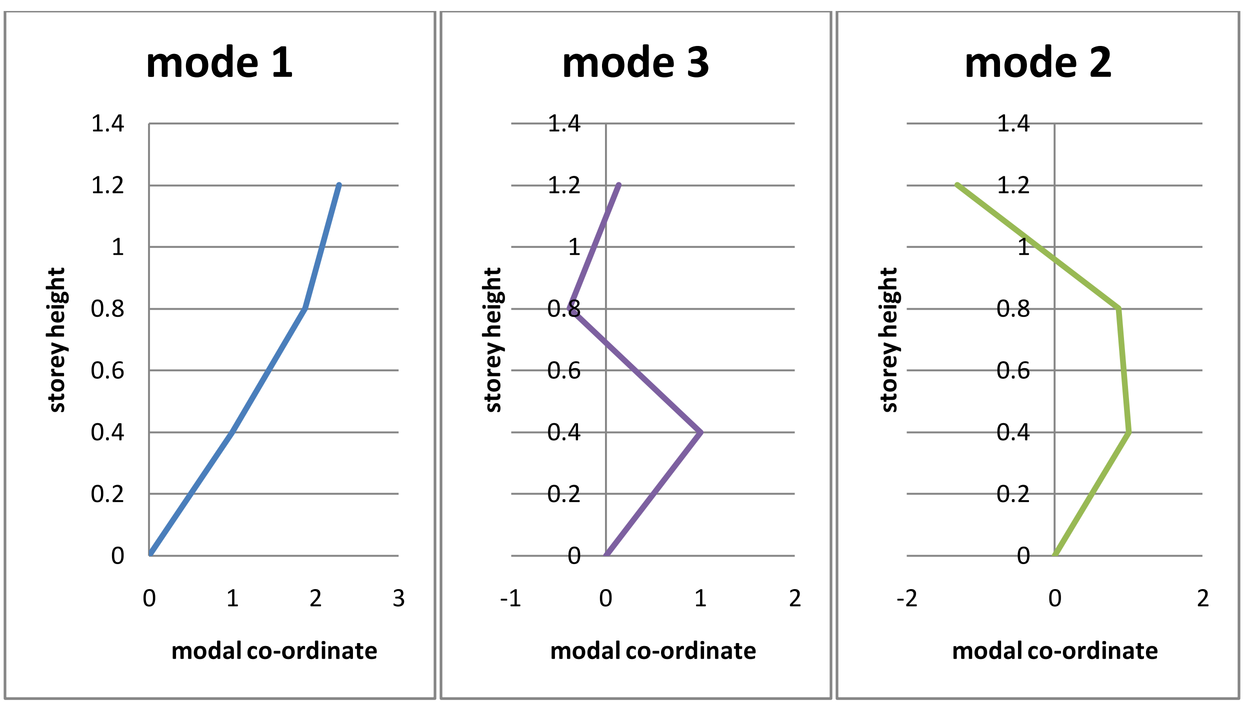

Fig. 7.2 mode shape from analytical analysis with additional mass on 1st floor

Case 2: In this case additional mass of 2 kg is considered on second floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.3. The mode shapes are plotted in fig. 7.3.

Table 7.3 Frequencies, time period and mode shapes from analytical analysis with additional mass on 2nd floor

Modes	1	2	3
Angular frequency (rad/sec)	14.57	46.05	59.43
Frequency (Hz)	2.32	7.34	9.46
Time Period (sec)	0.43	0.14	0.11
1st floor	1.00	1.00	1.00
2nd floor	1.85	0.53	-0.44
3rd floor	2.14	-1.55	0.35

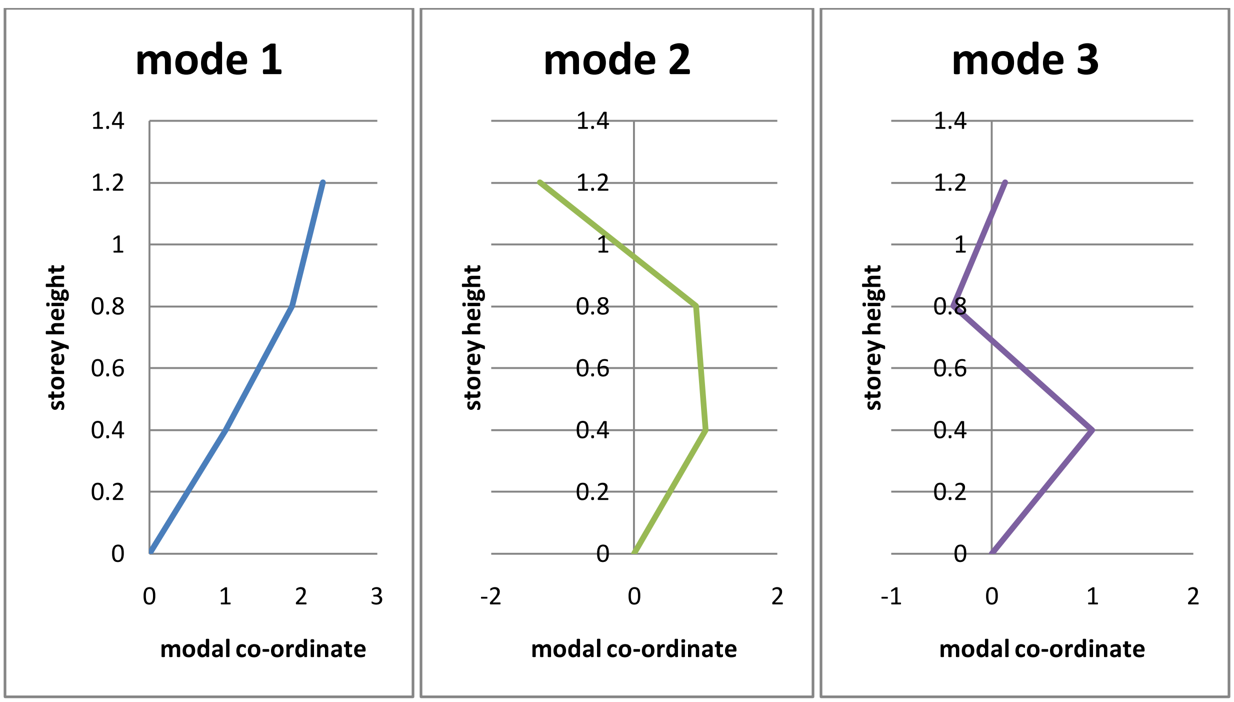

Fig 7.3 mode shape from analytical analysis with additional mass on 2nd floor

Case 3: In this case additional mass of 2 kg is considered on third floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.4. The mode shapes are plotted in fig. 7.4.

