DYNAMIC ANALYSIS OF SLOPED BUILDINGS:

EXPERIMENTAL AND NUMERICAL STUDIES

A Thesis

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In partial fulfilment of the requirements for

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NATIONAL INSTITUTE OF TECHNOLOGY
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CERTIFICATE

This is to certify that the thesis entitled "**DYNAMIC ANALYSIS OF SLOPED BUILDINGS: EXPERIMENTAL AND NUMERICAL STUDIES**" *submitted by* **Sandeep Goyal** bearing Roll No. 213CE2062 *in partial fulfilment of the requirements for the award of* **Master of Technology** *Degree in* **Civil Engineering** *with specialization in* **Structural Engineering** *during* **2013-2015** *session to the* **National Institute of Technology Rourkela** is an authentic work carried out by him under my supervision and guidance. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any Degree or Diploma.

Date

Prof. K. C. Biswal
Department of Civil Engineering
NIT, Rourkela

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Dedicated

To

MY BELOVED PARENTS

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Abstract

The buildings situated in hilly areas are much more prone to seismic environment in comparison to the buildings that are located in flat regions. Structures on slopes differ from other buildings since they are irregular both vertically and horizontally hence torsionally coupled and are susceptible to severe damage when subjected to seismic action. The columns of ground storey have varying height of columns due to sloping ground. In this study, behaviour of two storied sloped frame having step back configuration is analyzed for sinusoidal ground motion with different slope angles i.e., 15°, 20° and 25° with an experimental set up and are validated by developing a Finite Element code executed in MATLAB platform and using structural analysis tool STAAD Pro. by performing a linear time history analysis. From the above analysis, it has been observed that as the slope angle increases, stiffness of the model increases due to decrease in height of short column and that results in increase of earthquake forces on short column which is about 75% of total base shear and chances of damage is increased considerably due to the formation of plastic hinges therefore proper analysis is required to quantify the effects of various ground slopes.

Keywords: Ground Motion, linear time history analysis, frequency content, finite element code

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Chapter 1

INTRODUCTION

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INTRODUCTION

1.1 Introduction

Earthquake is the most disastrous and unpredictable phenomenon of nature. When a structure is subjected to seismic forces it does not cause loss to human lives directly but due to the damage cause to the structures that leads to the collapse of the building and hence to the occupants and the property. Mass destruction of the low and high rise buildings in the recent earthquakes leads to the need of investigation especially in a developing country like India. Structure subjected to seismic/earthquake forces are always vulnerable to damage and if it occurs on a sloped building as on hills which is at some inclination to the ground the chances of damage increases much more due to increased lateral forces on short columns on uphill side and thus leads to the formation of plastic hinges. Structures on slopes differ from those on plains because they are irregular horizontally as well as vertically. In north and north-eastern parts of India have large scale of hilly terrain which fall in the category of seismic zone IV and V. Recently Sikkim (2011), Doda (2013) and Nepal earthquake (2015) caused huge destruction. In this region there is a demand of construction of multistory RC framed buildings due to the rapid urbanization and increase in economic growth and therefore increase in population density. Due to the scarcity of the plain terrain in this region there is an obligation of the construction of the buildings on the sloping ground.
In present work, a two storeyed framed building with an inclination of 15°, 20° and 25° to the ground subjected to sinusoidal ground motion is modelled with an experimental setup and validated with a finite element coding executed in the MATLAB platform and results obtained are validated by performing linear time history analysis in structural analysis and design software (STAAD Pro.).

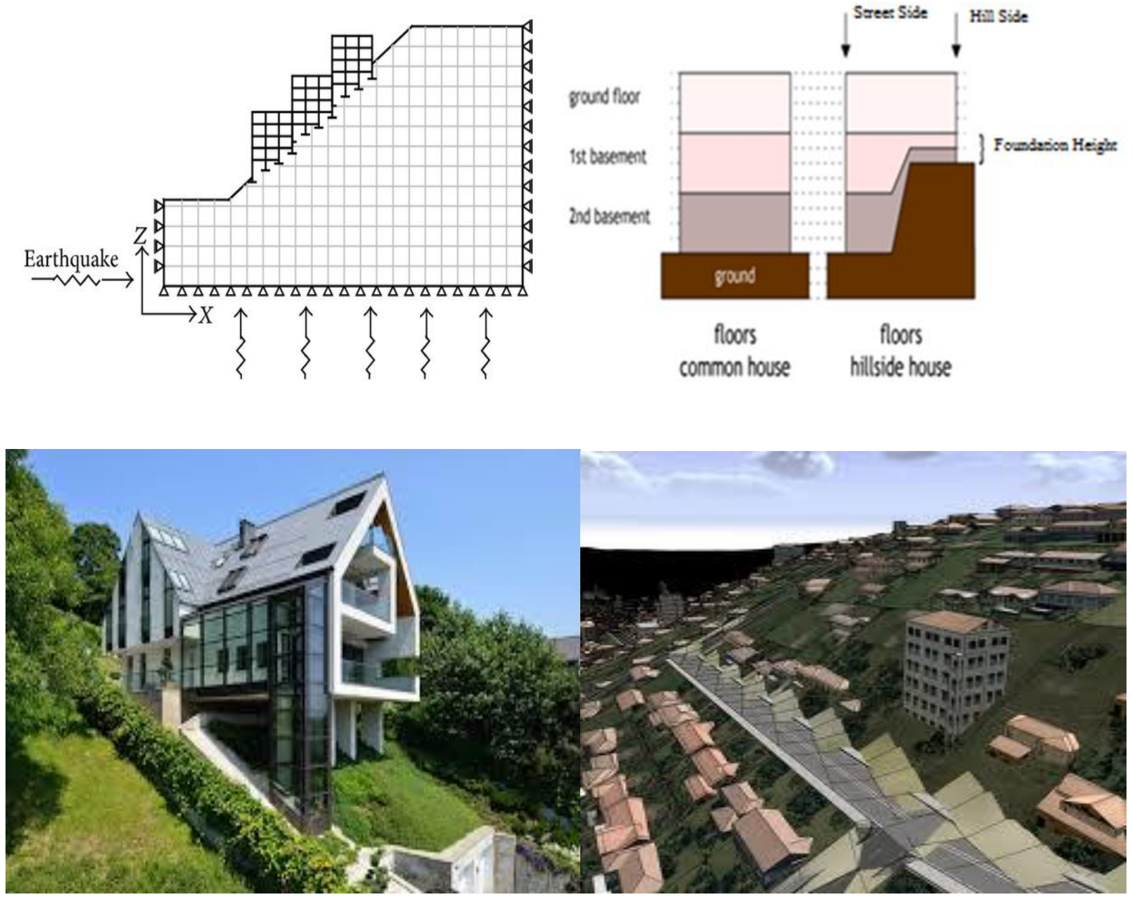

Figure 1: Buildings on sloping ground

1.2 Origin of the Project

Few research works is carried out on the seismic behaviour of structures on slopes subjected to ground motion of sinusoidal nature. Sreerama and Ramancharla (2013) studied numerically the effect on seismic behaviour on varying slope angle and compared with the same on flat ground. No work is carried out regarding the seismic behaviour of the structures on sloping ground with an experimental set up.

1.3 Research Significance

India consists of great arc of mountains which consists of Himalayas in its northern part which was formed by on-going tectonic collision of plates. In this region the housing

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densities were approximately 62159 per square Km as per 2011 census. Hence there is need of study of seismic safety and the design of the structures on slopes.

The response of a sloped building depends on frequency content of the earthquake as it affects its performance when it is subjected to ground motion. In this research work experimental and numerical study is done by varying sloping angle. .

1.4 Objective and Scope

The purpose of this project is to study experimentally and numerically the dynamic response of sloped building subjected to sinusoidal ground motion and earthquake excitations.

The scope of this study is summarized as follows:

- The experimental study is undertaken with a two storied sloped frame model mounted rigidly to a shake table, capable of producing sinusoidal acceleration to study the dynamic response of sloped frame due to change of slope inclination by keeping the total height of frame constant.
- Finite element method is used as a numerical tool to solve the governing differential equation for undamped free vibration to find the natural frequency of model.
- Newmark method is used for numerical evaluation of dynamic response of the frame model.
- Linear time history analysis is performed using structural analysis tool i.e., STAAD Pro. by introducing compatible time history as per spectra of IS 1893 (Part 1):2002 for 5 % damping at rocky soil.

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Chapter 2

LITERATURE REVIEW

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LITERATURE REVIEW

2.1 Overview

In this review, characteristics of the structures due to the variation of the slope angle are explained. Then the effect of the irregular configurations on vulnerability due to seismic forces is discussed. There are very few researchers who explained the effect of change of sloping angle.

No research work is done based on experimental investigation of the structures on sloping ground.

2.2 Seismic Behaviour of Irregular Buildings on slopes in India

Ravikumar et al. (2012) studied two kinds of irregularities in building model namely the plan irregularity with geometric and diaphragm discontinuity and vertical irregularity with setback and sloping ground. Pushover analysis was performed taking different lateral load cases in all three directions to identify the seismic demands. All the buildings considered are three storied with different plan and elevation irregularities pattern. Plan irregular models give more deformation for fewer amounts of forces where the vulnerability of the sloping model was found remarkable. The performances of all the models except sloping models lie between life safety and collapse prevention. Hence it can be concluded that buildings resting on sloping ground are more prone to damage than on buildings resting on flat ground even with plan irregularities.

Sreerama and Ramancharla (2013) observed that recent earthquakes like Bihar-Nepal (1980), Shillong Plateau and the Kangra earthquake killed more than 375,000 people and over 100,000 of the buildings got collapsed. Dynamic characteristics of the buildings on flat ground differ to that of buildings on slope ground as the geometrical configurations of the

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building differ horizontally as well as vertically. Due to this irregularity the centre of mass and the centre of stiffness does not coincide to each other and it results in torsional response. The stiffness and mass of the column vary within the storeys that result in increase of lateral forces on column on uphill side and vulnerable to damage. In their analysis they took five G+3 buildings of varying slope angles of 0, 15, 30, 45, 60° which were designed and analysed using IS-456 and SAP2000 and further the building is subjected and analysed for earthquake load i.e., N90E with PGA of 0.565g and magnitude of M6.7. They found that short column attract more forces due to the increased stiffness. The base reaction for the shorter column increases as the slope angle increases while for other columns it decreases and then increases. The natural time period of the building decreases as the slope angle increases and short column resist almost all the storey shear as the long columns are flexible and cannot resist the loads.

Patel et al. (2014) studied 3D analytical model of eight storied building was analysed using analysis tool ETabs with symmetric and asymmetric model to study the effect of variation of height of column due to sloping ground and the effect of concrete shear wall at different locations during earthquake. In the present study lateral load analysis as per seismic code was done to study the effect of seismic load and assess the seismic vulnerability by performing pushover analysis. It was observed that vulnerability of buildings on sloping ground increases due to formation of plastic hinges on columns in each base level and on beams at each storey level at performance point. The number of plastic hinges are more in the direction in which building is more asymmetric. Buidings on sloping ground have more storey displacement as compared to that of buildings on flat ground and without having shear wall. Presence of shear wall considerably reduces the base shear and lateral displacement.

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2.3 Seismic Behaviour of buildings with Different Configurations

Birajdar and Nalawade (2004) performed 3D analysis of 24 RC buildings with three different configurations like set back, step back and step set back building. Response spectrum analysis including the torsional effect has been carried out. The dynamic properties which are top storey displacement, base shear and fundamental time period have been studied considering the suitability of buildings on sloping ground. In this study three types of configuration mentioned above are used in two (step back and step set back building) are on sloping ground while the third one (set back) is on plain ground. The sloping angle is taken as 27 degrees. The number of stories taken is from 4 to 11 and hence total of 24 RC buildings where studied. Set back building- As the number of stories increases there is a linear increase in top storey displacement and time period for the earthquake in longitudinal direction. The value of top storey displacement and fundamental time period in transverse direction are higher compared to longitudinal direction due to increase in torsional moments due to effect of static and accidental eccentricity. From design point of view proper attention should be given to the strength, orientation and ductility demand of shortest column at ground level to ensure its safety under worst combination of load case in X and Y direction. Step set back building-The results obtained in the static and dynamic analysis do not differ substantially as in the case of step back building. The top storey displacement is about 3.8 to 4 times higher in transverse direction than the corresponding values in longitudinal direction. Set back building- Shear forces induced in set back building is found to be least in comparison with the other two buildings. The distribution of shear forces in set back building is even and there is little problem of development of torsional moment. Step back buildings are found to be most vulnerable compared to other configurations and the development of torsional moment is highest in step back building. The column at ground level is prone to damage as it is worst affected.

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Singh et al. (2012) carried out an analytical study using linear and nonlinear time history analysis. They considered 9 story RC frame building (Step back) with 45 degrees to the horizontal located on steep slope. The number of storeys was 3 and 9 and 7 bays along the slope and 3 across the slope. They took 5 set of ground motions i.e., 1999 Chi-Chi, 1979 Imperial Valley, 1994 Northridge , 1971 San Fernando , 1995 Kobe from strong motion database of pacific Earthquake Engineering Research Centre (PEER). They observed that almost all the storey shear is resisted by the short column. The effect of torsional irregularity is represented by the ratio of maximum to average inter storey drifts ($\Delta_{max}/\Delta_{avg}$) in a storey. They observed the step back buildings are subjected to considerable amount of torsional effects under cross slope excitations.

Babu et al. (2012) performed pushover analysis of various symmetric and asymmetric structures constructed on plain as well as on sloping ground. They conducted analysis using structures with different configurations which are plan symmetry and asymmetry having different bay sizes. They considered a 4 storey building in which one storey is above ground level and it is constructed at a slope of 30 degree. They observed that the short column subjected to worst level of severity and lie beyond collapse prevention (CP) from pushover analysis. They obtained displacement as 104 mm and base shear as $2.77*10^3$ kN. Based on these results they developed pushover curves with X-axis as displacement and Y-axis as base shear and gave various comparisons for the cases they considered. They found that up to failure limit for maximum displacement by symmetric structure is 70% and by asymmetric building is 24% more than the structure on plain ground. They concluded that structure is more critical in elevation irregularity than in plan irregularity.

Prashant and Jagadish (2013) studied the seismic response of one way slope RC building with a soft storey. They have focussed their work to the buildings with infill wall and without infill wall i.e., bare frame. They carried out pushover analysis in a 10 storey building which

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include bare frame with and without infill wall. The buildings were situated at an inclination of 27 degrees to the horizontal and having 5 bays along the slope. Frame system considered was specially moment resisting frame (SMRF). In this study, they found that time period of building consisting of bare frame is 1.975 sec. which is about 96-135% higher as compared to the building having infill walls which is due to the reason of increased stiffness of the building and hence the increase in frequency. Further they observed that the displacement of the building is more in case of bare frame due to reduced stiffness and absence of infill wall. They also found that the base shear in infilled frames is about 250% more as compared to bare frame. Therefore formation of plastic hinges is more in bare frame model consisting of soft storey.

Halkude et al. (2013) conducted seismic analysis of buildings resting on sloping ground by varying number of bays and slope inclination. They studied the dynamic characteristics of the building i.e., base shear, top storey displacement and natural time period with respect to variation in number of stories and number of bays along the slope and hill slope. They considered a step back building of 4 to 11 storey and 3 to 6 bays in longitudinal directions. They have not considered the variation of bays in transverse direction so they have kept the single bay in Y-direction. The slope angles taken are 16.32°, 21.58°, 26.56° and 31.50° with the horizontal and seismic zone III. In all configurations it was observed that base shear increases with increase in number of storey, increases with increase in number of bays but decreases with increase in slope angle. Comparing within different configurations, step back building have higher base shear with respect to the step set back buildings. They also found time period increases with the increase in number of storey in both the configurations, with the increase in number of bays in step back building time period increases while in case of step set back building time period decreases. As the slope angle increases the stiffness of the building increases therefore the time period in all the configurations decreases. Top storey

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displacement decreases with the increase in hill slope, increases with the increase in the number of storey and decreases when the number of bays is increased. They concluded that more number of bays are better as this increases the time period and therefore it reduces top storey displacement.

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Chapter 3

EXPERIMENTAL MODELING

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Experimental Modeling

3.1 Introduction

This chapter deals with experimental works performed on free vibration and forced vibration on sloped frame model. The results obtained from the experimental analysis are compared with the finite element coding executed in MATLAB platform. The work performed is categorized into three sections which are as follows:-

- Details of Laboratory Equipments
- Fabrication and Arrangement
- Free and Forced Vibration Analysis

3.2 Experimental Modeling

3.2.1 Details of Laboratory Equipments

1. **Three Mild Steel plates-** In this model, there are three mild steel plates, two of same sizes and the other of different size. Plate no. 1 and 2 are used in each storey level and plate no. 3 used as base plate. The dimension of plates is shown in table 3.1:-

Table 3.1: Dimensions and Mass of mild steel plate

Plate No.	Dimension (cm)	Mass (kg)
Plate 1 & 2	50x40x1	15.44
Plate 3	70x40x1	21.76

2. **Four Threaded rods-** The threaded rods are used as columns which are connected with mild steel plates in each storey level. The diameter of threaded rod used is 7.7 mm.

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3. **Nuts and washers**- The number of set of Nuts and washers used is 32. Each 8 sets for two storey levels to connect threaded rods with steel plates and 8 nos. for base plate and 8 nos. for connecting threaded rod to the plate of shake table.

4. **Wooden logs and planks**- The wooden logs and planks are used to obtain firm ground. The logs of wood are inserted in between base plate and shake table to fill the space between inclined base plate and platform of shake table. Wedge shaped small logs of wood are also used which facilitates in erect fitting of column with plates.

Figure 3.1: Wooden Wedge and logs

5. **Shake Table**- Shake table is used to simulate the seismic event happening on the site. The shake table consists of horizontal, unidirectional sliding platform of size 1000 mm x 1000 mm. It consists 81 tie down points at a grid of 100 mmx 100mm. The maximum payload is 100 kg. The maximum displacement of the table is 100 mm (±50 mm). The rectangular platform is used to test the response of structures to verify their seismic performance. In this table the test specimen is fixed to the platform and shaken. The frequency of the table is controlled by a control panel which is run by input voltage of 440 volts.

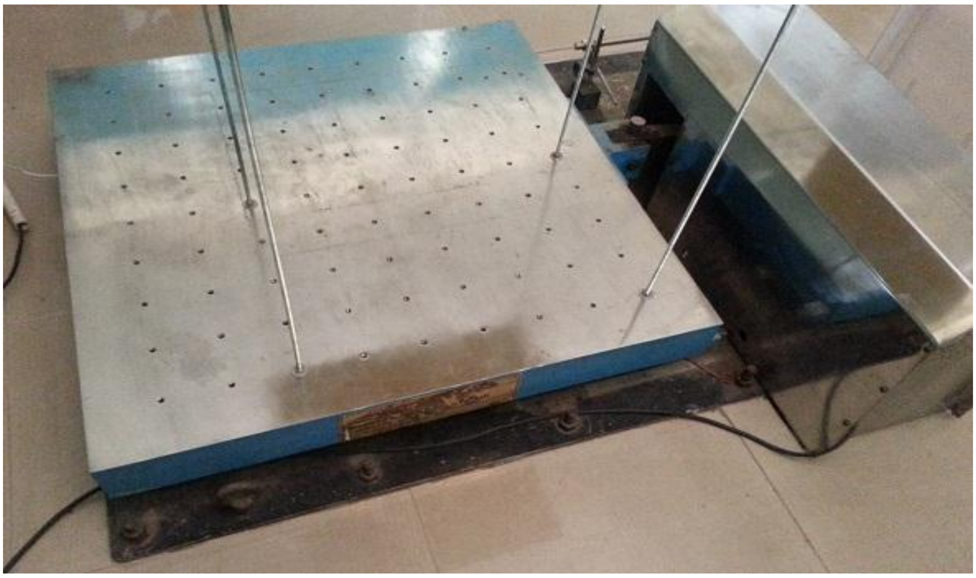

Figure 3.2: Shake Table

6. **Vibration Analyser**- Vibration analyser (VA) is an important component to condition monitoring program. It is also referred as predictive maintenance. It is used to measure the acceleration, velocity and displacement displayed in time waveform (TWF). But the commonly used spectrum is that derived from a Fast Fourier Transform (FFT). Vibration Analyser provides key information about the frequency information of the model.