Table 7.4 Frequencies, time period and mode shapes from analytical analysis with additional mass on 3rd floor

Modes	1	2	3
Angular frequency (rad/sec)	13.55	42.92	66.99
Frequency (Hz)	2.16	6.83	10.66
Time Period (sec)	0.46	0.15	0.09
1st floor	1.00	1.00	1.00
2nd floor	1.87	0.73	-1.10
3rd floor	2.51	-0.47	0.21

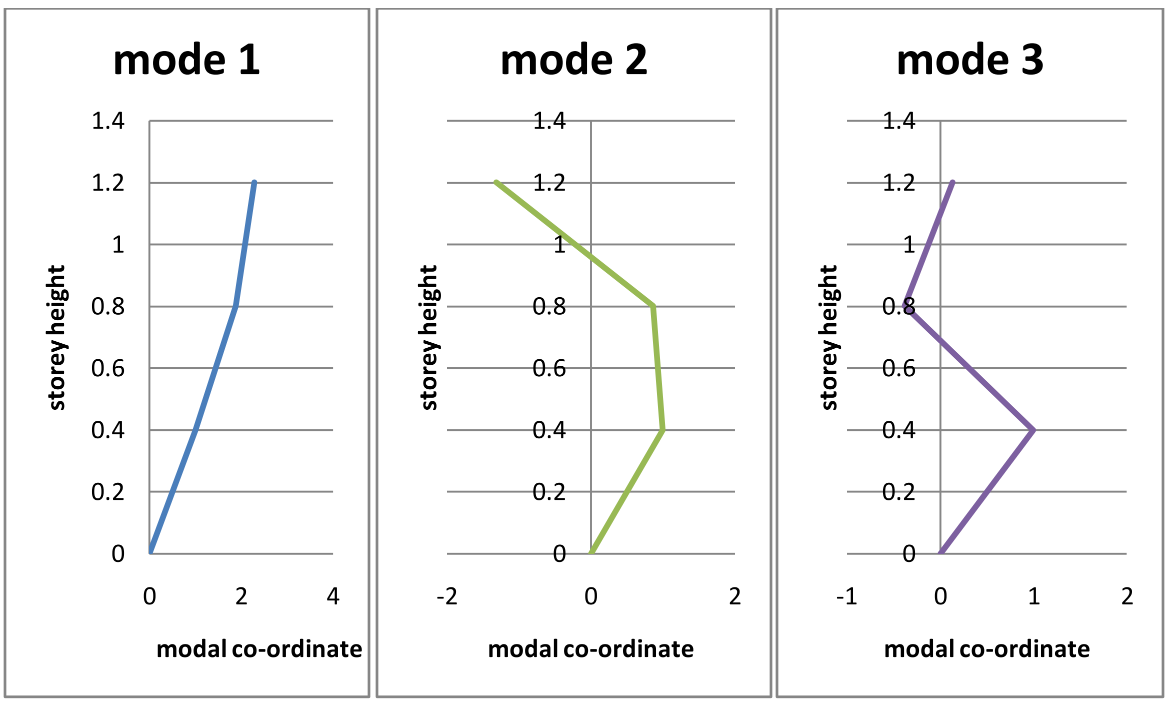

Fig 7.4 mode shape from analytical analysis with additional mass on 3rd floor

Case 4: In this case additional mass of 2 kg is considered on first floor and mass of 1 kg is considered on second floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.5. The mode shapes are plotted in fig. 7.5.

Table 7.5 Frequencies, time period and mode shapes from analytical analysis with additional mass on 1st floor and 2nd floor

Modes	1	2	3
Angular frequency (rad/sec)	14.97	30.09	56.35
Frequency (Hz)	2.38	6.06	8.97
Time Period (sec)	0.42	0.16	0.11
1st floor	1.00	1.00	1.00
2nd floor	1.68	-0.03	-2.45
3rd floor	1.97	-1.02	-2.19

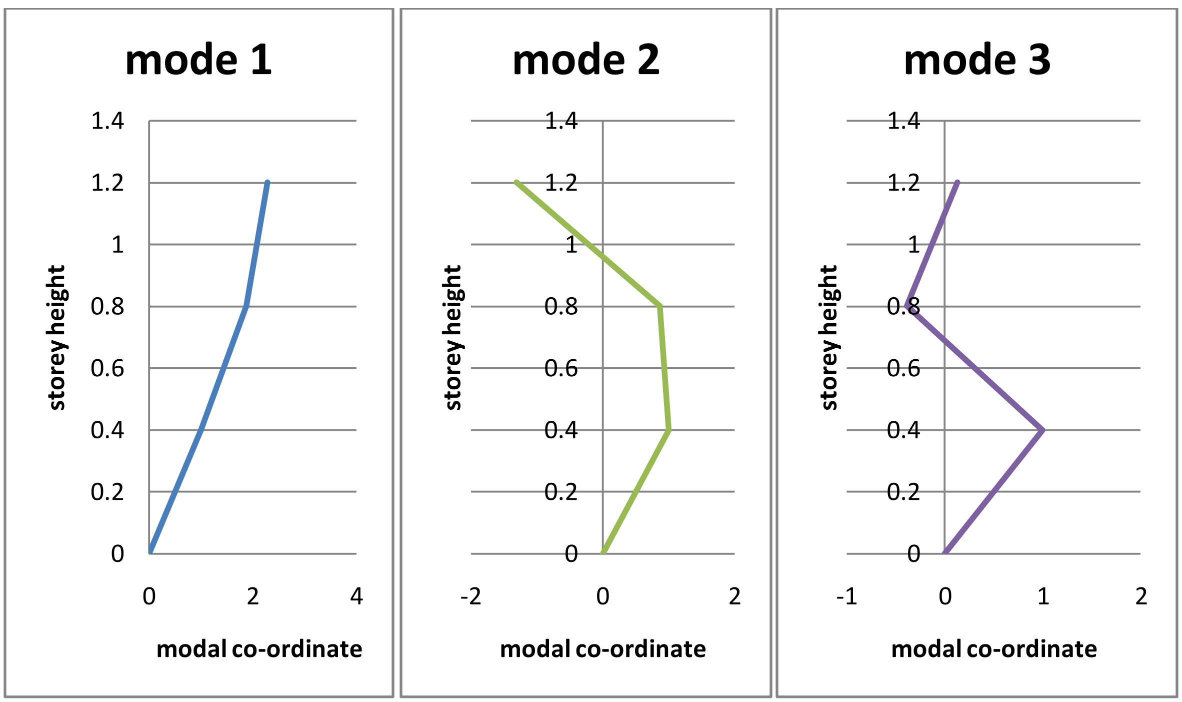

Fig .7.5 mode shape from analytical analysis with additional mass on 1st floor and on 2nd floor

Case 5: In this case additional mass of 2 kg is considered on second floor and mass of 1 kg is considered on third floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.6. The mode shapes are plotted in fig. 7.6.

Table 7.6 Frequencies, time period and mode shapes from analytical analysis with additional mass on 2st floor and 3rd floor

Modes	**1**	**2**	**3**
Angular frequency (rad/sec)	13.26	40.57	58.80
Frequency (Hz)	2.11	6.45	9.36
Time Period (sec)	0.47	0.15	0.11
1st floor	1.00	1.00	1.00
2nd floor	1.87	0.86	-0.38
3rd floor	2.28	-1.31	0.13

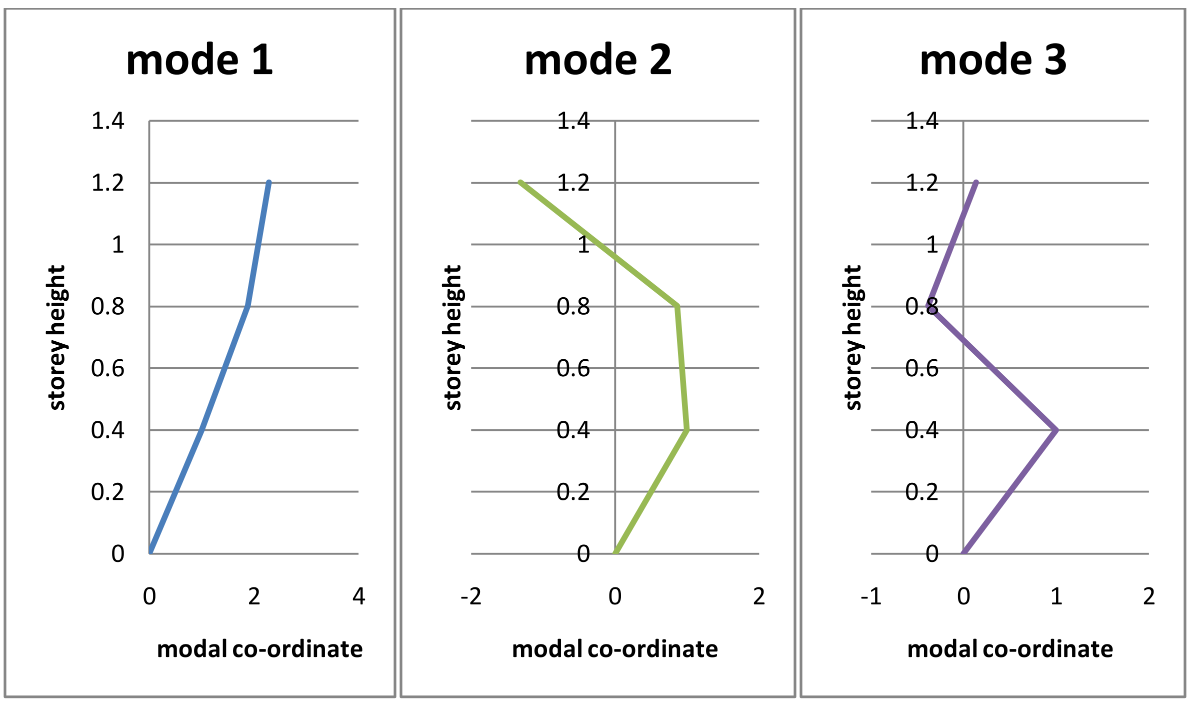

Fig. 7.6 mode shape from analytical analysis with additional mass on 2nd floor and 3rd floor

7.3 Experimental Analysis

Experimental analysis is carried out with or without mass variation.

7.3.1 without mass variation

In this section no additional mass is putted on different floor level. The results of experimental analysis are presented in table 7.7 and the mode shapes are plotted in the fig. 7.7.

Table 7.7 Frequencies, time period and mode shapes from experimental analysis without mass variation

Modes	1	2	3
Angular frequency (rad/sec)	16.96	49.01	78.00
Frequency (Hz)	2.70	7.80	11.30
Time Period (sec)	0.37	0.13	0.09
1^{st} floor	1.00	1.00	1.00
2^{nd} floor	1.80	-0.88	-1.48
3^{rd} floor	2.24	0.34	1.21