Figure 3.3: Vibration Analyser

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7. **Control Panel**- This device is used to allow the user to view and manipulate the forcing frequency of the model. The range of frequency available for the operation of shake table is from 0 to 20 Hz.

Figure 3.4: Control Panel

8. **Personal Computer** – The computer system used to perform the test consists of Intel(R) Core (TM) i5 processor with 4 GB RAM, 32-bit operating system and running Windows 7 professional. The software used for data acquisition is NV Gate. This software facilitates user to conduct the FFT analysis of the received signal and record various graphs i.e., time versus acceleration, time versus velocity and time versus displacement. All the records obtained during the vibration of the model is simultaneously displayed in the monitor.

9. **Accelerometer**- It is a device which is used to measure the proper acceleration. Proper acceleration does not meant to be the co-ordinate acceleration (rate of change of velocity with tim) but it is the acceleration which it experiences due to the free fall of an object. Accelerometer transfers its record to the vibration analyser which is received by computer and transforms it to a signal.

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Figure 3.5: Accelerometer

3.2.2 Fabrication and Arrangement

The holes of 8 mm diameter are driven in the plates 4 nos. through which threaded bar passes. The holes are made at a radial distance of $5\sqrt{2}$ cm from each corner of the plate. In plate 3 slot cut of 2 cm is done at a radial distance of $5\sqrt{2}$ cm from each corner of base plate which is connected to platform of shake table. A slot cut of 5 cm is made on base plate to accommodate slope angle of 15°, 20° and 25° at a distance of 41 cm from slot cut of connected leg. The threaded rods are passed through these slots and holes and are fixed to the platform using nuts and washers. Now the base plate is fixed maintaining the slope angle of 15°, 20° and 25° (one at a time). Now the Plate 1 and 2 are fixed at a clear distance of 51 cm and 92.5 cm from connected end of base plate respectively. The screw is tightened well to ensure proper fixity. The wooden logs are inserted in between base plate and platform to achieve firm base similar to that of a sloping ground. Now three accelerometers are connected to the plates, two of them with plate 1 and one with plate 2. These accelerometers are connected with the vibration analyser and this analyser is connected to the computer. The readings obtained due to the vibration are recorded through the accelerometer. One LVDT (Linear Variable Displacement Transducer) is also used to record the displacement of the

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shake table at the time of forced vibration. The maximum amplitude of the ground motion is kept 5 mm. The entire tests were conducted in the "Structural Engineering" laboratory of NIT Rourkela.

3.2.3 Free and Forced Vibration Analysis

Free Vibration Analysis

A vibration is said to be free when a mechanical system is set off to an initial input and then set to vibrate freely. The vibrating system will damp to zero before that it will provide one or more natural frequency. In this experimental model, free vibration analysis is performed to obtain the natural frequencies of the model. By conducting FFT analysis we obtained two dominating frequencies which are natural frequencies. These two frequencies will be used as a basis for further analysis. A slight push is given to the Plate 1 (Top storey) and the readings are taken and by doing FFT analysis natural frequency of the system are obtained.

Forced Vibration Analysis-

A forced vibration is one in which system is subjected to disturbance varying with time. The disturbance may be load, displacement or velocity and it may be periodic or non-periodic, transient or steady. The periodic input may be harmonic or non-harmonic in nature. Example vibration of building subjected to earthquake. If the frequency of vibration of the model is equal to its natural frequency then the system will be said to have condition of resonance. The response of the system is large during the resonance and it may be of such magnitude that it may lead to failure of structure.

3.3 Experimental Models

Following are figures showing the experimental model with different slope angle:-

3.3.1 Experimental Model for 15° slope

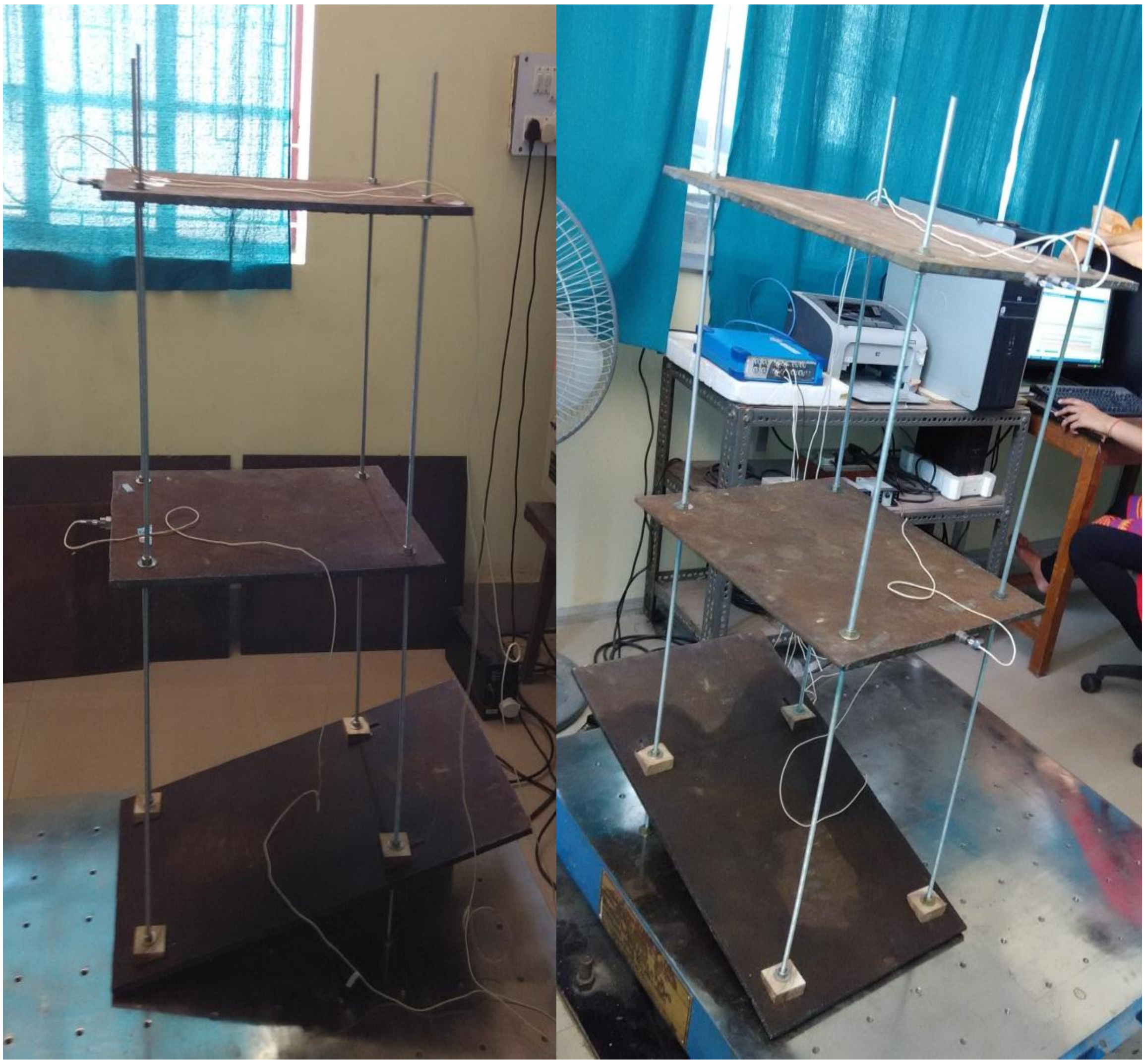

Figure 3.6: Experimental Model for 15° slope

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3.3.2 Experimental Model for 20° slope

Figure 3.7: Experimental Model for 20° slope

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3.3.3 Experimental Model for 25° slope

Figure 3.8: Experimental Model for 25° slope

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3.4 Experimental Results and Discussions

During the experiment, free vibration analysis was performed for each frame model as mentioned in article 3.2.3. The first two natural frequencies obtained for two modes are shown in table 3.2.

Table 3.2: Natural frequencies of model with different slope inclinations

Type of Model	Natural Frequency (Hz)	
	Mode 1	Mode 2
15°	2.05	5.80
20°	2.2	5.945
25°	2.6	6.55

Each of the above frame model were excited with sinusoidal harmonic loading which is defined by following expression

$x = x_o \sin\omega t$; $[\omega = 2\pi f]$

where x_o is the amplitude of excitation (mm)

f is the frequency of excitation (Hz)

In the above expression, the frequency of excitation is applied over a range which included the natural frequency of the model. The displacement amplitude of excitation was kept constant i.e., x_o = 5 mm. The maximum storey displacements obtained at resonance condition

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i.e., when excitation frequency matches with the natural frequency of the model for all the slope angles is shown in table 3.3, table 3.4 and table 3.5.

Table 3.3: Maximum Storey Displacements (Absolute) for frame model of 15° inclination

Storey No.	Maximum Storey Displacement (mm)
1	55.2
2	76.6

Table 3.4 : Maximum Storey Displacements (Absolute) for frame model of 20° inclination

Storey No.	Maximum Storey Displacement (mm)
1	44
2	68.3

Table 3.5: Maximum Storey Displacements (Absolute) for frame model of 25° inclination

Storey No.	Maximum Storey Displacement (mm)
1	32.9
2	58.3

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3.5 Frequency Response Analysis

Figure 3.9 shows the response of frequency (Hz) on X-axis with Top storey displacement (mm) on Y-axis for all three slope angles. In this plot the displacement is decreasing due to the increase in frequency and slope angle and the increased stiffness of short column on hill side.

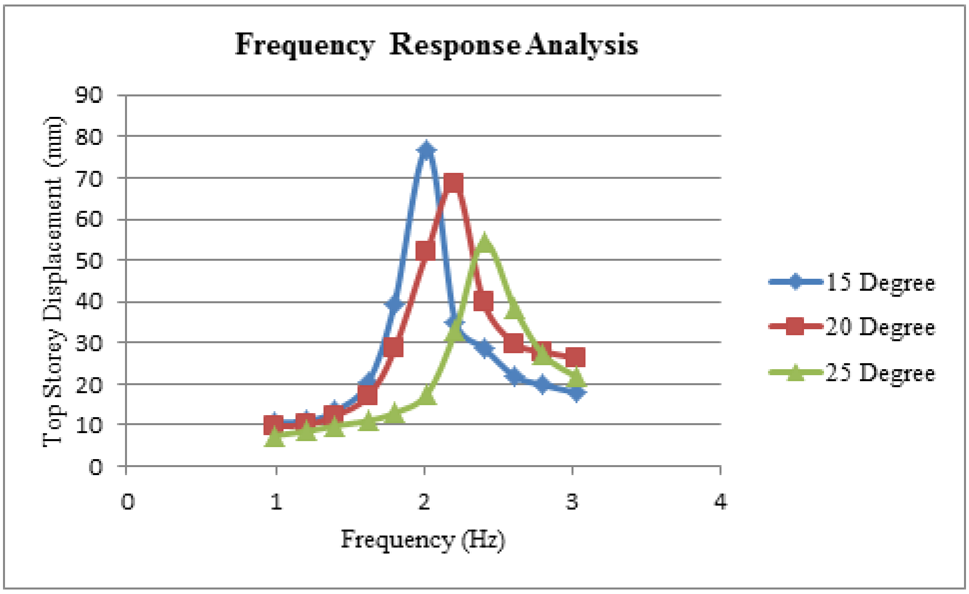

Figure 3.9: Frequency Response analysis

Figure 3.10(a) and 3.10(b) for acceleration (top storey) versus time showing the dominance of first fundamental frequency (2.05 Hz) obtained by superimposing it with the excitation frequency of value lower (1.62 Hz) than the fundamental frequency and of value higher (2.80 Hz) than the fundamental frequency. In both the plots it is observed that fundamental frequency dominates the response over the excitation frequencies of 1.62 Hz and 2.80 Hz.

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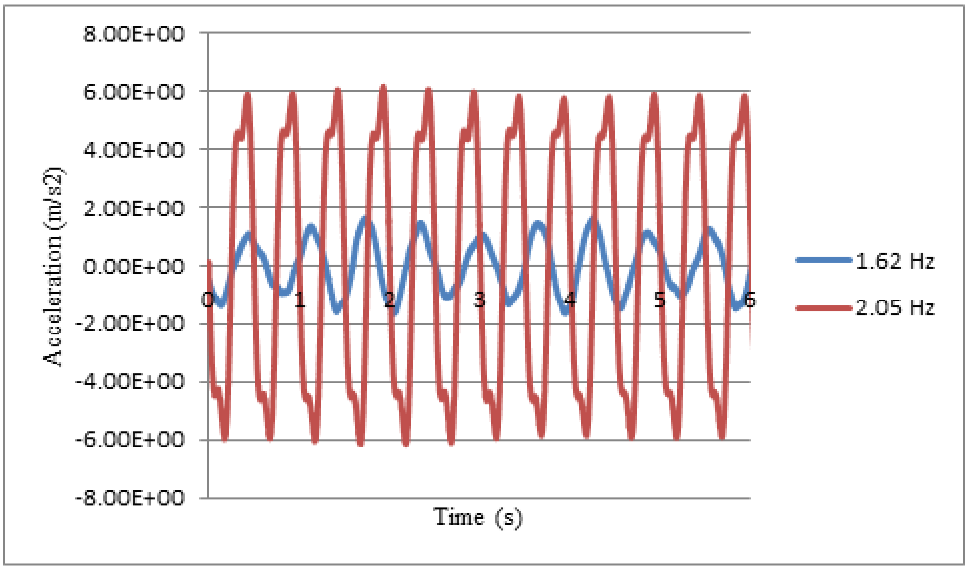

Figure 3.10(a): Time history of Top floor acceleration under sinusoidal ground motion with amplitude of 5 mm and frequencies 1.62 Hz and 2.05 Hz

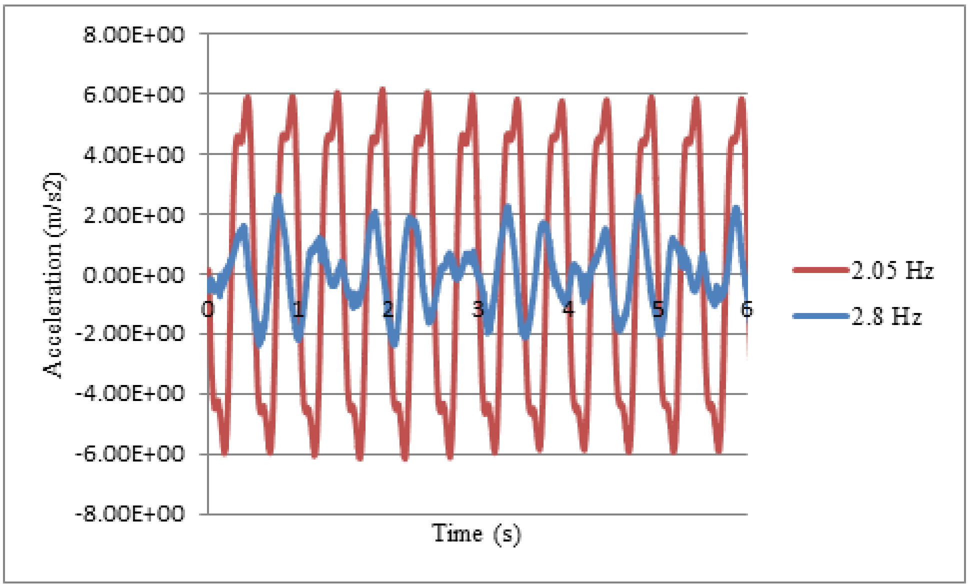

Figure 3.10(b): Time history of Top floor acceleration under sinusoidal ground motion with amplitude of 5 mm and frequencies 2.05 Hz and 2.80 Hz

Figure 3.11(a) and 3.11(b) for acceleration (top storey) versus time showing the dominance of first fundamental frequency (2.21 Hz) obtained by superimposing it with the forcing frequency of value lower (1.80 Hz) than the fundamental frequency and of value higher (2.80

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Hz) than the fundamental frequency. In both the plots it is observed that fundamental frequency dominates the response over the excitation frequencies of 1.80 Hz and 2.8 Hz.

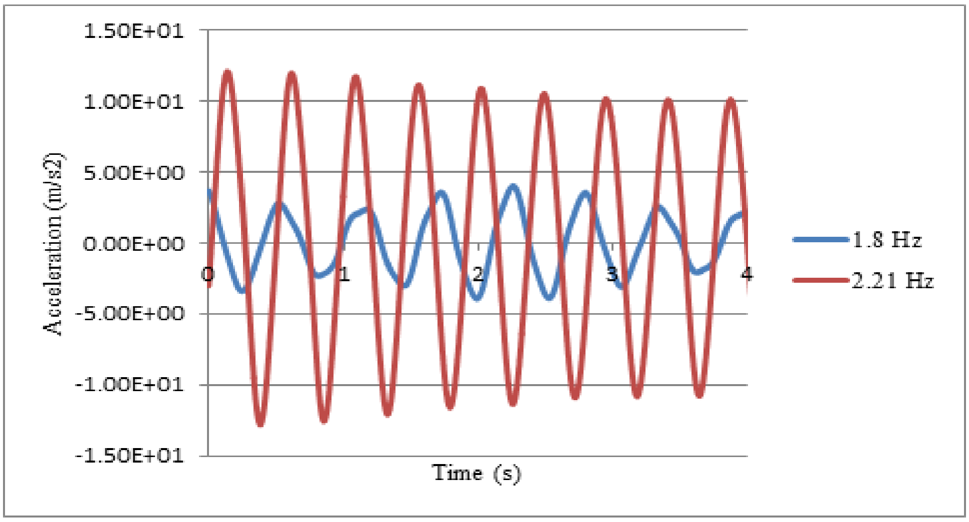

Figure 3.11(a): Time history of Top floor acceleration under sinusoidal ground motion with amplitude of 5 mm and frequencies 1.8 Hz and 2.21 Hz

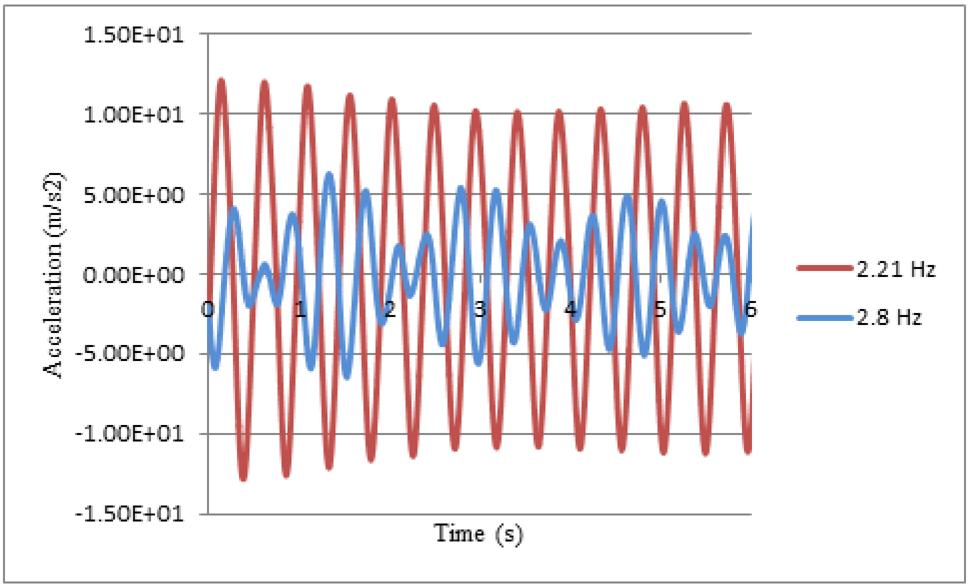

Figure 3.11(b): Time history of Top floor acceleration under sinusoidal ground motion with amplitude of 5 mm and frequencies 2.21 Hz and 2.8 Hz

Figure 3.12(a) and 3.12(b) for acceleration (top storey) versus time showing the dominance of first fundamental frequency (2.6 Hz) obtained by superimposing it with the forcing

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frequency of value lower (2.02 Hz) than the fundamental frequency and of value higher (2.80 Hz) than the fundamental frequency. In both the plots it is observed that fundamental frequency dominates the response over the excitation frequencies of 2.02 Hz and 2.80 Hz.