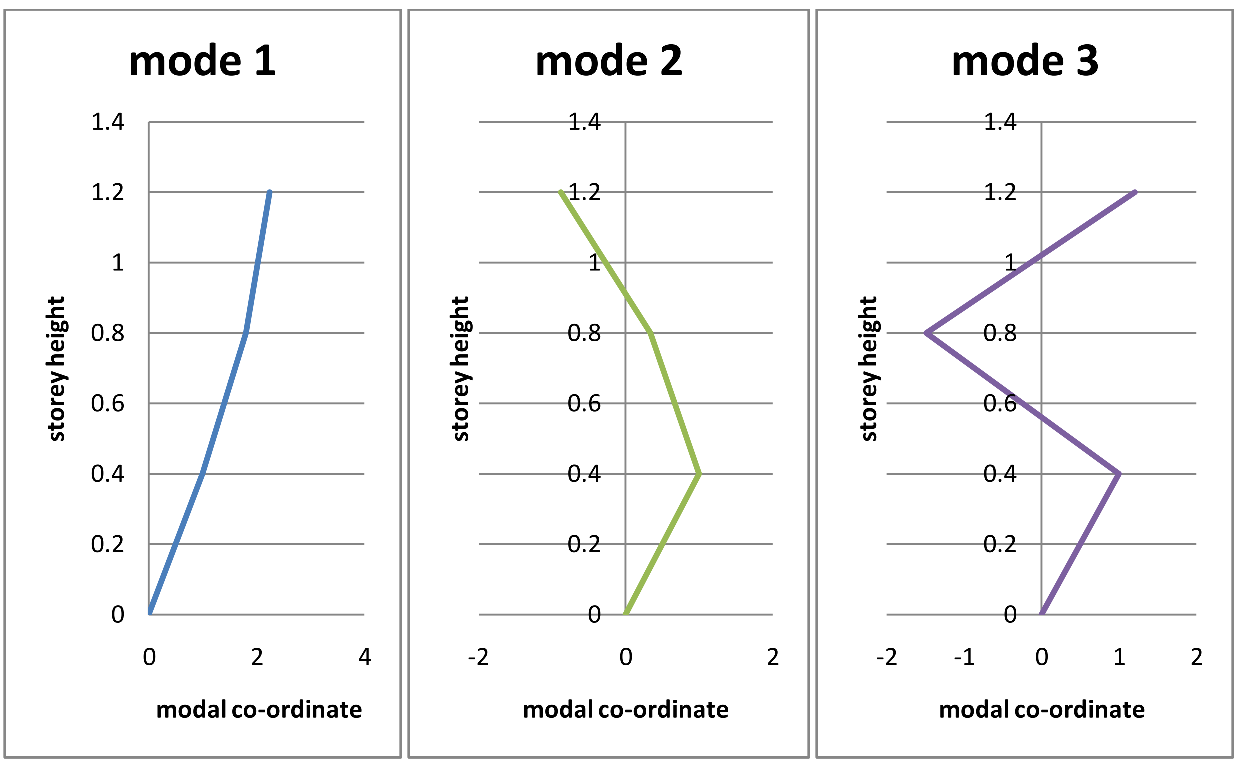

Fig 7.7 mode shape from experimental analysis without mass variation

7.3.2 with mass variation

In this section mass is putted at different floor level. Following cases are adopted:

Case 1: In this case additional mass of 2 kg is putted on first floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.8. The mode shapes are plotted in fig. 7.8.

Table 7.8 Frequencies, time period and mode shapes from experimental analysis with additional mass on 1st floor

Modes	1	2	3
Angular frequency (rad/sec)	16.34	36.44	69.74
Frequency (Hz)	2.60	5.80	11.10
Time Period (sec)	0.38	0.17	0.09
1st floor	1.00	1.00	1.00
2nd floor	1.62	0.09	-4.98
3rd floor	1.93	-0.89	5.78

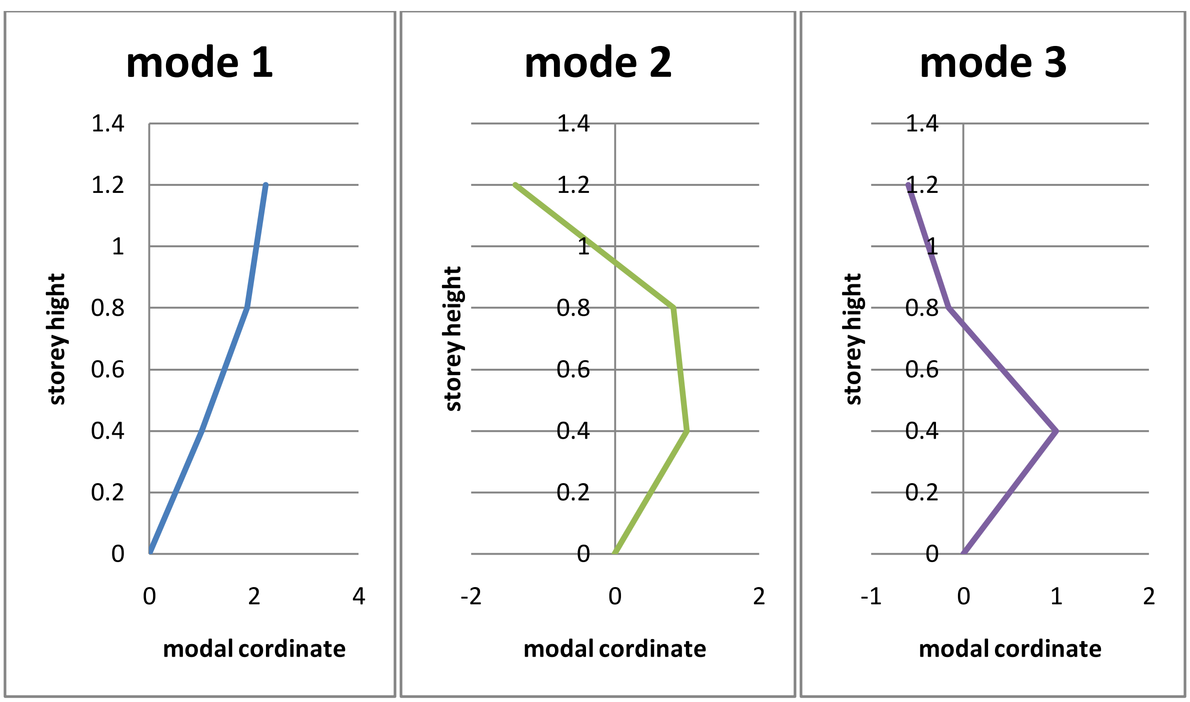

Fig.7.8 mode shape from experimental analysis with additional mass on 1st floor

Case 2: In this case additional mass of 2 kg is putted on second floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.9. The mode shapes are plotted in fig. 7.9.

Table 7.9 Frequencies, time period and mode shapes from experimental analysis with additional mass on 2nd floor

Modes	1	2	3
Angular frequency (rad/sec)	15.08	44.61	57.81
Frequency (Hz)	2.40	7.10	9.20
Time Period (sec)	0.42	0.14	0.11
1st floor	1.00	1.00	1.00
2nd floor	1.84	0.62	-0.31
3rd floor	2.08	-1.53	-0.13

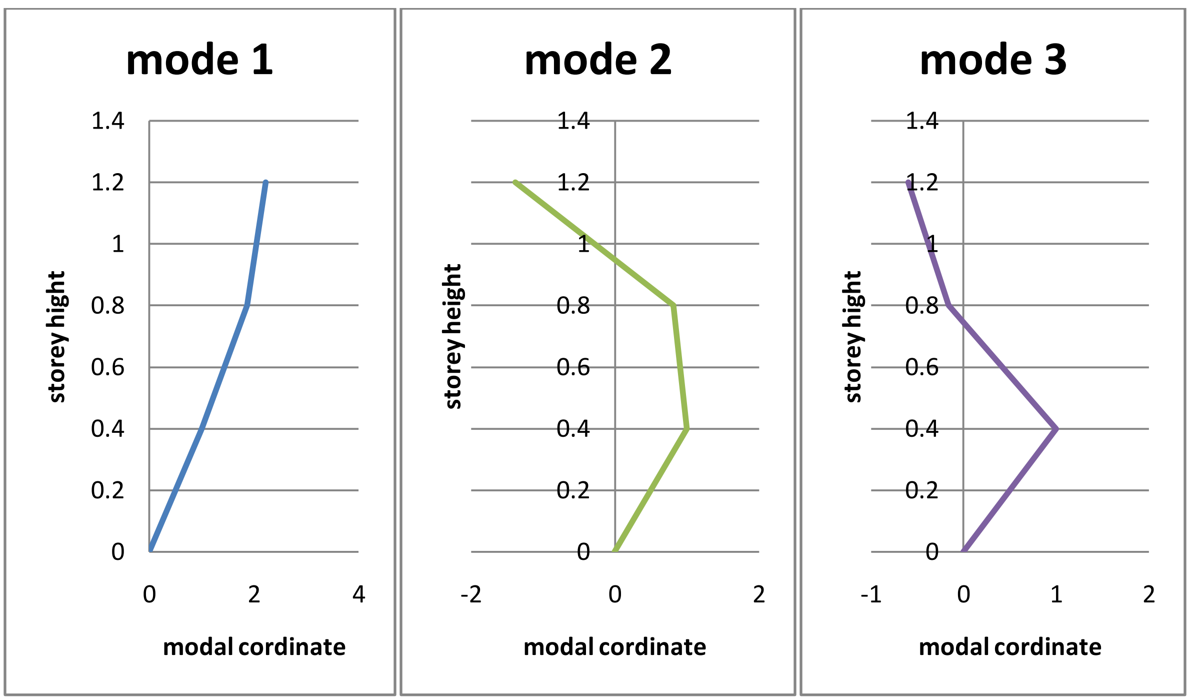

Fig. 7.9 mode shape from experimental analysis with additional mass on 2nd floor

Case 3: In this case additional mass of 2 kg is putted on third floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.10. The mode shapes are plotted in fig. 7.10.

Table 7.10 Frequencies, time period and mode shapes from experimental analysis with additional mass on 3rd floor

Modes	1	2	3
Angular frequency (rad/sec)	13.82	43.35	69.74
Frequency (Hz)	2.20	6.90	11.10
Time Period (sec)	0.45	0.14	0.09
1st floor	1.00	1.00	1.00
2nd floor	1.86	0.70	-1.36
3rd floor	2.28	-0.50	0.85

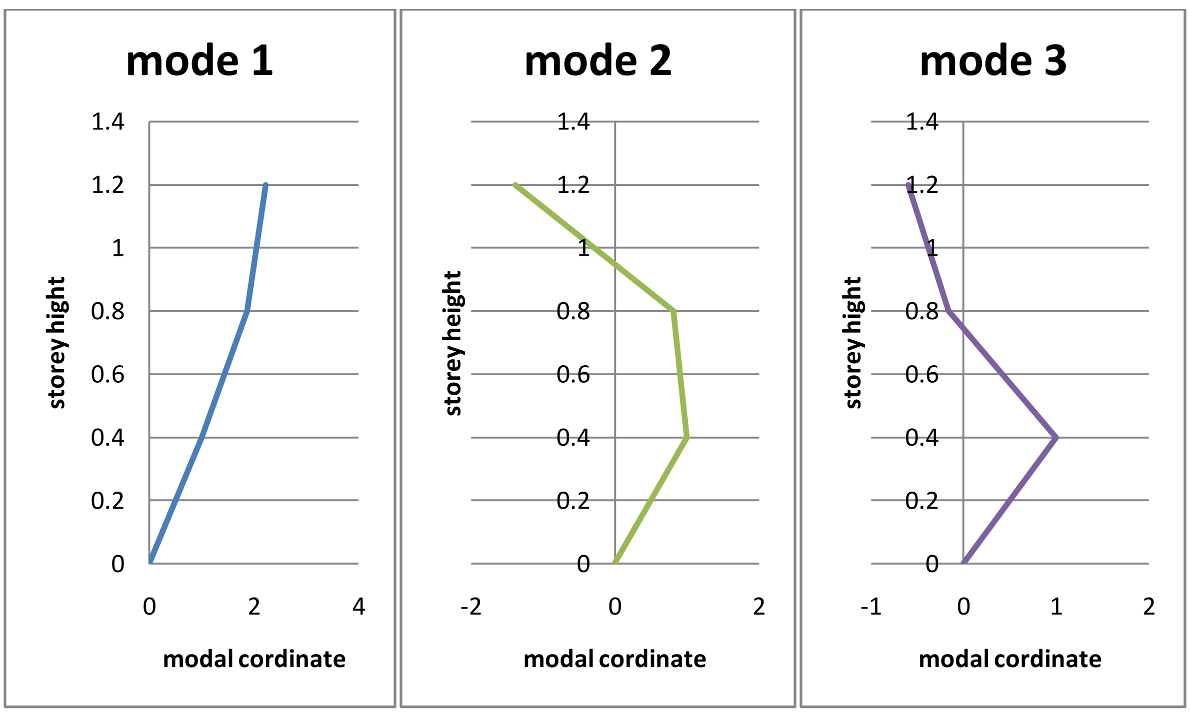

Fig. 7.10 mode shape from experimental analysis with additional mass on 3rd floor

Case 4: In this case additional mass of 2 kg is putted on first floor and mass of 1 kg is putted on second floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.11. The mode shapes are plotted in fig. 7.11.