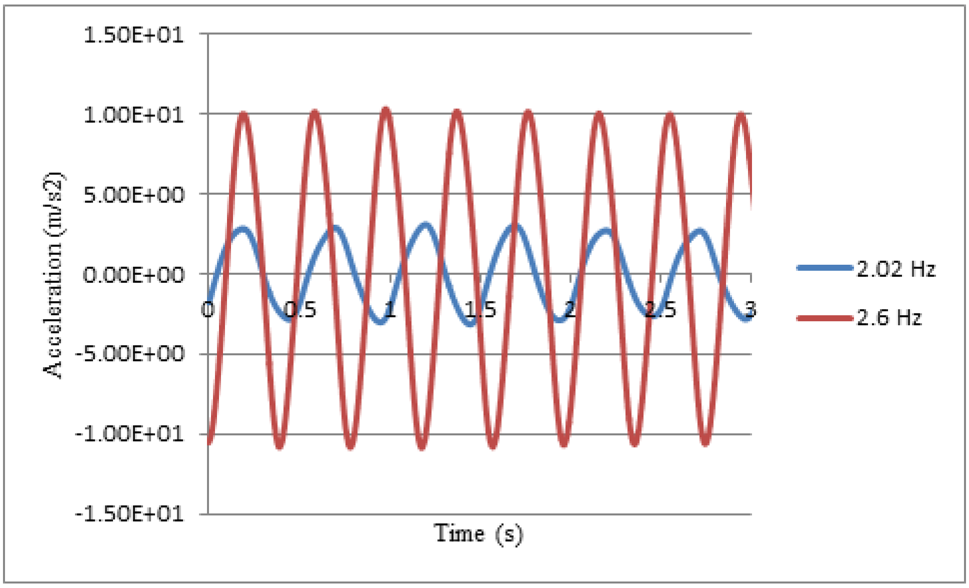

Figure 3.12(a): Time history of Top floor acceleration under sinusoidal ground motion with amplitude of 5 mm and frequencies 2.02 Hz and 2.6 Hz

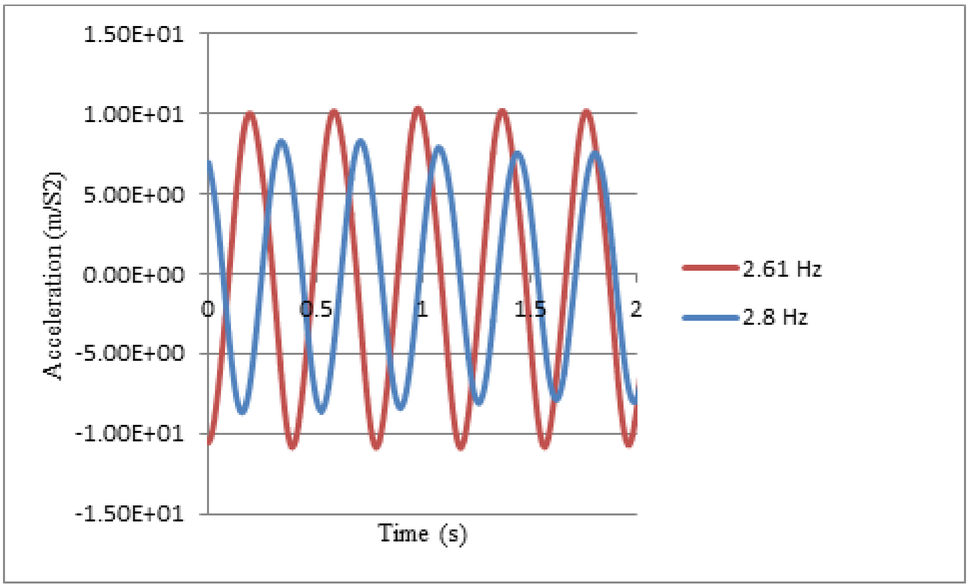

Figure 3.12(b): Time history of Top floor acceleration under sinusoidal ground motion with amplitude of 5 mm and frequencies 2.61 Hz and 2.8 Hz

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Chapter 4

NUMERICAL MODELING

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Numerical Modeling

4.1 Introduction

Form the literature review we observed that there is a need to develop a Finite Element model on sloped frame to validate the results obtained from the commercial software like STAAD Pro., ETABs, and SAP 2000 etc. Therefore a finite element modeling is carried out for the forced vibration analysis. A finite element model is developed for the sloped frame and its natural frequencies are computed by conducting free vibration analysis. Forced vibration analysis is used to study the dynamic response of the frame model with the help of Newmark's integration method and the results obtained are validated with structural analysis tool i.e., STAAD Pro.

4.2 Finite Element Modeling

4.2.1 Newmark Direct Integration Approach

Flow chart is developed to understand the classification of analysis. In this numerical model, out of various direct integration approach, newmark's direct integration approach is used.

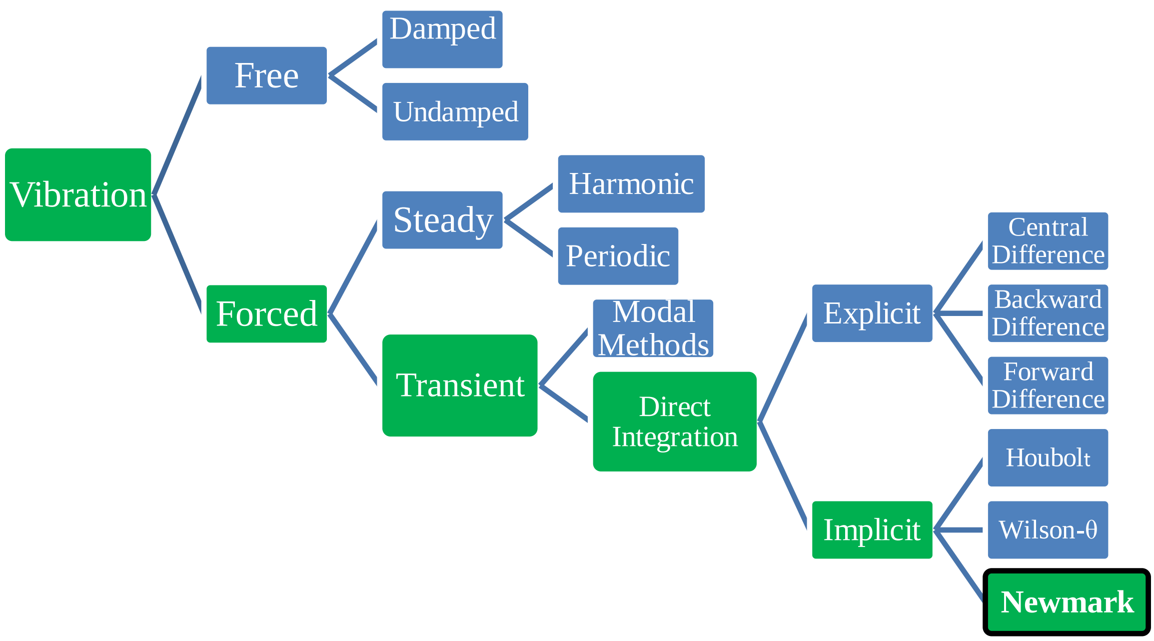

Figure 4.1: Flow chart for Classification of Vibration

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Direct integration method considers a step by step integration in time. These are of two types:

1. Explicit

2. Implicit

In explicit type of direct integration data used from past n number of steps to protect forward in time. It is popular for non-linear cases and is easy to code. It can become unstable and its stability require small steps i.e., accuracy is directly related to step size. Thus it is conditionally stable. Used in linear acceleration method for $\Delta t \leq 0.551T_j$, where T_j is natural period of j^{th} mode.

In implicit type, information from the past time and equation of motion at the present time is used. It is tougher than explicit method to program. It can be made unconditionally stable independent of step size. It has a strong filtering action to smoothen and attenuate the predictive response and we don't get the response that calculated response diverges or oscillates and the penalty to use the large step size is to lose the high frequency character to smooth out the response. Used in average acceleration method.

4.2.2 Newmark Method

It is an implicit method which can be made unconditionally stable. It is a method of numerical integration used to solve the differential equation. It has parameters ß and γ that are adjusted for accuracy and stability.

4.2.3 Procedure

What Newmark (1959) proposed has become the most popular to solve the problems in structural dynamics among the family of algorithms. The method of Newmark relies on the following interpolation that relate the response-displacement, velocity and acceleration-increments over the time step n to n+1.

$$\mathbf{v}_{n+1} = \mathbf{v}_n + \Delta t \, [\, (1-\gamma)\, \mathbf{a}_n + \gamma\, \mathbf{a}_{n+1}] \qquad ......(1)$$

$$\mathbf{x}_{n+1} = \mathbf{x}_n + \Delta t\, \mathbf{v}_n + \frac{\Delta t^2}{2} [(1-2ß)\, \mathbf{a}_n + 2ß\mathbf{a}_{n+1}] \qquad ......(2)$$

`

where, $\mathbf{x}_{n}$, $\mathbf{v}_{n}$, $\mathbf{a}_{n}$ are the approximation to the position, velocity and acceleration vectors at time step n.

Δt is the time increment in each step.

ß and γ are the parameters whose values define the method.

The general approach is unconditionally stable when $\gamma \geq 0.5$ and $ß \geq 0.25\ (\gamma + 0.5)^2$.

Newmark showed $\gamma = 0.5$ is the only responsible value otherwise get damping (didn't get effect of artificial damping). Newmark chose ß = 0.25 as the best comparison between accuracy and stability. Use of $\gamma = 0.5$ and ß = 0.25 is called as **Newmark- ß (Beta) method**. If the value of $\gamma = 0.5$ and ß = 1/6 then this method is conditionally stable and it is used as the basis for another important method known as Wilson- θ (theta) method.

Similar to multistep methods, the implicit algorithms of equations 1 and 2 with fixed point iteration can be used in fashion of predictor- corrector. But it is not done in such way interpolations of equations 1 and 2 are introduced directly into the equations of motion. Depending upon the type of problem linear or non-linear set of algebraic equations are developed.

The algebraic equation are solved by substituting a_{n+1}, v_{n+1} in terms of x_n, v_n, a_n and x_{n+1}, with x_{n+1} as primary unknown.

Rewriting the equations (1) and (2) as follows:

$$a_{n+1} = \frac{1}{ß\Delta t^2}(x_{n+1} - x_n) - \frac{1}{ß\Delta t}v_n - \left(1 - \frac{1}{2ß}\right)a_n \qquad ......(3)$$

$$\mathbf{v}_{n+1} = \frac{\gamma}{ß\Delta t}(\mathbf{x}n+1 - \mathbf{x}n) - \left(\frac{\gamma}{ß} - 1\right)v_n - \left(\frac{\gamma}{2ß} - 1\right)\Delta t\, a_n \qquad ......(4)$$

These equations are applied to linear dynamic structural problem to explain the procedure which has following form,

`

$$\mathbf{M}\,\mathbf{a}_{n+1} + \mathbf{C}\,\mathbf{v}_{n+1} + \mathbf{K}\,\mathbf{x}_{n+1} = \mathbf{F}(t) \qquad ......(5)$$

Where M represents mass matrix, C represents damping matrix and K represents the stiffness matrix. F*(t)* is the force vector for externally applied force.

Substituting equations (3) and (4) in equation (5),

$$\left[\frac{1}{ß\Delta t^2}M + \frac{\gamma}{ß\Delta t}C + K\right]x_{n+1} = F(t) + M\left[\frac{1}{ß\Delta t^2}x_n + \frac{1}{\gamma\Delta t}v_n + (1-\frac{1}{2ß})\,a_n\right] +$$

$$C\left[\frac{\gamma}{ß\Delta t}\,xn + (\frac{\gamma}{ß} - 1)vn + (\frac{\gamma}{2ß} - 1)\,\Delta t\,an\right]$$

If the constant matrix on the left hand side of the above equation which is the multiple of x_{n+1} is triangularized, then solution for displacement only requires formation of right hand side of equation (6) plus a forward reduction and a backward substitution. Newmark converted dynamic equation to a linear static equation.

$$F_{eff} = K_{eff} * x \qquad .....(7)$$

Where F_{eff} is the effective force and is equal to all the terms in right hand side of equation (6)

K_{eff} is the effective force and is equal to the multiples of x_{n+1} of equation (6)

x is displacement at a particular time step.

4.2.4 Details about M, C and K matrices

4.2.4.1 Mass matrix [M]

$$[M] = \begin{bmatrix} m1 & 0 \\ 0 & m2 \end{bmatrix} \qquad .....(8)$$

`

where $m1$ and $m2$ are the mass of storey 1 and 2.

$$[M] = \begin{bmatrix} 15.44 & 0 \\ 0 & 15.44 \end{bmatrix} \quad ......(8A)$$

4.2.4.2 Stiffness matrix [K]

The stiffness matrix is calculated for a slope angle of 15°, 20° and 25° by calculating effective length of varying length of column in bottom floor.

Calculation of effective length for 15° slope

Effective stiffness of columns in a storey for single bay is given by

k=k1+ k2 .....(9)

where k is the effective stiffness of the storey

k1 is the stiffness of long column

k2 is the stiffness of short column

k1 = $\frac{24EI}{l1^3}$; $\frac{24EI}{l1^3}$ instead of $\frac{12EI}{l1^3}$ is because for two columns in one bay in transverse direction

k2 = $\frac{24EI}{l2^3}$

k = 2*$\frac{24EI}{l^3}$

where, E is modulus of elasticity of column (Threaded rod) =77.3*10^3 MPa.

E for Mild steel plate = 2*10^5 Mpa.

I is moment of inertia of column

`

$I = \pi d^4/64;$

$I = (22*.0077^4)/(7*64)$

$I = 1.72626*10^{-10}\ m^4$

$l_{1\,=}\ 0.51$ m

$l_{2\,=}\ 0.4165$ m

Putting the above values in equation (8)

$$2*\frac{24EI}{l^3} = \frac{24EI}{l1^3} + \frac{24EI}{l2^3}$$

$$\frac{2}{l^3} = \frac{1}{l1^3} + \frac{1}{l2^3} \qquad \text{….. (10)}$$

l = 0.446779 m

Calculation of effective length for 20° slope

$l_{1\,=}\ 0.51$ m

$l_{2\,=}\ 0.373$ m

From equation (9)

$$\frac{2}{l^3} = \frac{1}{l1^3} + \frac{1}{l2^3}$$

Effective length

l =0 .42097 m

Calculation of effective length for 25° slope

$l_{1\,=}\ 0.51$ m

`

$l_2 = 0.32$ m

Again from equation (9)

$$\frac{2}{l^3} = \frac{1}{l1^3} + \frac{1}{l2^3}$$

Effective length

l = 0.37457 m

Element Stiffness matrix for one element-

$$[Ke] = \begin{bmatrix} ke1 & -ke1 \\ -ke1 & ke1 \end{bmatrix}$$

$$ke1 = \frac{24EI}{l^3}$$

Global stiffness Matrix-

$$[K] = \begin{bmatrix} ke1+ke2 & -ke2 \\ -ke2 & ke2 \end{bmatrix} \qquad \text{…..(11)}$$

Global Stiffness Matrix for entire building for 15° slope

$$[K] = \begin{bmatrix} 1.6871 & -0.9366 \\ -0.9366 & 0.9366 \end{bmatrix} * 10^4 \qquad \text{….(11A)}$$

Global Stiffness Matrix for entire building for 20° slope

$$[K] = \begin{bmatrix} 1.8338 & -0.9366 \\ -0.9366 & 0.9366 \end{bmatrix} * 10^4 \qquad \text{…(11B)}$$

Global Stiffness Matrix for entire building for 25° slope

$$[K] = \begin{bmatrix} 2.2103 & -0.9366 \\ -0.9366 & 0.9366 \end{bmatrix} * 10^4 \qquad \text{…(11C)}$$

`

4.2.4.3 Damping matrix [C]

Damping Matrix [C] is given by,

$C = [M]^* \phi^{**}([Mn]^{-1} * c * [Mn]^{-1})^* \phi^T*[M]$

$[C] = \begin{bmatrix} C1 + C2 & -C2 \\ -C2 & C1 + C2 \end{bmatrix}$

$c = 2^* \zeta * M_n{}^*\omega$

where, c is coefficient of damping and ζ is damping factor or ratio, $\zeta = 0.12$

Following steps are used to calculate the damping matrix-

1. Calculate the natural frequency of the model by using eigen value solution

Governing differential equation for free undamped vibration of structure is given by

$[M]\{\ddot{x}\} + [K]\{x\} = \{0\}$ ......(12)

Assuming a solution that satisfies displacement boundary condition

$x= A\sin\omega t$

By differentiating above equation with respect to 't' we get

$\dot{x} = A\omega\cos\omega t$

$\ddot{x} = -A\omega^2 \sin\omega t$

Putting the values of $\ddot{x}$ and x in above governing equation (12) we obtain

$\{[K]- \omega^2[M]\}\{ \phi\}= \{0\}$ …...(12A)

where, ϕ is the vector representing mode shape and ω is the natural frequency of the model.

`

Putting the values from equations 8(A), (11A), (11B) and (11C) into equation (12A)

Mode shapes and Natural frequencies

Mode shapes (ϕ) **and Natural frequencies (ω) for 15° slope**

$$\phi = \begin{bmatrix} -0.1426 & -0.2108 \\ -0.2108 & 0.1426 \end{bmatrix}$$

ω = 14.0063 and 38.7699 rad/s.

In Hz , the frequency can be converted as

$$f = \frac{\omega}{2\pi}$$

f = 2.2283 and 6.1679 Hz.

Mode shapes (ϕ) **and Natural frequencies (ω) for 20° slope**

$$\phi = \begin{bmatrix} -0.1356 & -0.2153 \\ -0.2153 & 0.1356 \end{bmatrix}$$

ω = 14.9854 and 39.6197 rad/s.

In Hz, the frequency can be converted as

$$f = \frac{\omega}{2\pi}$$

f = 2.384 and 6.3031 Hz.