Table 7.11 Frequencies, time period and mode shapes from experimental analysis with additional mass on 1st floor and 2nd floor

Modes	1	2	3
Angular frequency (rad/sec)	15.71	40.84	57.81
Frequency (Hz)	2.50	6.5	9.20
Time Period (sec)	0.40	0.15	0.12
1st floor	1.00	1.00	1.00
2nd floor	1.65	-0.34	-2.68
3rd floor	1.88	-1.09	2.96

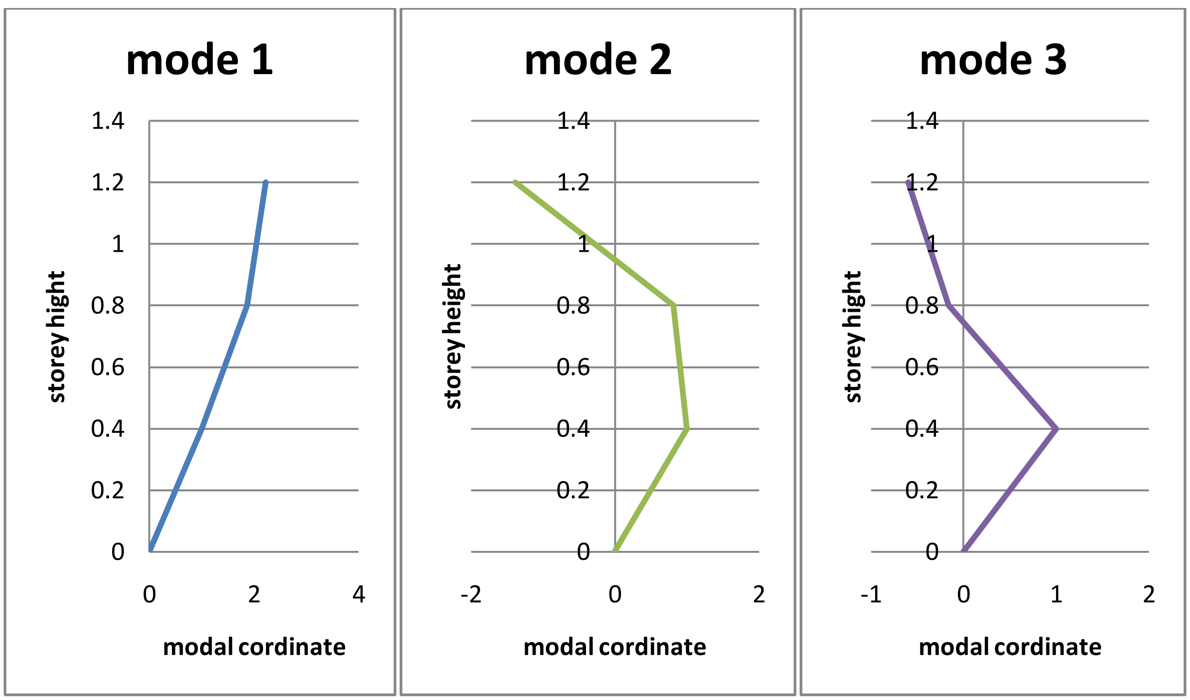

Fig.7.11 mode shape from experimental analysis with additional mass on 1st floor and 2nd floor

Case 5: In this case additional mass of 2 kg is putted on second floor and mass of 1 kg is putted on third floor. The frequencies, time periods and mode shapes are calculated. The results are presented in table 7.12. The mode shapes are plotted in fig. 7.12.

Table 7.12 Frequencies, time period and mode shapes from experimental analysis with additional mass on 2nd floor and 3rd floor

Modes	1	2	3
Angular frequency (rad/sec)	13.82	41.47	55.92
Frequency (Hz)	2.20	6.60	8.90
Time Period (sec)	0.45	0.15	0.11
1st floor	1.00	1.00	1.00
2nd floor	1.87	0.81	-0.16
3rd floor	2.22	-1.38	-0.59

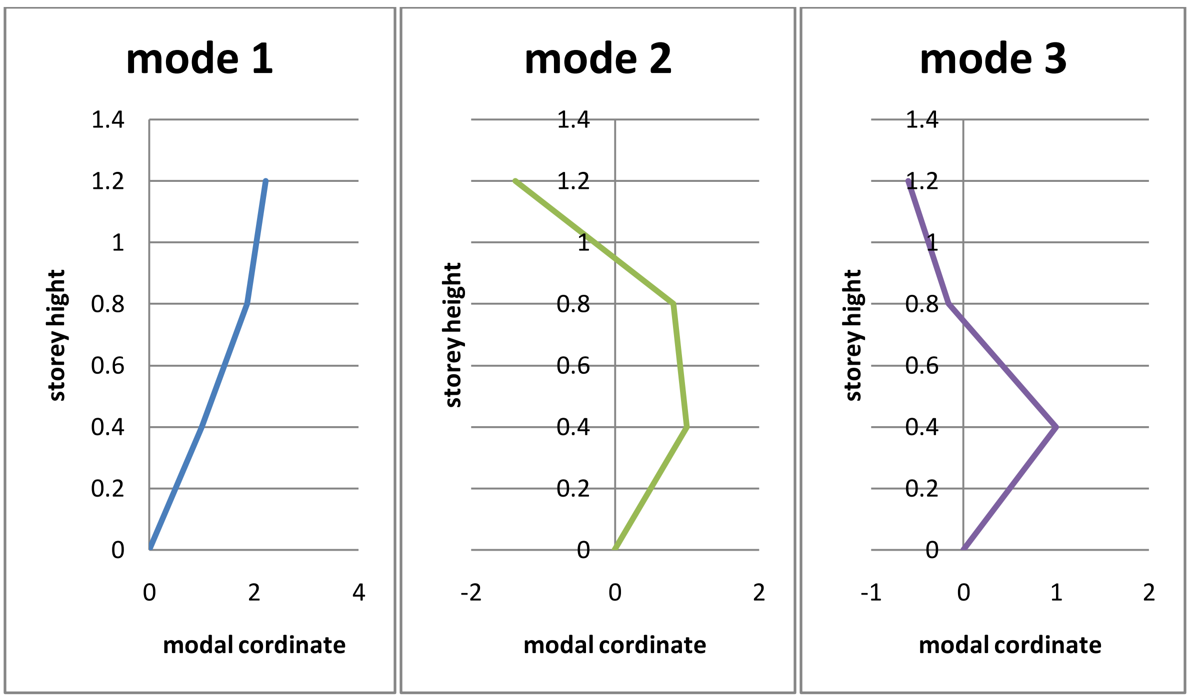

Fig 7.12 mode shape from experimental analysis with additional mass on 2nd floor and on 3rd floor

7.4 Comparison of results

The frequencies and time periods obtained from analytical, experimental and Etabs software are compared.

7.4.1 without mass variation

The results are presented in table 7.13.

Table 7.13 Frequencies, and time period without mass variation

Mode	1			2			3		
	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs
Angular Frequency (rad/sec)	17.33	16.96	13.78	48.18	49.01	48.33	68.90	78.00	69.04
Frequency (Hz)	2.76	2.70	2.76	7.67	7.80	7.69	10.97	11.30	10.99
Time Period (sec)	0.36	0.37	0.36	0.13	0.13	0.13	0.09	0.09	0.09

7.4.2 with mass variation

In this section following cases are adopted:

Case 1: additional load on first floor

In this case additional load of 2 kg is considered on first floor. The frequency and time period are presented in table 7.14.

Table 7.14 Frequencies, and time period additional mass on 1st floor

Mode	1			2			3		
	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs
Angular Frequency (rad/sec)	16.25	16.34	17.31	38.12	36.44	47.60	64.41	69.74	68.29
Frequency (Hz)	2.59	2.60	2.77	6.07	5.80	7.58	10.25	11.10	10.87
Time Period (sec)	0.39	0.39	0.36	0.17	0.17	0.13	0.10	0.09	0.09

Case 2: additional load on second floor

In this case additional load of 2 kg is considered on second floor. The frequency and time period are presented in table 7.15.

Table 7.15 Frequencies, and time period with additional mass on 2nd floor

Mode	1			2			3		
	Analytical	**Experimental**	**Etabs**	**Analytical**	**Experimental**	**Etabs**	**Analytical**	**Experimental**	**Etabs**
Angular Frequency (rad/sec)	14.57	15.08	17.21	46.05	44.61	0.13	59.43	57.81	68.30
Frequency (Hz)	2.32	2.40	2.74	7.34	7.10	7.69	9.46	9.20	10.87
Time Period (sec)	0.43	0.42	0.37	0.14	0.14	48.33	0.11	0.11	0.09

Case 3: additional load on third floor

In this case additional load of 2 kg is considered on third floor. The frequency and time period are presented in table 7.16.

Table 7.16 Frequencies, and time period with additional mass on 3rd floor

Mode	1			2			3		
	Analytical	**Experimental**	**Etabs**	**Analytical**	**Experimental**	**Etabs**	**Analytical**	**Experimental**	**Etabs**
Angular Frequency (rad/sec)	13.55	13.82	17.12	42.92	43.35	48.33	66.99	69.74	69.05
Frequency (Hz)	2.16	2.20	2.73	6.83	6.90	7.69	10.66	11.10	10.99
Time Period (sec)	0.46	0.45	0.37	0.15	0.15	0.13	0.09	0.09	0.09

Case 4: additional load on first floor and on second floor

In this case additional load of 2 kg is considered on first floor and 1 kg on second floor. The frequency and time period are presented in table 7.17.

Table 7.17 Frequencies, and time period with additional mass on 1st floor and 2nd floor

Mode	1			2			3		
	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs
Angular Frequency (rad/sec)	14.97	15.71	17.26	30.09	40.84	47.96	56.35	57.81	68.29
Frequency (Hz)	2.38	2.50	2.75	6.06	6.5	7.63	8.97	9.20	10.87
Time Period (sec)	0.42	0.40	0.36	0.16	0.15	0.13	0.11	0.11	0.09

Case 5: additional load on second floor and on third floor

In this case additional load of 2 kg is considered on second floor and third floor. The frequency and time period are presented in table 7.18.

Table 7.18 Frequencies, and time period with additional mass on 2nd floor and 3rd floor

Mode	1			2			3		
	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs	Analytical	Experimental	Etabs
Angular Frequency (rad/sec)	13.26	13.82	17.12	40.57	41.47	48.33	58.80	55.92	68.29
Frequency (Hz)	2.11	2.20	2.76	6.45	6.60	7.69	9.36	8.90	10.67
Time Period (sec)	0.47	0.45	0.37	0.16	0.15	0.13	0.11	0.11	0.09

CHAPTER 8

CONCLUSIONS

8.1 Conclusions

Following are the conclusions:

1. The study shows that mass irregularities in a frame may affect the behavior of the frame differently in a seismic event depending on its locations. This aspect does not seem to be clear in standard codes of practices.
2. The eigenvalues analysis gives quite accurate results for calculation of time periods of framed structures. The experimental values come quite close to calculated values.
3. Floor displacement corresponding to guiding frequencies without mass variation in the frame has been found to be connected with frequency of excitation.
4. It seems that maximum displacement in case of presence of mass irregularity decreases. It may be because of the fact that mass used for creating mass eccentricity increases the total mass of the frames and consequently the stiffness. But this is not entirely helpful for the frame during seismic event because the increased stiffness is concentrated near to the locations of mass eccentricity and the frame may not act as a regular frame.
5. Nature grasp showing floor displacement and frequency seems to be uniform even when mass eccentricity is created at different locations.
6. It seems that maximum floor displacement occurs near to some frequency ranges and these values drops to very small values in between these ranges. It may indicate presence of some conflicting effects in between these ranges.
7. Floor displacement and frequency graphs tend to becomes more disorganized when mass eccentricity is created towards the top portion of the frames.
8. Mass eccentricity created near to top portion of the frame seems to affect response related parameters more than the mass eccentricity is created towards the base of the frames.
9. The Etabs model of the frame provides a good co-relation with the experimental results.
10. Differences in the experimental results and results of the mode created in Etabs may be because of differences in joints. This may be because of different types of fixity conditions at the location of joints.

8.2 Future scope of study

Future scope of study may consist of the following aspects of the study:

1. To consider effect of mass irregularity on the response of various types of RC buildings frames and other types of buildings such as masonry buildings.
2. To consider various other types of irregularities and the comparisons undertaken to fix up importance.
3. To consider undertaking experimental programme for various types of buildings with the help of appropriate types structural models.
4. To develop ways and means to fined methodology giving a true correspondence between actual buildings, experimental models and theoretical models.
5. To consider all the above points for different types of earthquake under different seismic zones of the country.
6. To undertake development of presentation, academic resource materials, user friendly solution for analysis and design of various types of buildings.
7. To consider new types of construction patterns and practices and their effects and incorporation of resulting knowledge for the revision and upgradation of codes of practices.