Mode shapes (ϕ) **and Natural frequencies (ω) for 25° slope**

$$\phi = \begin{bmatrix} -0.1191 & -0.2249 \\ -0.2249 & 0.1191 \end{bmatrix}$$

ω = 16.8973 and 41.8639 rad/s.

`

In Hz, the frequency can be converted as

$$f = \frac{\omega}{2\pi}$$

f = 2.6882and 6.6602 Hz.

2. Calculation of normalized mass matrix [M_n] Matrix

$M_n = \phi^T * M * \phi$

For 15° slope

[C] is calculated as

$$[C] = \begin{bmatrix} 114.8500 & -42.5899 \\ -42.5899 & 114.8500 \end{bmatrix} \quad \text{.....(14A)}$$

For 20° slope

$$[C] = \begin{bmatrix} 120.8909 & -41.1633 \\ -41.1633 & 120.8909 \end{bmatrix} \quad \text{.....(14B)}$$

For 25° slope

$$[C] = \begin{bmatrix} 134.8846 & -38.2519 \\ -38.2519 & 134.8846 \end{bmatrix} \quad \text{.....(14C)}$$

The calculation of [M], [C] and [K] matrices have been done by writing finite model coding and executed in MATLAB platform and the result obtained are shown in the equations above.

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STAAD MODELING

4.3 Introduction

In this study, numerical modeling in STAAD Pro platform of the sloped frame is described. The plan and elevation of two storied sloped building subjected to ground motion record as per spectra of IS 1893 (Part 1)-2002 is shown. There are three different slope angle taken which are 15°, 20° and 25°. All the material properties of steel beam and column element are explained. Gravity loads considered are also explained. At the end the size of the elements are described.

4.4 Frame Modeling in STAAD

In this article, modelling is done in STAAD Pro. A two storied sloped frame model with plan and elevation is shown from figure 4.2 to figure 4.7 with different slope angle. But the total height of the building in all the three model is kept same i.e., 92.5cm of which height of first floor is 51 cm and 41.5 cm for the second floor. The length of bay is taken as 40 cm in longitudinal direction and 30 cm in transverse direction.

4.4.1 Two storied sloped frame with inclination of 15° to the horizontal

Plan

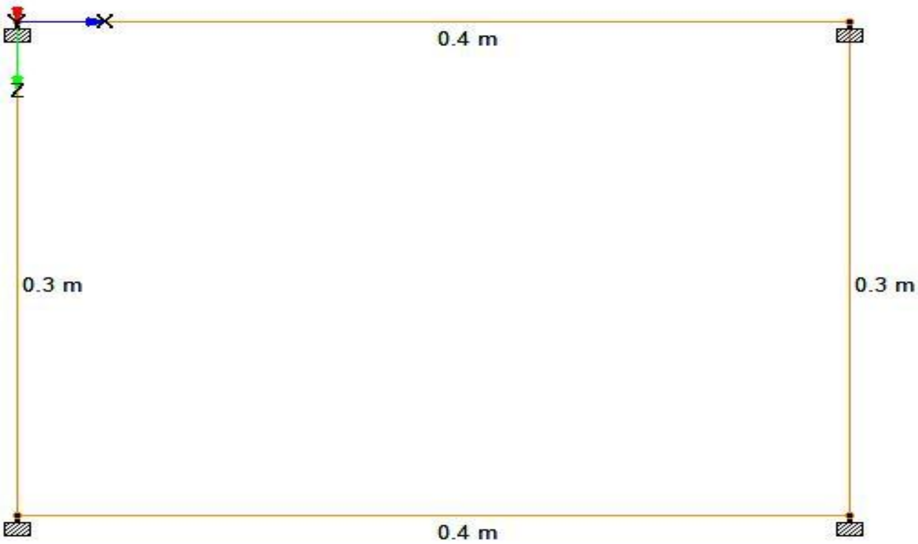

Figure 4.2: Plan of sloped frame for 15° inclination

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Elevation

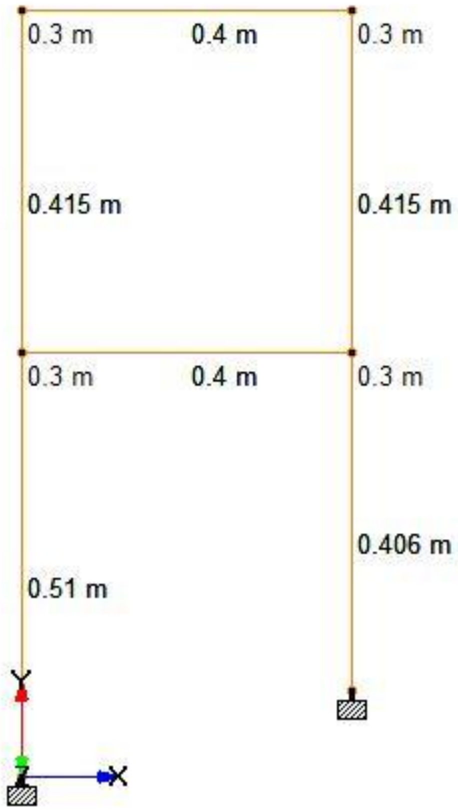

Figure 4.3: Elevation of sloped frame for 15° inclination

4.4.2 Two storied sloped frame with inclination of 20° to the horizontal

Plan

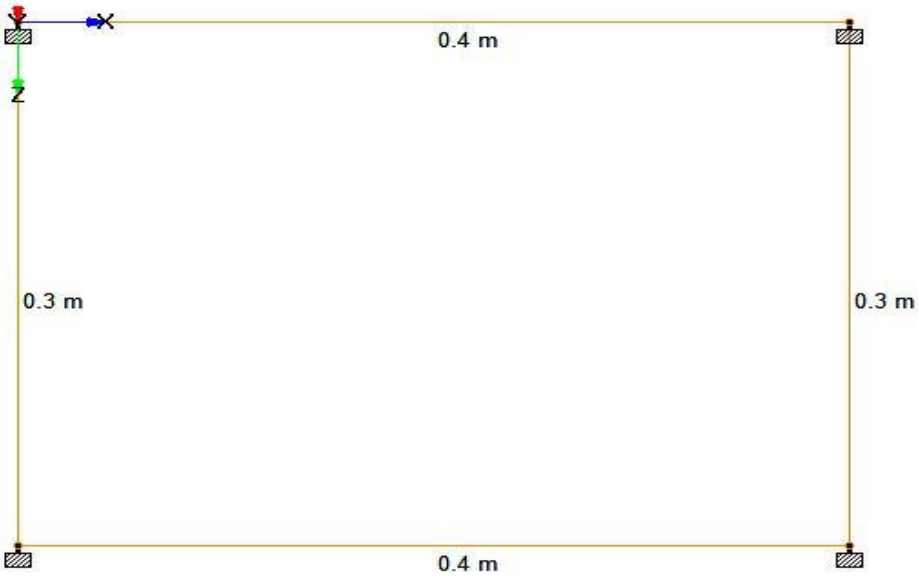

Figure 4.4: Plan of sloped frame for 20° inclination

`

Elevation

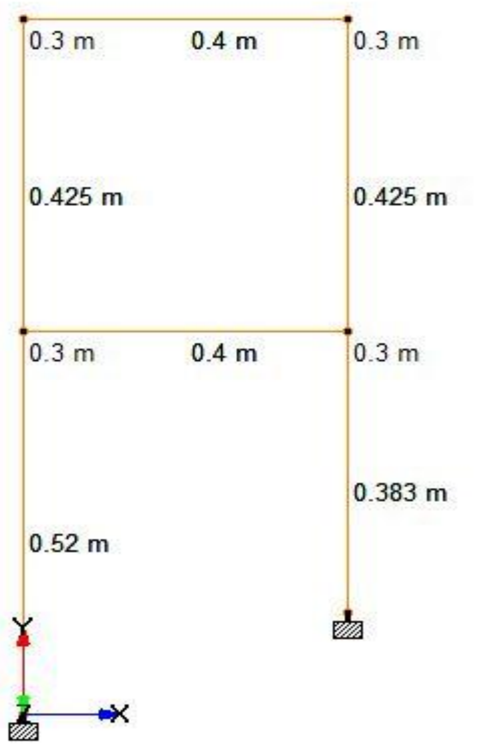

Figure 4.5: Elevation of sloped frame for 20° inclination

4.4.3 Two storied sloped frame with inclination of 25° to the horizontal

Plan

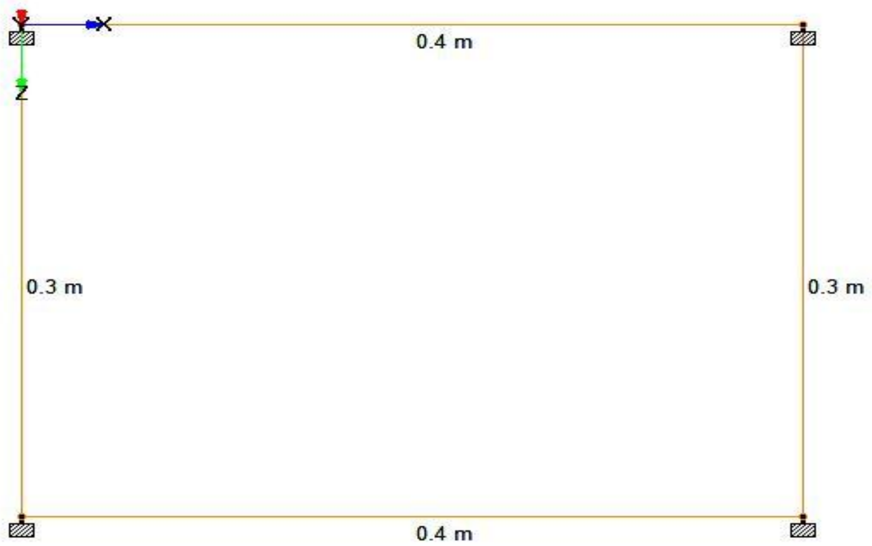

Figure 4.6: Plan of sloped frame for 25° inclination

Elevation

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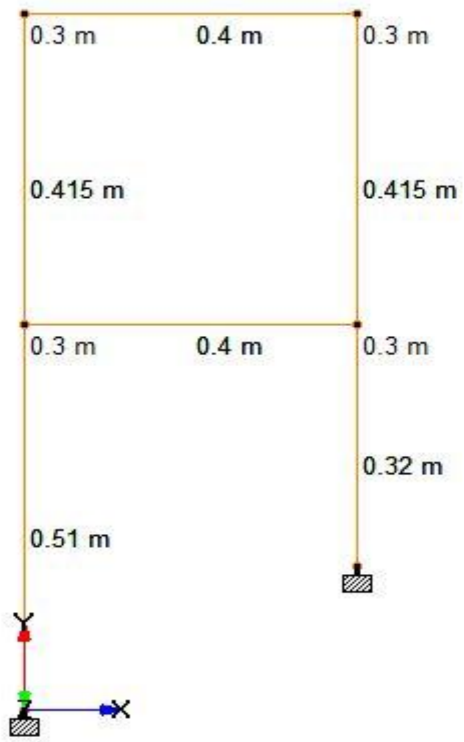

Figure 4.7: Elevation of sloped frame for 25° inclination

4.5 Loads

Uniformly distributed load of 0.5044 kN/m is applied in both longitudinal (X) direction and Y-direction at each storey level. The figure 4.8 shows front and side elevation of applied loads in X and Y directions.

The load applied is the mass of plate which is experimental model multiplied by the acceleration due to gravity i.e., 15.44*9.81=151.466 N or .151466 kN.

This value of load is uniformly distributed throughout the length of beam 0.151466/0.3=.50488 kN/m.

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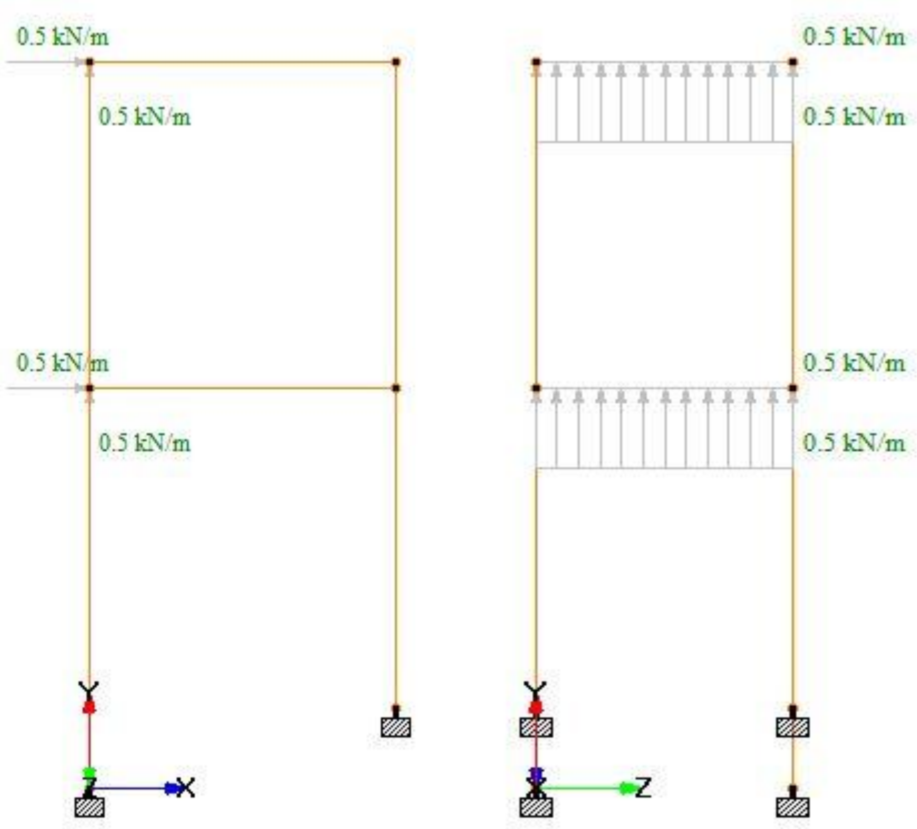

Figure 4.8: Load distribution in Longitudinal-X and Vertical-Y direction

4.6 Material Properties

The table 4.1 shows the properties of materials that are used in the modelling of structure in STAAD Pro.

Table 4.1: Steel and Column Bar Properties

Title	Steel Properties	Column Bar Properties
Modulus of Elasticity	20000 GPa	77.3 GPa
Poisons ratio (ν)	0.3	0.3
Mass Density (Kg/m^3)	7720	7300
Shear modulus	7692.307 GPa	29.615 GPa

`

4.7 Structural Elements

In STAAD Pro. Linear Time History Analysis is performed on above models subjected to ground motion of intermediate frequency content as per spectra of IS 1893(Part I): 2002. Height of storey for first and second floor is taken as 51 cm and 41.5 cm respectively. While the length of short column (on right) is 40.65 cm , 37.3 cm and 32 cm for slope of 15°, 20° and 25° respectively. The length of beam is 40 cm in longitudinal (X) direction and 30 cm in transverse (Z) direction. The details of size of beam and column are shown in table 4.2.

Table 4.2: Details of Beam and Column with length and cross section dimensions

Element	Cross Section Dimension(mm)	Length (cm)
Beam (X)	100x100	40
Beam (Y)	80x80	30
Column 1st floor	7.7	51
Column 2nd floor	7.7	41.5

4.8 Ground Motion and Time History Analysis

4.8.1 Ground Motion

It is the motion of earth's surface due to the earthquake or any explosion. It is produced due to the waves which are generated by slip of fault plane or sudden pressure at the explosive source which travel through the surface of the earth.

Earthquake is a term which is used to refer sudden release of seismic energy caused by sudden slip on a fault or due to any volcanic or magmatic activity. The strain energy stored inside the earth crust is released due to tectonic movement of the plates and maximum part of it changes into heat and sound and the remaining is transforms into the form of seismic waves. Most of the earthquakes occur due to the plate tectonics. The tectonic plates are large in size thin and rigid plates that moves relative to one another on the earth's outer surface.

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These plates are found in uppermost part of mantle which is together referred to as lithosphere. There are seven major plates which are Pacific, American, Australian, Indian, Eurasian, African and Antarctic plates.

The main concern of Engineers is the property and nature of ground motion while the scientists and researchers are interested in the nature and property of earthquake. Engineers use accelerograph to measure the ground acceleration whereas scientists use seismograph to record the seismic waves. The seismic waves are mainly of two types i.e., body waves and surface waves. The body waves further comprises of two types which are primary waves (P-wave) and secondary waves (S-wave). The surface waves are also of two types i.e., Rayleigh and Love waves.

When the shaking of earth is strong that is close to 50 km range is referred to as strong ground motion. The motion occurs in three linear displacements and three rotational displacements. Peak ground acceleration (PGA) is the maximum absolute value of ground acceleration. The frequency content, PGA and time duration are the three most important characteristics of an earthquake. The frequency content of an earthquake is the ratio of peak ground acceleration (PGA) in terms of acceleration due to gravity (g) to the peak ground velocity (m/s) (PGV). It is classified into three high, intermediate and low frequency content.

PGA/PGV > 1.2	High Frequency Content
0.8 < PGA/PGV > 1.2	Intermediate Frequency Content
PGA/PGV < 0.8	Low frequency content

The first natural frequency (corresponding to first mode) of a structure is termed as the fundamental frequency. When the excitation frequency and natural frequency matches then the resonance occurs. Earthquake ground motion is dynamic in nature and can be classified as deterministic non-periodic transient load as well as probabilistic load.

`

Earthquake is classified based on focal depth, location, epicentral distance, causes and magnitude. Intensity and magnitude are two specific parameters of earthquake. The intensity of earthquake is measured by the severity of shaking of ground at a certain location. It is a qualitative measure of an earthquake and is measured by MM scale (Modified Mercalli) scale. Magnitude is the amount of seismic energy released at the source of earthquake. It is a quantitative measure of an earthquake which is determined by Richter magnitude scale. For a particular earthquake the magnitude is constant irrespective of its location but its intensity varies from one location to another.

Figure 4.9 shows the variation of ground acceleration with time. The duration of ground motion is 40 seconds and its peak value is -1.0g which occurs at time t=11.90 seconds.

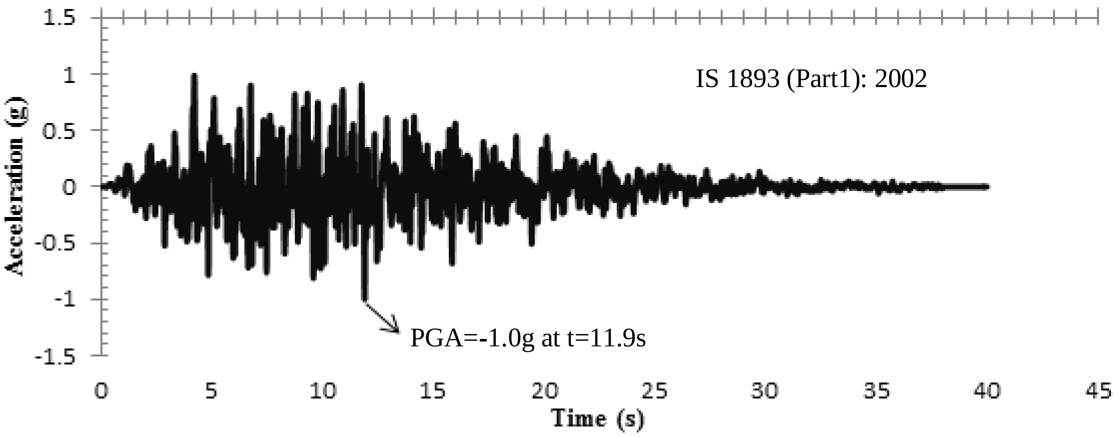

Figure 4.9: Compatible Time History as per spectra of IS 1893 (Part1):2002 for 5% damping at rocky soil

4.8.2 Time History Analysis

Structural analysis deals with finding out physical response of a structure when subjected to any action (force). This action can be static or dynamic. If the action is constant for a span of time then it is termed as static and if it varies fairly quickly then it is termed as dynamic. The study of response of the structure subjected to dynamic loading is called as structural dynamics. Ground motion comes under type of dynamic loading. Dynamic analysis is also related to inertial forces developed when the structure is subjected by suddenly applied loads for example wind blasts, explosion and earthquake.