REFERENCES

1. Tso, W.K., and Sadek, A.W. (1989), “strength eccentricity concept for inelastic analysis of asymmetrical structures”, Engineering Structures, Vol. 11, No.3, pp.189-194.
2. Tso, W.K., and Bozorgnia, Y. (1986). “Effective eccentricity for inelastic seismic response of buildings”, Earthquake engineering and Structural Dynamics, Vol. 14, No.3, pp. 413-427.
3. Tso, W.K., and Dempsey, K.m. (1980), “Seismic torsional provisions for dynamic eccentricity”, Earthquake Engineering and Structural Dynamics, Vol. 8, No.3, pp.275-289.
4. Sarkar P., Prasad A. Meher, Menon Devdas, 2010, Vertical geometric irregularity in stepped building frames, Engineering Structures, 32(2010) 2175-2182.
5. Karavallis, Bazeos and Beskos, (2008) Estimation of seismic inelastic deformation demands in plane steeMRF with vertical mass irregularities, Engineering Structures 30 (2008) 3265-3275.
6. Karavasils, T.L. and Bazeos, N. and Beskos, D.E. (2008). “Seismic response of plane steel frame MRF with setbacks: Estimation of inelastic deformation demands”, Journal of Construction and steel research, Vol.64, pp.644-654.
7. Poonam, Kumar Anil and Gupta Ashok K, (2012), “Study of response of structural irregular building frames to seismic excitations, International Journal of Civil, Structural, Environmental and Infrastructure Engineering Research and Development (IJCSEIERD), ISSN 2249-6866 Vo.2, Issue 2 (2012) 25-31.
8. Moghadam, A.S., AND Aziminejad, A. (2005), “ Interaction of torsion and P-Delta effects in tall buildings”, In proceeding of Thirteenth World Conference on Earthquake Engineering Vancouver, B.C., Canada August 1-6, 2004, Paper No. 799.
9. Humar, J.L., and Wright, E.W. (1977), “Earthquake response of steel frame multistory buildings with setbacks”, Earthquake Engineering and Structural Dynamics, Vol. 5, No. 1, pp. 15-39.
10. Aranda, G.R. (1994). “Ductility demands for RC frames irregular in elevation”, in proceedings of the eighth world conference on Earthquake Engineering , San Francisco, U.S.A., Vol. pp. 559-566.

11. Fernadez, J. (1983) “Earthquake response analysis of buildings considering the effect of structural configuration, Bulletin of the International Institute of Seismology and Earthquake Engineering (Tokyo, Japan, Nov 1983), Vol, 19 pp.203-215.
12. Moehle, J.P.(1984), “Seismic response of vertical irregular structures”, Journal of Structural Engineering, ASCE, Vol. 110, pp. 2002-2014.
13. Moehle, J.P. and Alarcon, L.F. (1986), “Seismic analysis methods for irregular buildings”, Journal of Structural Engineering, ASCE, Vol. 112, No. 1, pp. 35-52.
14. Bariola, V. and Brokken, S. (1991), “ Influence of strength and stiffness on seismic structural behavior”, Bulletin of Seismology and Earthquake Engineering, Vol.23 pp.427-434.
15. Ruiz, S.E. and Diederich, R. (1989). (The Mexico Earthquake of September 19, 1985-the seismic performance of buildings with weak first storey”, Earthquake Spectra Vol. 5, No. 1, pp. 89-102.
16. Shahrooz, B.M. and Moehle, J.P. (1990), “Seismic response and Design of setback buildings”, Journal of structural engineering, ASCE, Vol. 116, No. 5 pp. 1423-1439.
17. Nassar, A.A. and Krawinkler, H. (1991), “seismic demands for SDOF and MDOF systems”, Report No. 95, The John A. Blume Earthquake Engineering Center, Department of civil engineering AND Environmental engineering, Stanford University, Stanford, U.S.A.
18. Esteva, L. (1992), Nonlinear seismic response of soft first storey buildings subjected to narrow band accelerograms”, Earthquake Sprectra, Vol. 8, pp. 373-389.
19. Wong, C.M. and Tso, W.K. (1994), “Seismic loading for buildings with setbacks”, Canadian Journal of Civil Engineering, Vol. 21, No. 5, pp. 863-871.
20. Daun, X.N. and Chandler, A.M. (1995), “Seismic torsional response and design procedures for a class of setback frame buildings”, Earthquake Engineering and Structural Dynamics, Vol.24, pp. 761-777.
21. Valmundsson, E.V. and Nau, J.M. (1997). “ Seismic response of building frames with vertical structural irregularities”, Journal of Structural Engineering, ASCE, Vol. 123, No. 1, pp. 30-41.
22. Al-Ali, A.A.K. and Krawinkler, H. (1998) “Effects of vertical irregularities on seismic behavior of building structures”, Report No. 130, 1998, The John A. Blume Earthquake

Engineering Center, Department of Civil and Environmental Engineering, Stanford University, Stanfor, U.S.A.

23. Kappos, A.J. A.J. and Scott, .G. (1998) “Seismic assessment of an R/C building with setbacks using nonlinear static and dynamic analysis procedures”, In Booth ED (ed) seismic design practice into the next century. Balkema, Rotterdam.
24. Mwafy, A.M. and Elnashai, A.S (2002), “Calibration of force reduction factors of RC buildings:, Journal of earthquake engineering, Vol. 6, N. 2, pp.293.273
25. Das, S. and Nau, J.M. (2003),”Seismic design aspects of vertically irregular reinforced concrete buildings”, Earthquake Spectra, Vol. 19, No. 3, pp. 455-477.
26. Chintanapakdee, C. and Chopra, A.K. (2004),”Seismic response of vertically irregular frames: response history and modal pushover analysis”, Journal of Structural Engineering, ASCE, Vol. 130 No. 8, pp. 1177-1185.
27. Tremblay, R. and Poncet, L. (2005), “Seismic performance of concentrically braced steel frames in multistory buildings with mass irregularity”, Journal of Structural Engineering, Vol. 131, pp. 1363-1375.
28. Ayidin, K. (2007). “Evaluation of Turkish seismic code for mass irregular buildings”, Indian Journal of engineering and material sciences, Vol.14, pp. 220-234.
29. IS 1893 (Part 1): 2002, “Criteria For Earthquake resistant Design of Structures”, General provisions and buildings (fifth revision).
30. A.K. Chopra, “Dynamics of Structures”, Book.
31. C S Manohar and S Venkatesha, (2006) “Development of experimental setups for earthquake engineering education”, Report, National Program on Earthquake Engineering Education, MHRD, Govt. of India.