Time history analysis is the dynamic response of a structure applied over the increment of time steps as a function of acceleration, force, moment or displacement. It provides the response under the loading which varies according to specified time function. The closer spacing of interval the greater is the accuracy achieved. This method is considered to be more realistic compared to response spectrum method. This method is useful for tall or high rise structures i.e., flexible structures. In linear dynamic model, structure is modelled with linear elastic stiffness matrix and equivalent damping matrix for multi degree of freedom structure. The main advantage of linear dynamic method over static method is that higher modes can also be taken into account.

In this study linear time history dynamic analysis is carried out to see the response of a two storied building. STAAD Pro. platform is used to perform the analysis. The structure is subjected to ground motion record [IS 1893 (Part1):2002 (Artificial ground motion)] compatible to time history of acceleration as per spectra of IS 1893 (Part1) for structural design in India (Refer figure 4.9: Time History of Ground Acceleration).

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Numerical Results and Discussions

4.9 Overview

In this chapter, the response of the structure subjected to ground motion and the results for two storied sloped building with ground inclination of 15°, 20° and 25° in terms of roof displacement, roof velocity and roof acceleration and base shear are presented. Also the storey displacement, story velocity and story acceleration for each inclination is illustrated. The responses due to ground motion as per spectra of IS 1893 (Part 1):2002 are shown. The results obtained based on numerical studies are shown with validation with experimental model.

4.10 Two storied sloped frame with ground inclination of 15°

With reference to the details in the article 3.2.3 and 4.2.4.3 by performing free vibration analysis we obtained the natural frequencies of the model for two different modes shown in table 4.3:

Table 4.3: Natural Frequency of sloped frame with 15° inclination validated with Present FEM

Type of Model	Natural Frequency (Hz)	
	Mode 1	Mode 2
Experimental	2.05	5.80
Present FEM	2.2283	6.1679

`

Table 4.4 shows maximum storey displacement (absolute) for both experimental and finite element and STAAD Pro. model for 15° slope.

Table 4.4: Maximum Storey Displacement (mm) for Experimental, Finite Element and STAAD model

Storey No.	Maximum Storey Displacement (mm)		
	Experimental	Present FEM	STAAD Pro.
1	55.2	52.43	54.4
2	76.6	77.3	80.2

Figure 4.10 shows Maximum Storey Displacement (Absolute) vs Storey Height for experimental and numerical model.

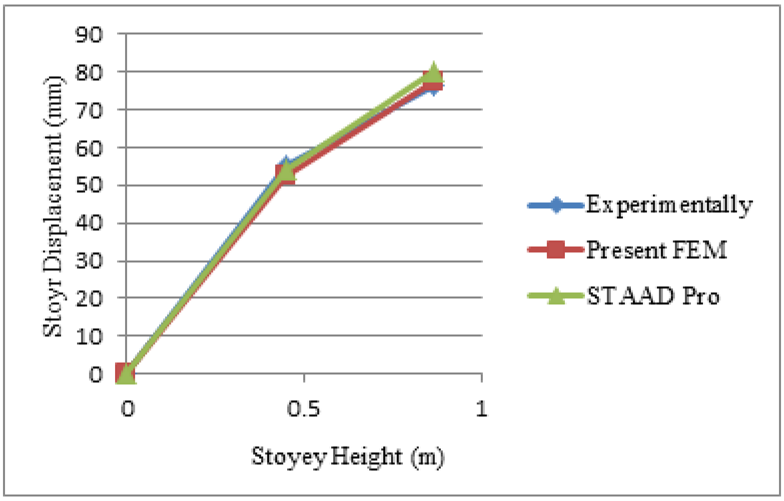

Figure 4.10: Storey Displacement vs Storey Height

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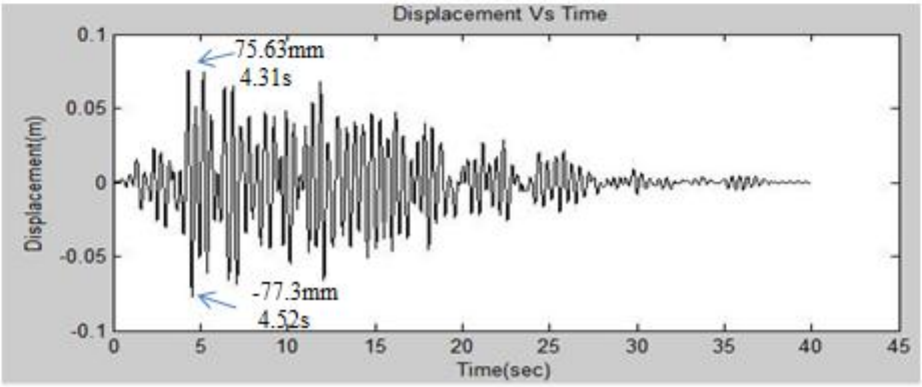
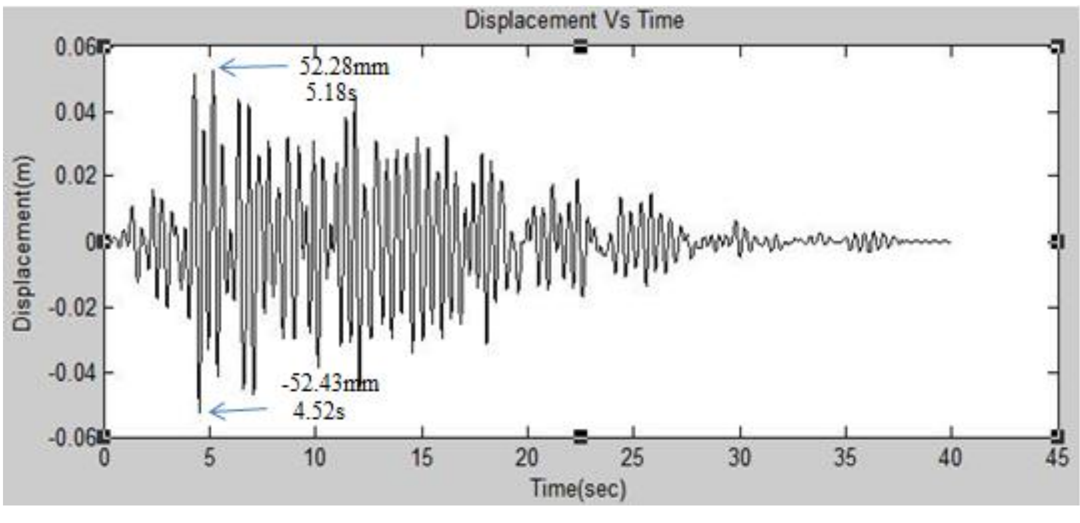
Figure 4.11 (a) and (b) and 4.12 (a) and (b) are the four plots shown for time history of top storey (roof) displacement and displacement of storey of 1st floor obtained in the numerical model i.e., Finite Element and STAAD Pro. model.



(a)

(b)

Figure 4.11: Time History of Top storey Displacement (a) Present FEM (b) STAAD Pro



(a)

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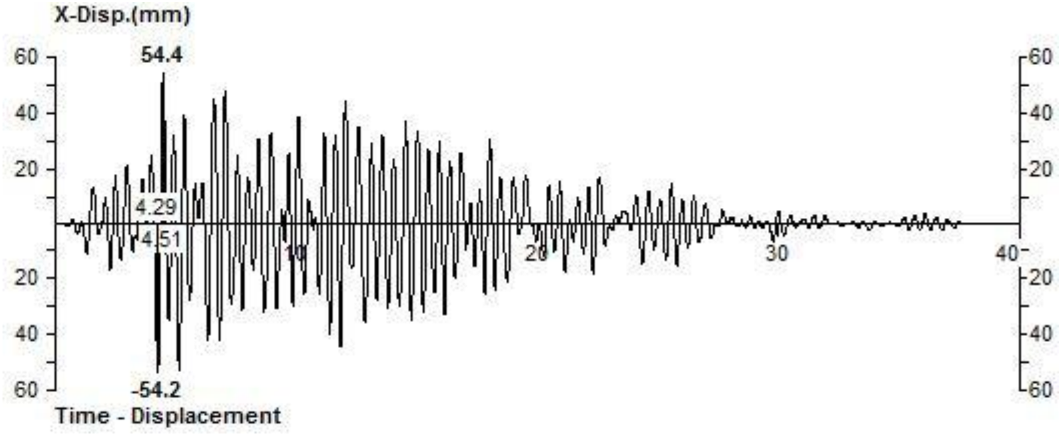

(b)

Figure 4.12: Time History of Storey (1st Floor) Displacement (a) Present FEM (b) STAAD Pro for 15° slope

Table 4.5 shows Maximum storey velocity (Absolute) for both Finite Element and STAAD Pro. model for 15° slope.

Table 4.5: Storey Velocity (mm/s) for Present FEM and STAAD model

Storey No.	Maximum Storey Velocity (mm/s)	
	Present FEM	STAAD Pro.
1	733.8	751
2	1133	1169

Figure 4.13 for Absolute Maximum Storey velocity (mm/s) vs Storey Height (m) for Present FEM and STAAD Pro model

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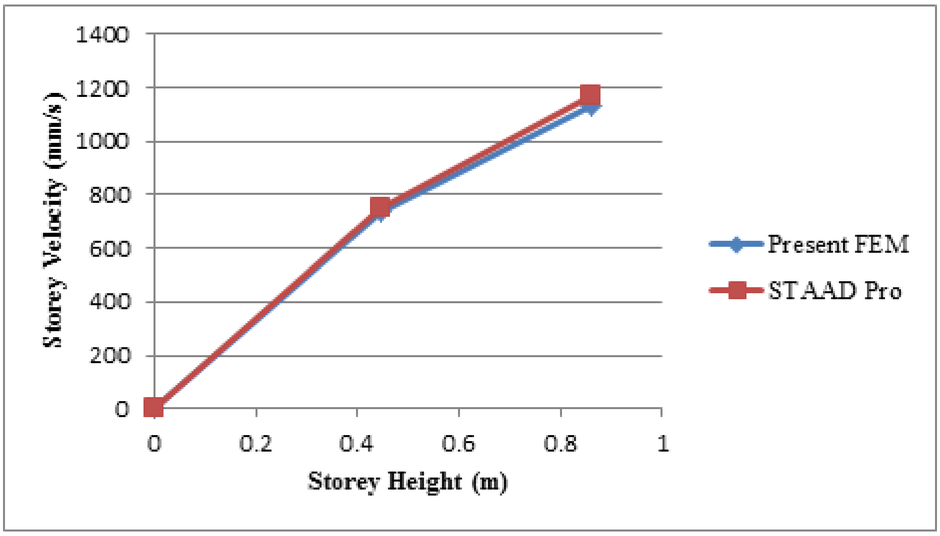
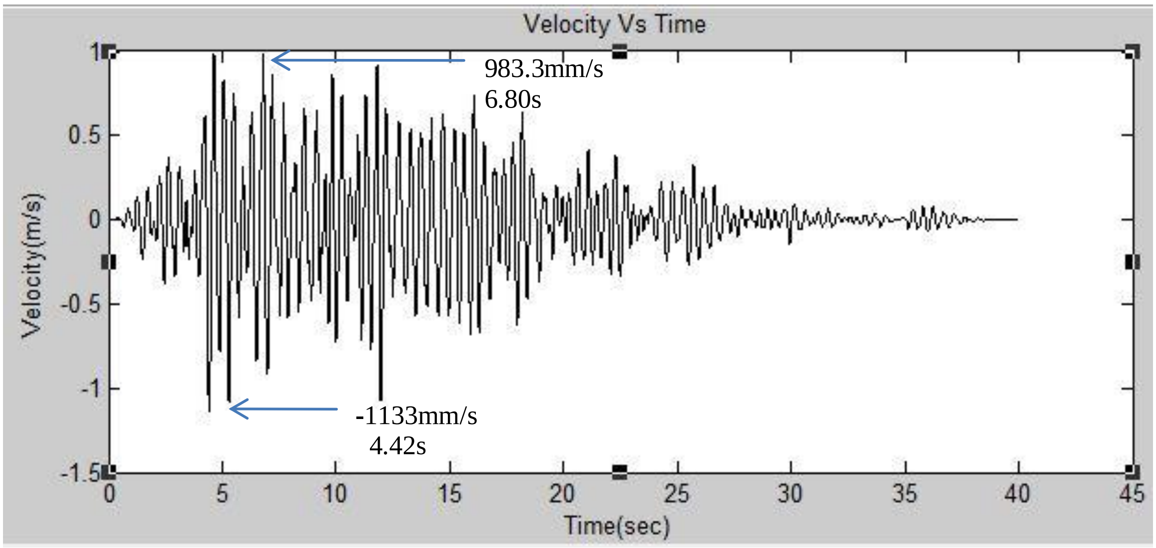


Figure 4.13: Storey Velocity vs Storey Height for 15° slope

Figure 4.14 (a) and (b) and Figure 4.15 (a) and (b) are the four plots shown for time history of top storey (roof) velocity and velocity of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.



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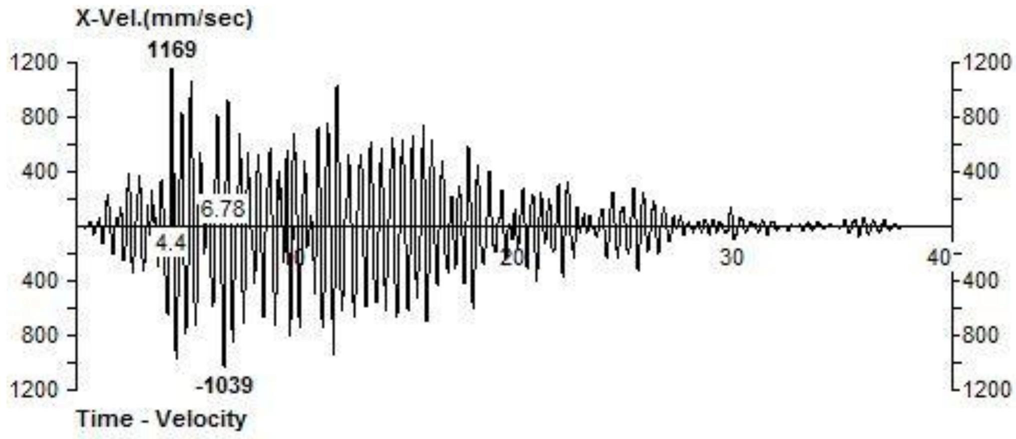
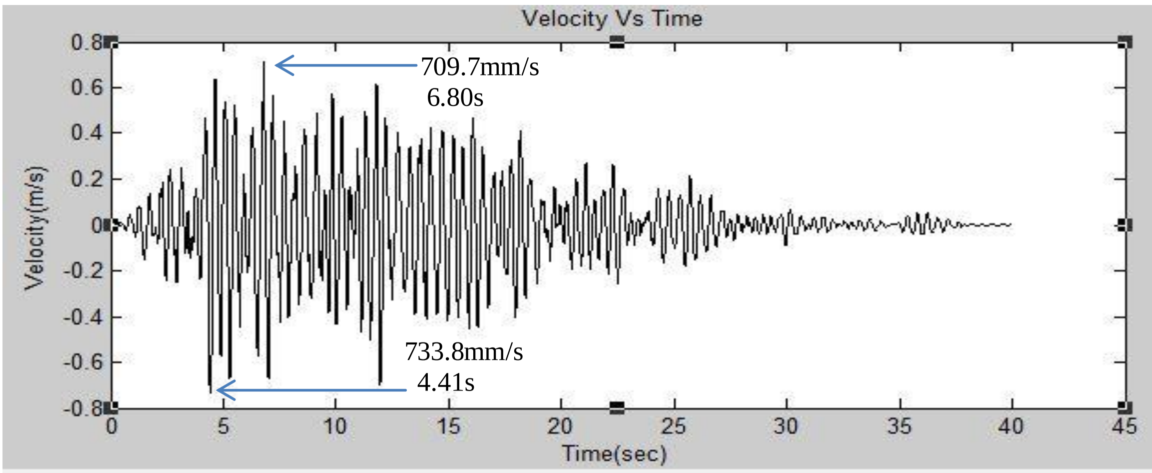


(b)

Figure 4.14: Time History of Top Storey Velocity (a) Present FEM (b) STAAD Pro for 15° slope



(a)

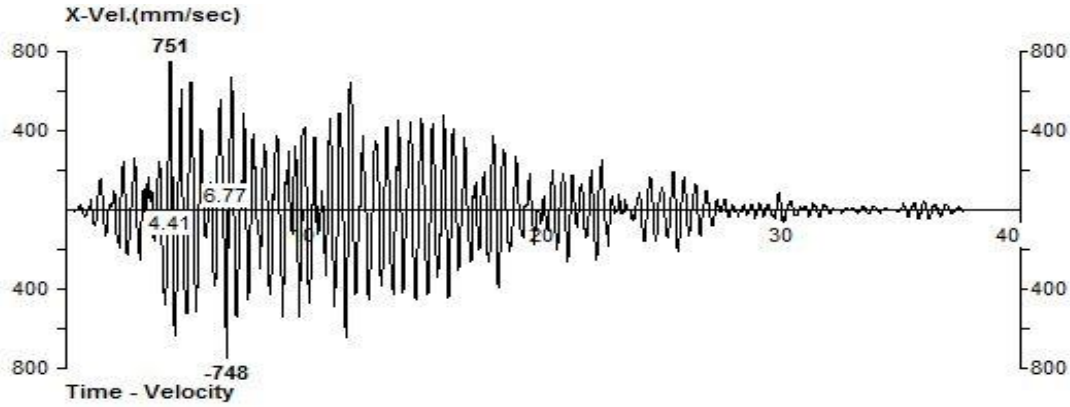


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Figure 4.15: Time History of Storey (1^{st} Floor) Velocity (a) Present FEM (b) STAAD Pro for 15° slope

Table 4.6 shows Maximum storey acceleration (Absolute) for both Finite Element and STAAD Pro. model for 15° slope

Table 4.6: Maximum Storey Acceleration (m/s^2) for Present FEM and STAAD model

Storey No.	Maximum Storey Acceleration (m/s^2)	
	Present FEM	STAAD Pro.
1	15.06	14.9
2	21.08	21.9

Figure 4.16 shows Absolute Maximum Storey Acceleration (m/s^2) vs Storey Height (m) for Present FEM and STAAD Pro. model

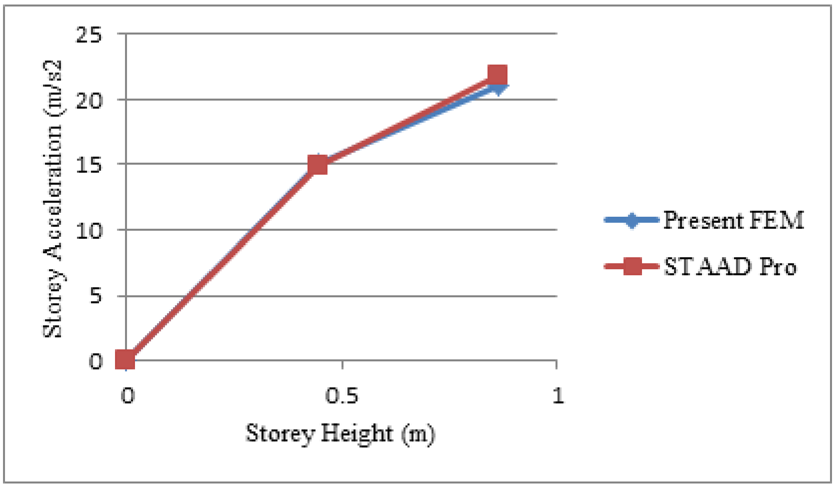

Figure 4.16: Storey Acceleration vs Storey Height

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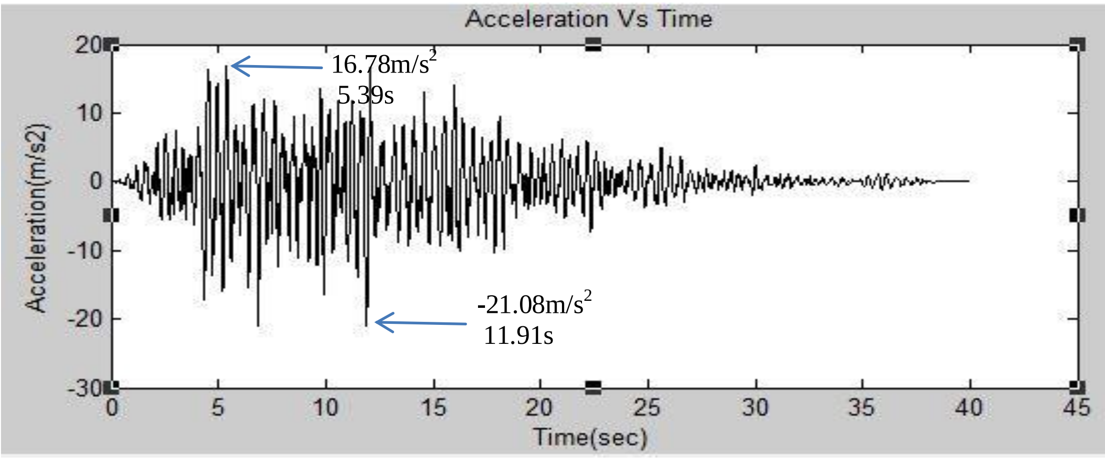
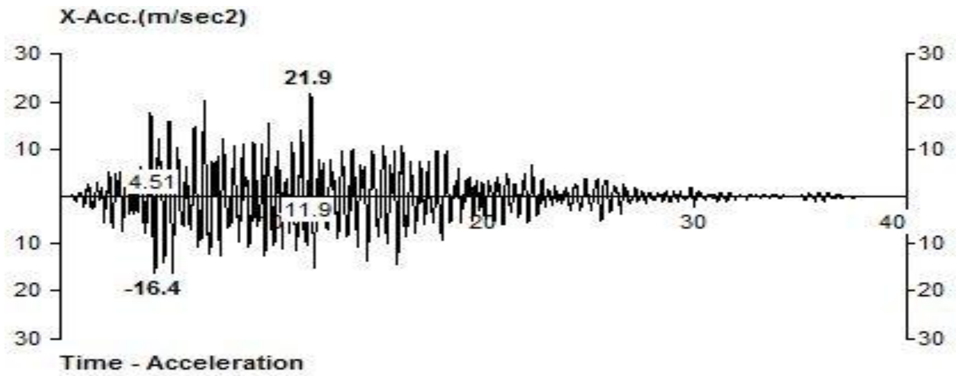
Figure 4.17 (a) and (b) and Figure 4.18 (a) and (b) are the four plots shown for time history of top storey (roof) acceleration and acceleration of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.

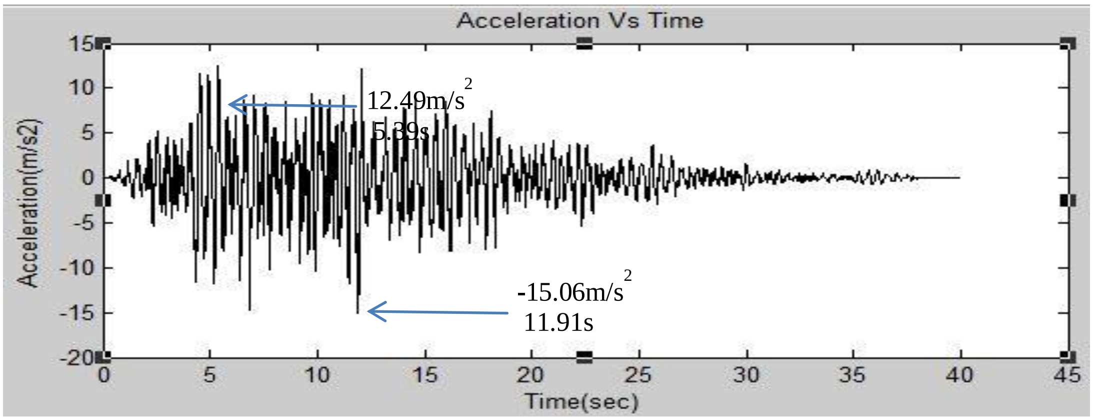


(a)



(b)

Figure 4.17: Time History of Top Storey Acceleration (a) Present FEM (b) STAAD Pro for 15° slope



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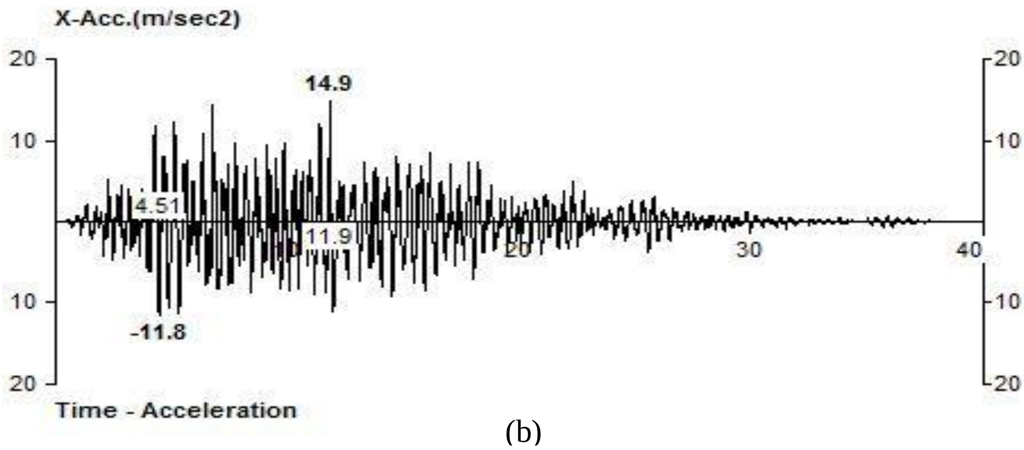
(a)



(b)

Figure 4.18: Time History of Storey (1st Floor) Acceleration (a) Present FEM (b) STAAD Pro for 15° slope

Table 4.7 showing Maximum Base Shear (Absolute) (N) of frame with respect to Finite Element and STAAD Pro. model.

Table 4.7: Maximum Base Shear (N) (Absolute) for Present FEM and STAAD model

Model	Maximum Base Shear (N)
Present FEM	393.6
STAAD Pro.	389.97

Figure 4.19 shows time history of base shear for FEM model for 15° slope

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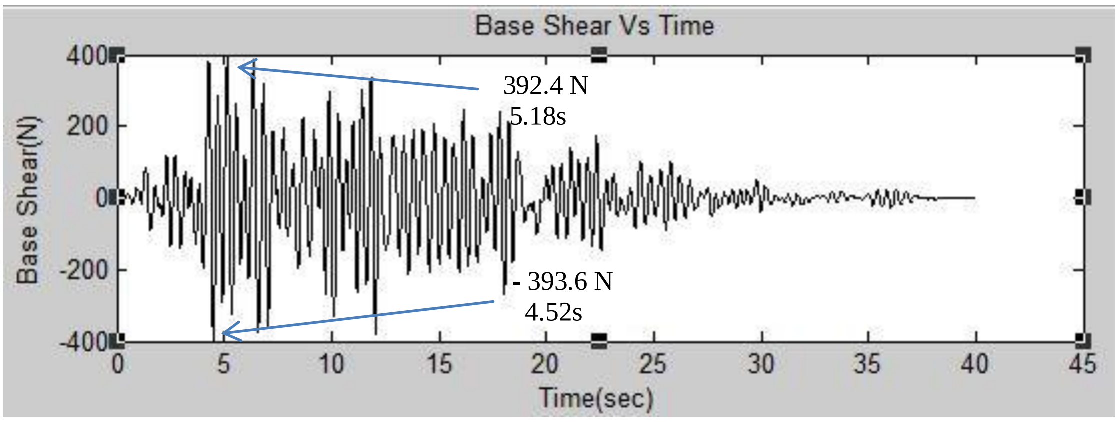

Figure 4.19: Time History of Base Shear for 15° slope

4.11 Two storied sloped frame with ground inclination of 20°

With reference to the details in the article 3.2.3 and 4.2.4.3 by performing free vibration analysis we obtained the natural frequencies of the model for two different modes shown in table 4.8:

Table 4.8: Natural Frequency of sloped frame with 20° inclination

Type of Model	Natural Frequency (Hz)	
	Mode 1	Mode 2
Experimental	2.2	5.945
Present FEM	2.38	6.303

`

Table 4.9 shows maximum storey displacement (absolute) for both experimental and finite element and STAAD Pro. model for 20° slope

Table 4.9: Maximum Storey Displacement (mm) for Experimental, Finite Element and STAAD Pro. model

Storey No.	Maximum Storey Displacement (mm)		
	Experimental	Present FEM	STAAD Pro.
1	44	44.58	46.8
2	68.3	70.57	73.7

Figure 4.20 shows Maximum Storey Displacement (Absolute) vs Storey Height for experimental and numerical model.

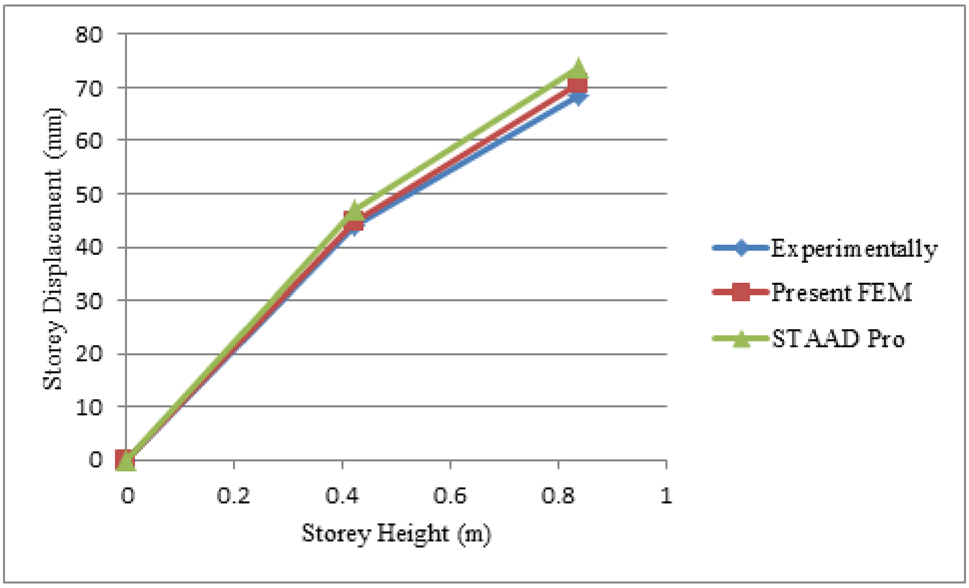

Figure 4.20: Storey Displacement vs Storey Height for 20° slope

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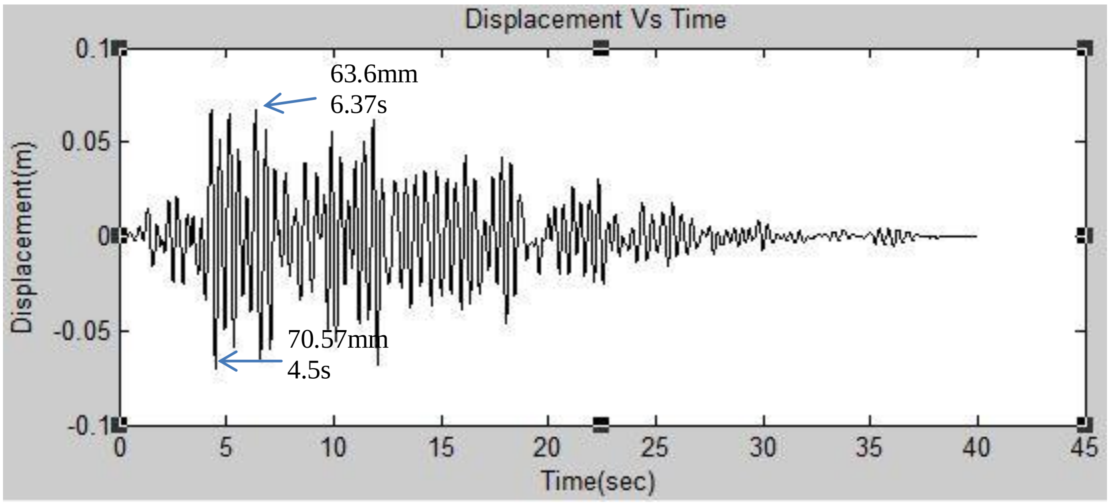
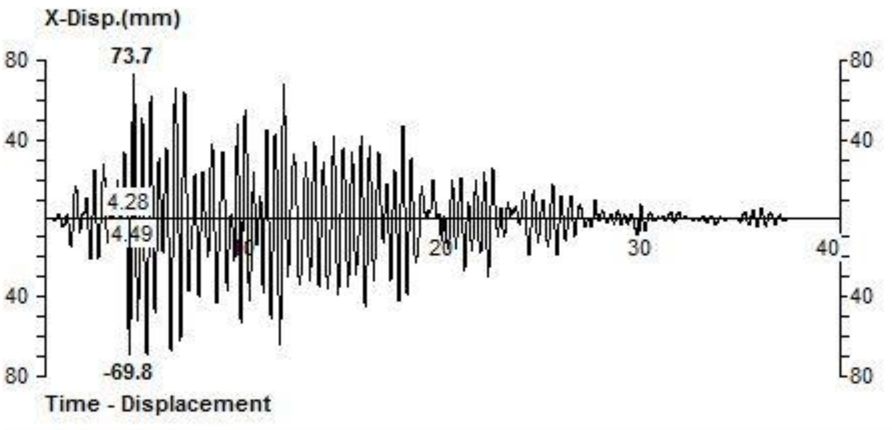
Figure 4.21 (a) and (b) and 4.22 (a) and (b) are the four plots shown for time history of top storey (roof) displacement and displacement of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.



(a)



(b)

Figure 4.21: Time History of Top storey Displacement (a) Present FEM (b) STAAD Pro for 20° slope

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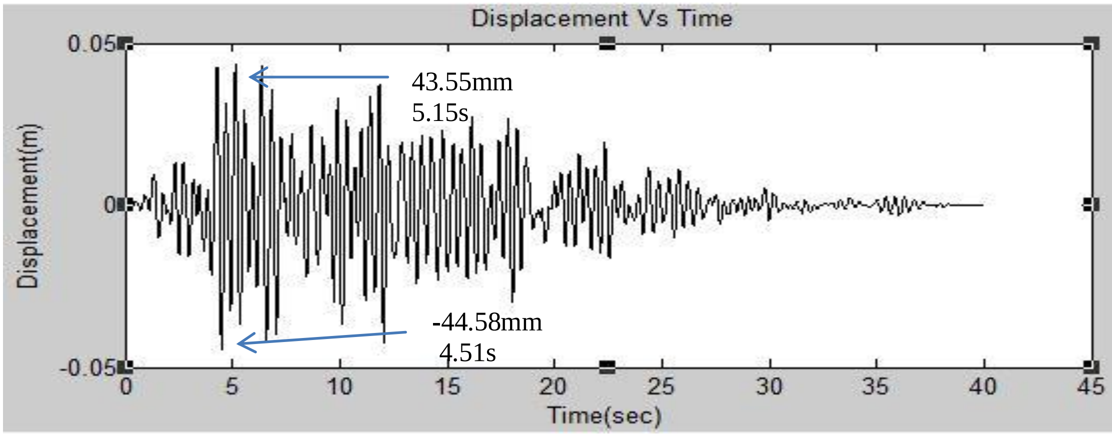
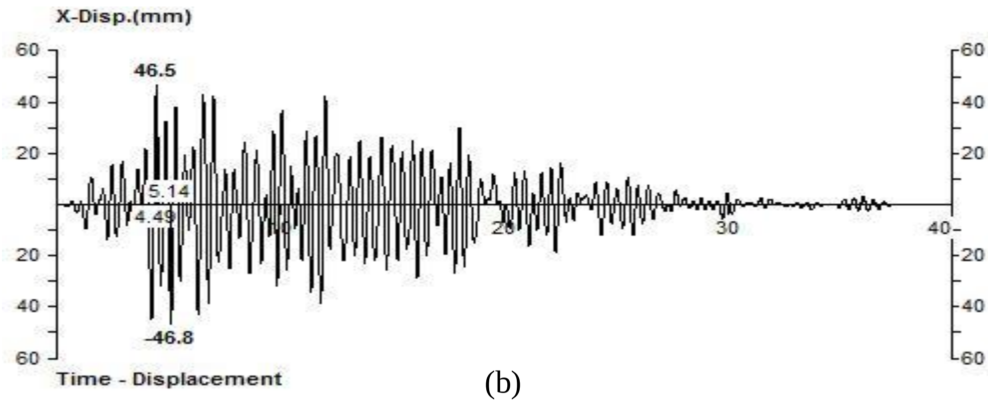


(a)



(b)

Figure 4.22: Time History of Storey (1st Floor) Displacement (a) Present FEM (b) STAAD Pro for 20° slope

Table 4.10 shows Maximum storey velocity (Absolute) for both Finite Element and STAAD Pro. model for 20° slope

Table 4.10: Maximum Storey Velocity (mm/s) for Present FEM and STAAD model

Storey No.	Maximum Storey Velocity (mm/s)	
	Present FEM	STAAD Pro.
1	720.5	697
2	1145	1134

`

Figure 4.23 for Absolute Maximum Storey velocity (mm/s) vs Storey Height (m) for Present FEM and STAAD Pro model

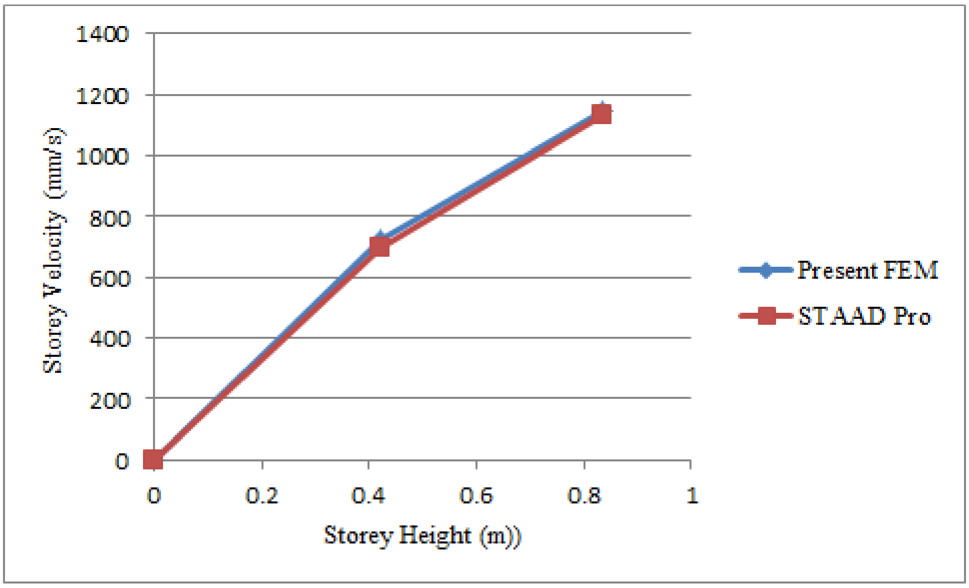
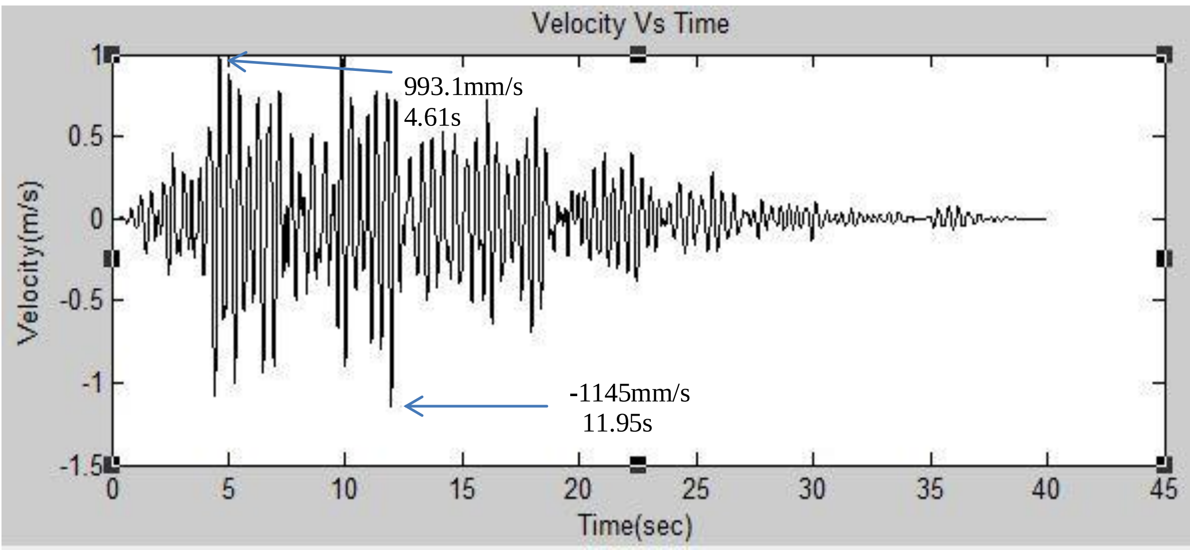


Figure 4.23: Storey Velocity vs Storey Height for 20° slope

Figure 4.24 (a) and (b) and 4.25 (a) and (b) are the four plots shown for time history of top storey (roof) velocity and velocity of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.



(a)

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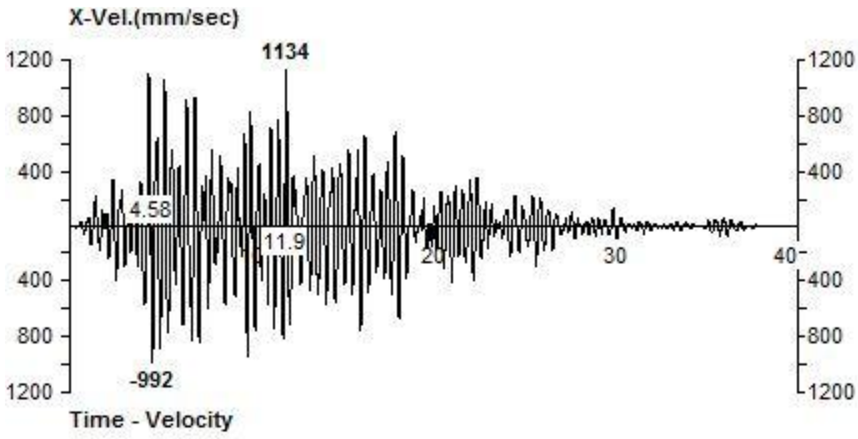
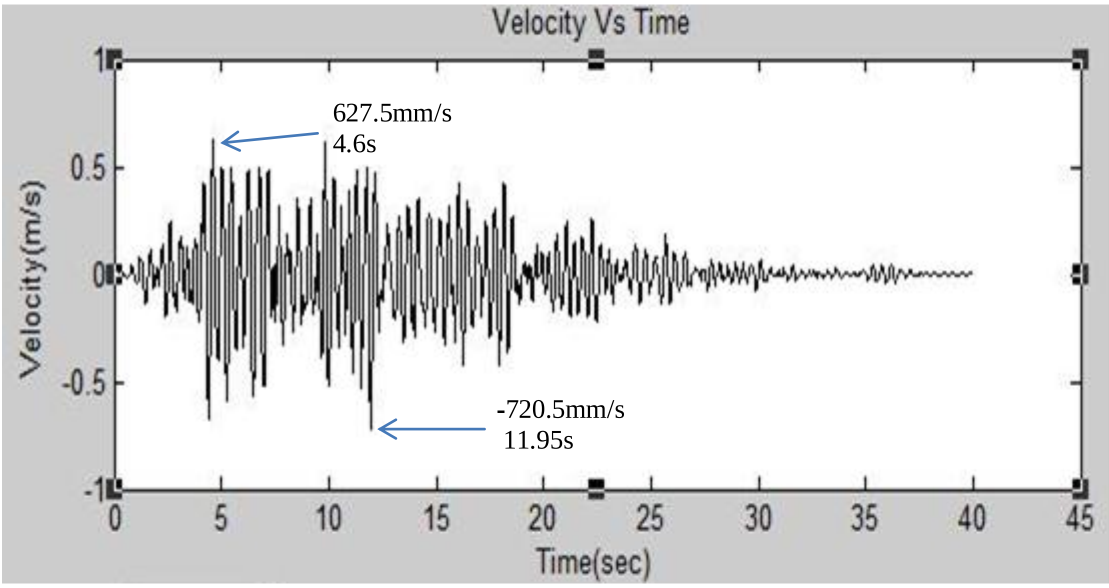


(b)

Figure 4.24: Time History of Top Storey Velocity (a) Present FEM (b) STAAD Pro for 20° slope



(a)

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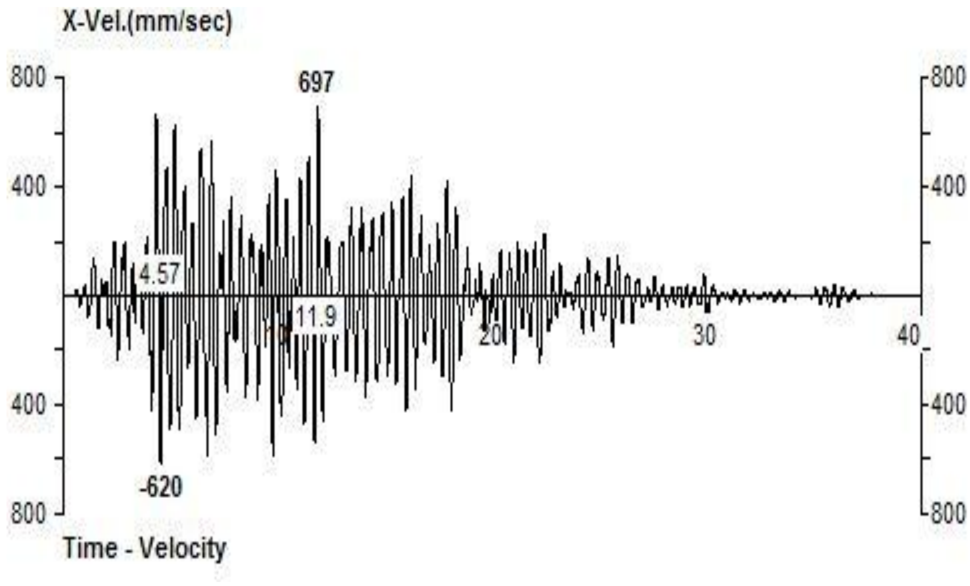

(b)

Figure 4.25: Time History of Storey (1st Floor) Velocity (a) Present FEM (b) STAAD Pro for 20° slope

Table 4.11 shows Maximum storey acceleration (Absolute) for both Finite Element and STAAD Pro. model for 20° slope

Table 4.11: Maximum Storey Acceleration (m/s^2) for Present FEM and STAAD model

Storey No.	Maximum Storey Acceleration (m/s^2)	
	Present FEM	STAAD Pro.
1	14.13	14.8
2	20.4	21.1

`

Figure 4.26 shows Absolute Maximum Storey Acceleration (m/s^2) vs Storey Height (m) for Present FEM and STAAD Pro. model

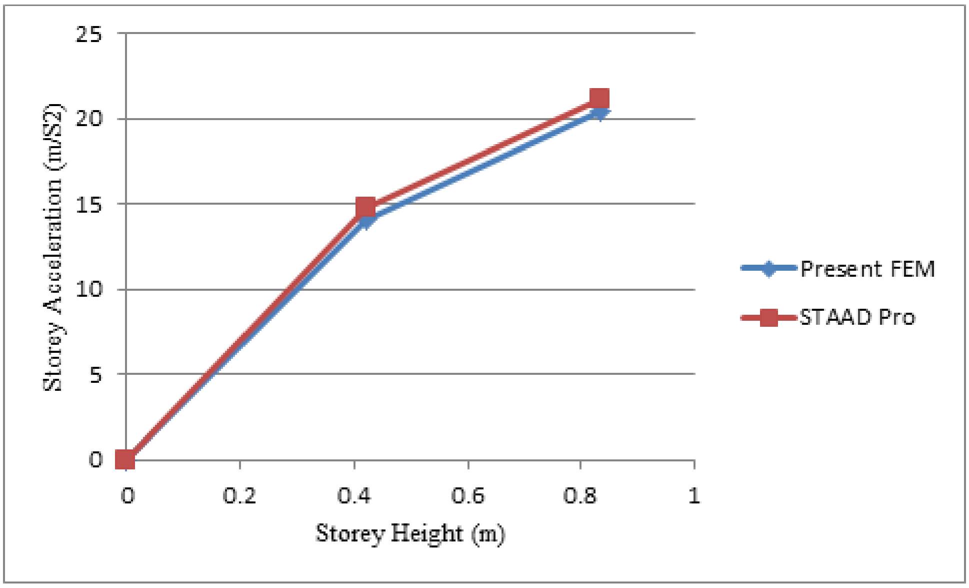

Figure 4.26: Storey Acceleration vs Storey Height for 20° slope

Figure 4.27 (a) and (b) and 4.28 (a) and (b) are the four plots shown for time history of top storey (roof) acceleration and acceleration of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model

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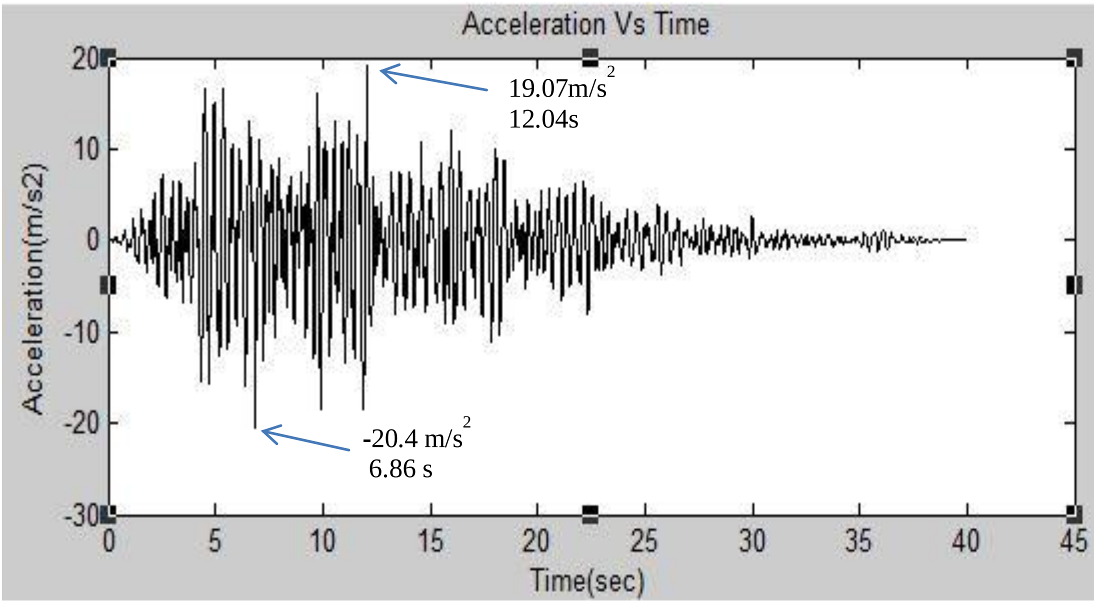
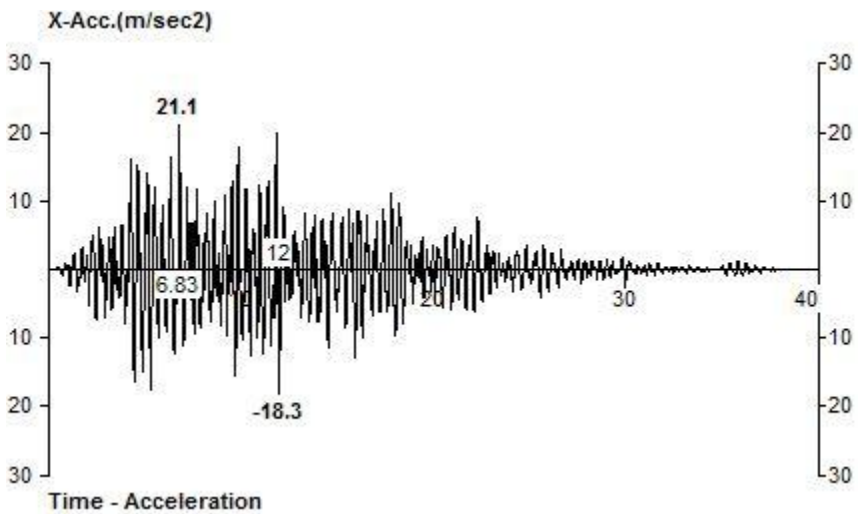


(a)



(b)

Figure 4.27: Time History of Top Storey Acceleration (a) Present FEM (b) STAAD Pro for 20° slope

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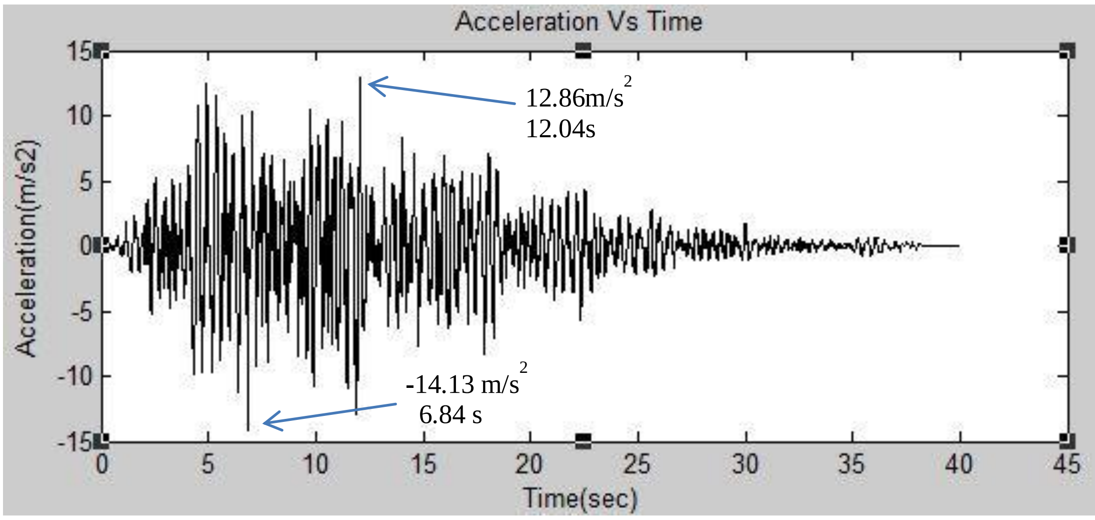
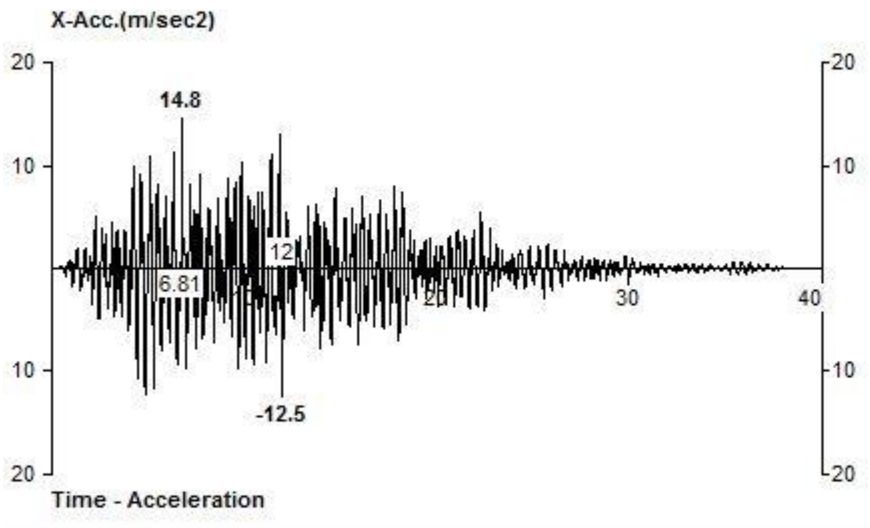


(a)



(b)

Figure 4.28: Time History of Storey (1st Floor) Acceleration (a) Present FEM (b) STAAD Pro for 20° slope

Table 4.12 showing Maximum Base Shear (Absolute) (N) of frame with respect to Finite Element and STAAD Pro. model

Table 4.12: Maximum Base Shear (N) (Absolute) for Present FEM and STAAD model

`

Model	Maximum Base Shear (N)
Present FEM	400
STAAD Pro.	401.14

Figure 4.29 shows time history of base shear for FEM model for 20° slope

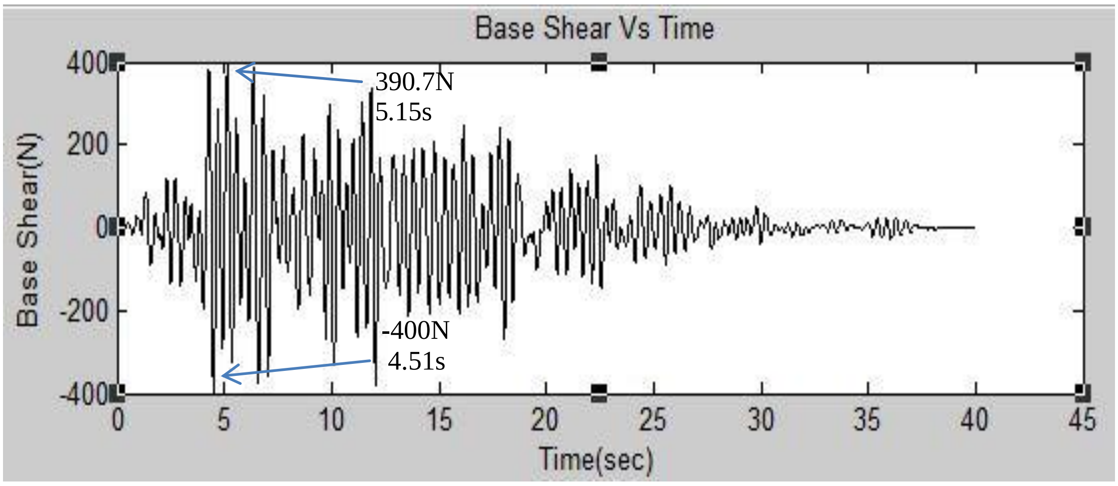


Figure 4.29: Time History of Base Shear

4.12 Two storied sloped frame with ground inclination of 25°

With reference to the details in the article 3.2.3 and 4.2.4.3 by performing free vibration analysis we obtained the natural frequencies of the model for two different modes shown in table 4.13:

Table 4.13: Natural Frequency of sloped frame with 25° inclination

`

Type of Model	Natural Frequency (Hz)	
	Mode 1	Mode 2
Experimental	2.6	6.55
Present FEM	2.688	6.6602

Table 4.14 shows maximum storey displacement (absolute) for both experimental and finite element and STAAD Pro. model for 25° slope

Table 4.14: Maximum Storey Displacement (mm) for Experimental, Finite Element and STAAD model

Storey No.	Maximum Storey Displacement (mm)		
	Experimental	Present FEM	STAAD Pro.
1	32.9	31.46	31.8
2	58.3	59.15	59.4

Figure 4.30 shows Maximum Storey Displacement (Absolute) vs Storey Height for experimental and numerical model.

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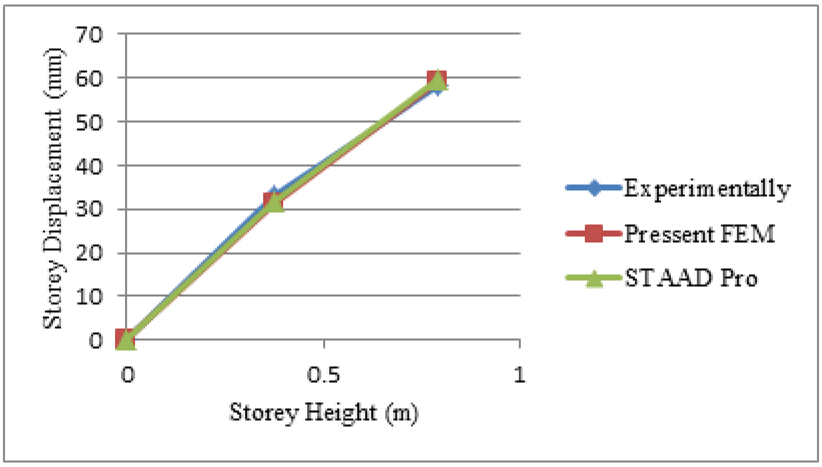
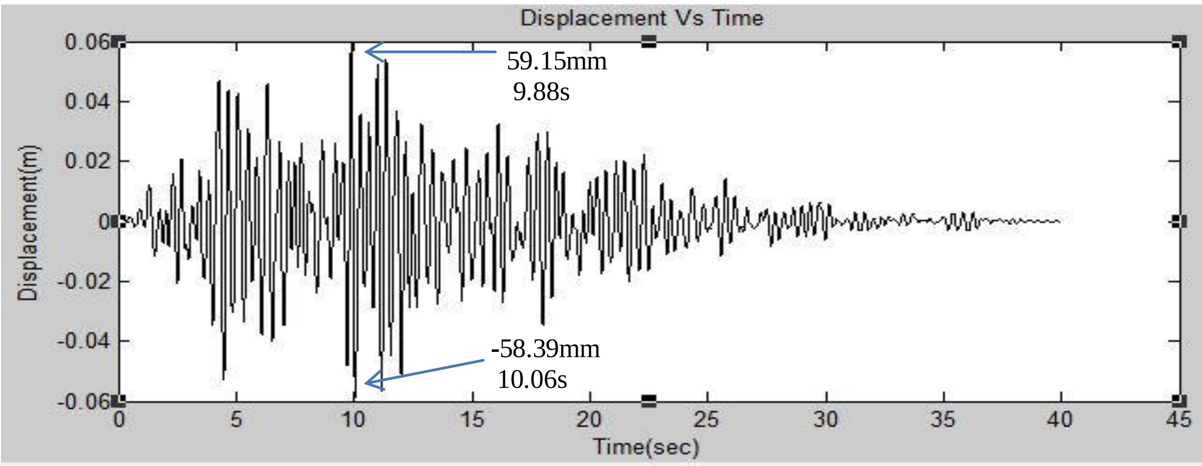
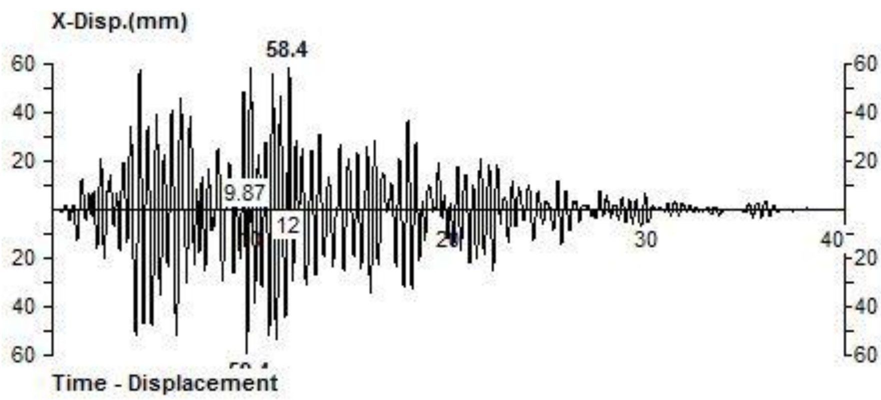


Figure 4.30: Storey Displacement vs Storey Height for 25° slope

Figure 4.31 (a) and (b) and 4.32 (a) and (b) are the four plots shown for time history of top storey (roof) displacement and displacement of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.



(a)



(b)

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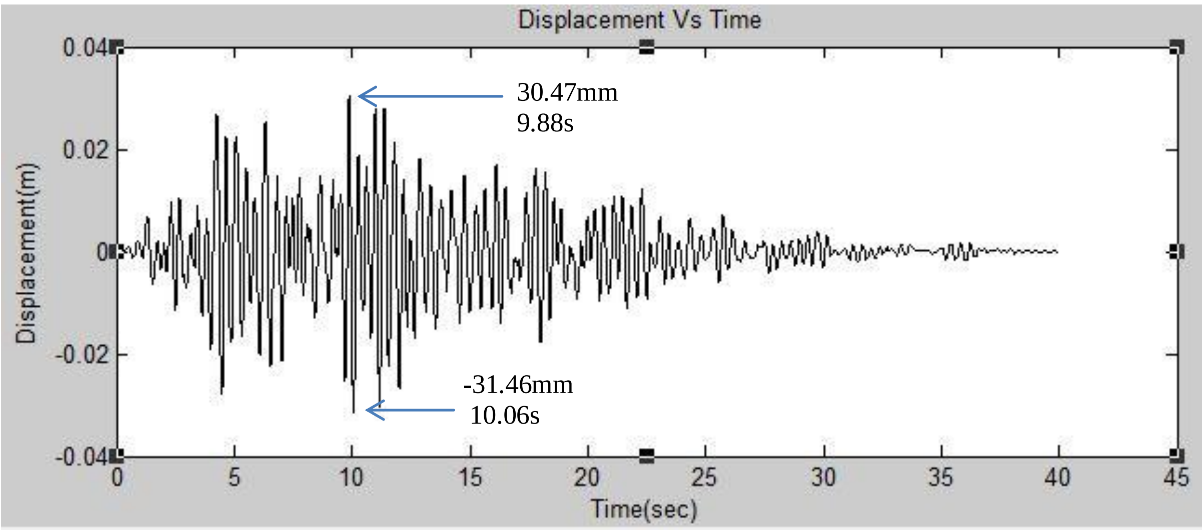
Figure 4.31: Time History of Top storey Displacement (a) Present FEM (b) STAAD Pro for 25° slope



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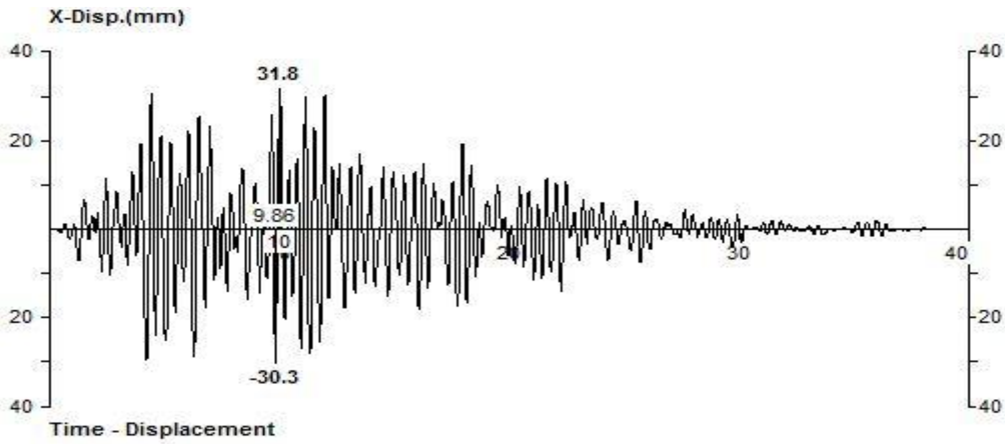

(b)

Figure 4.32: Time History of Storey (1st Floor) Displacement (a) Present FEM (b) STAAD Pro for 25° slope

Table 4.15 shows Maximum storey velocity (Absolute) for both Finite Element and STAAD Pro. model for 25° slope

Table 4.15: Maximum Storey Velocity (mm/s) for Present FEM and STAAD model

`

Storey No.	Maximum Storey Velocity (mm/s)	
	Present FEM	STAAD Pro.
1	582	550
2	1146	1111

Figure 4.13 for Absolute Maximum Storey velocity (mm/s) vs Storey Height (m) for Present FEM and STAAD Pro model.

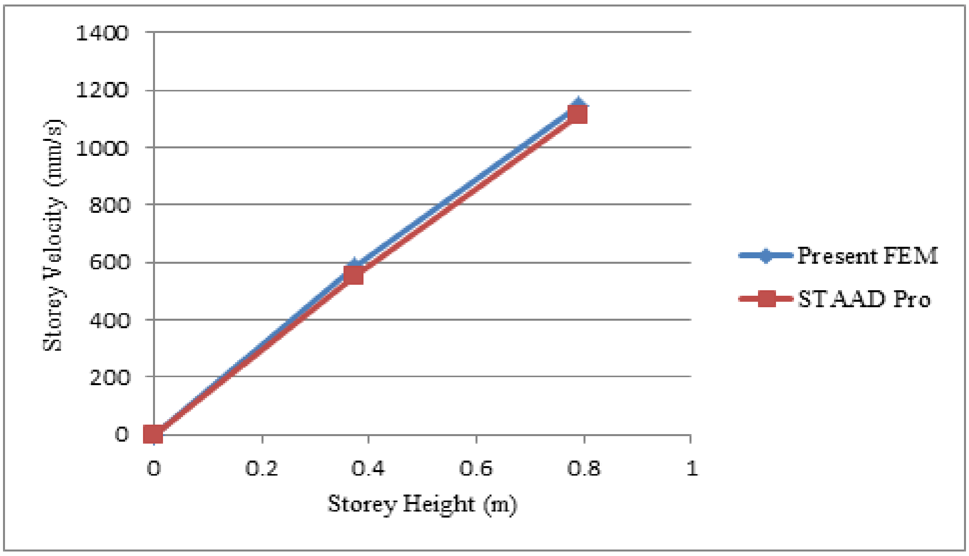

Figure 4.33: Storey Velocity vs Storey Height for 25° slope

Figure 4.34 (a) and (b) and 4.35 (a) and (b) are the four plots shown for time history of top storey (roof) velocity and velocity of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.

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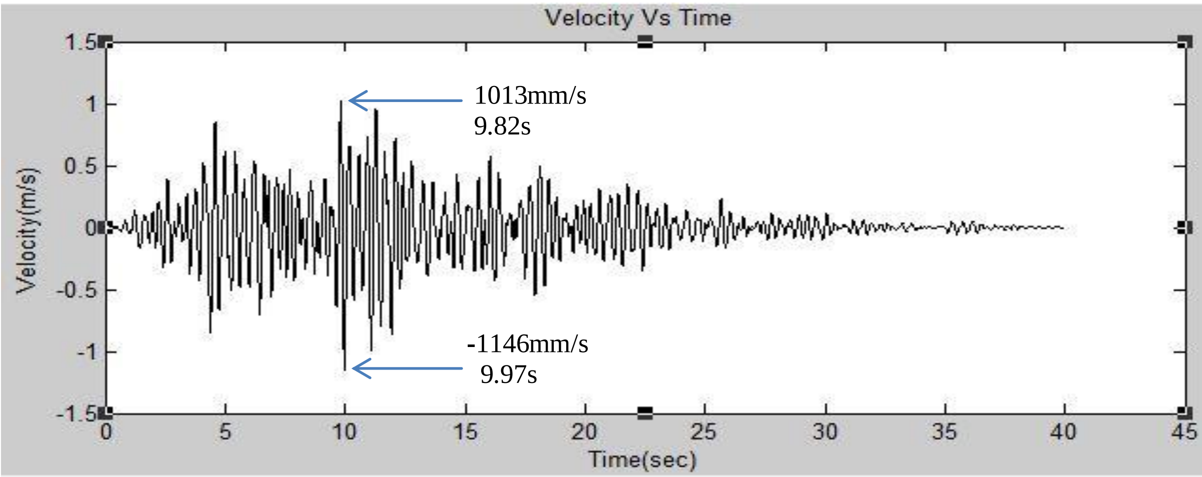

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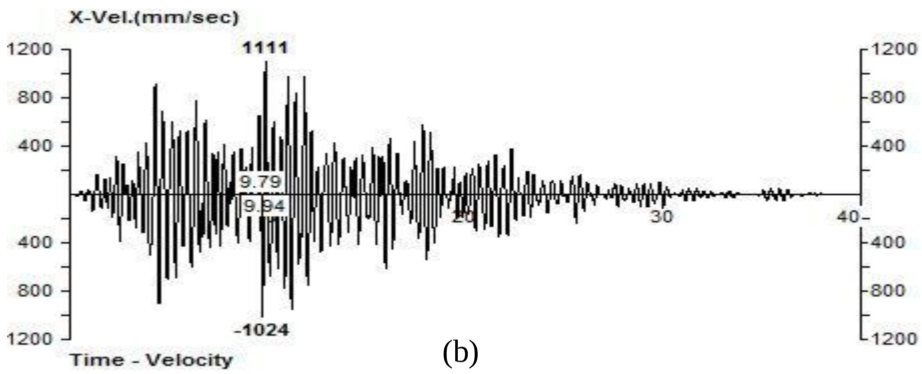
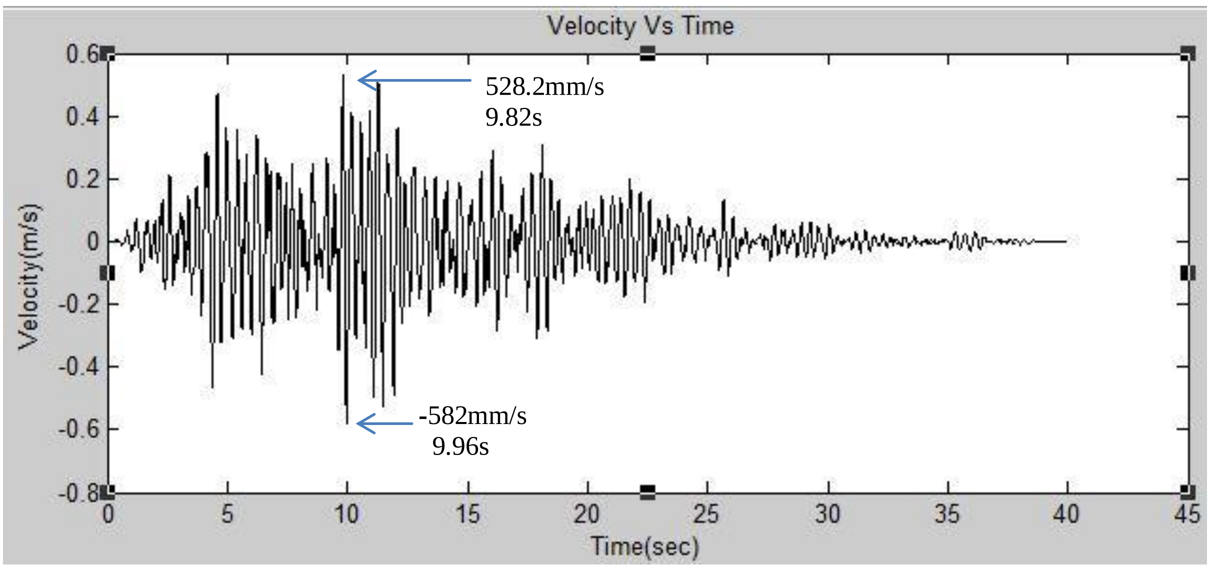


(b)

Figure 4.34: Time History of Top Storey Velocity (a) Present FEM (b) STAAD Pro for 25° slope



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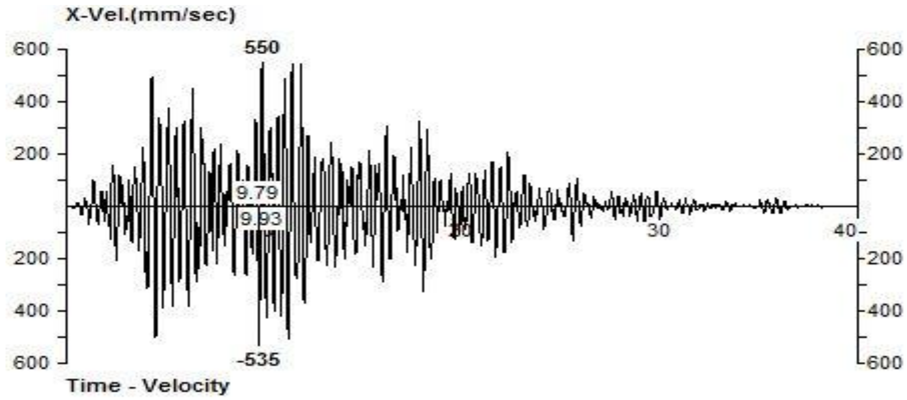
(a)



(b)

Figure 4.35: Time History of Storey (1st Floor) Velocity (a) Present FEM (b) STAAD Pro for 25° slope

Table 4.16 shows Maximum storey acceleration (Absolute) for both Finite Element and STAAD Pro. model for 25° slope

Table 4.16: Maximum Storey Acceleration (m/s^2) for Present FEM and STAAD model

Storey No.	Maximum Storey Acceleration (m/s^2)	
	Present FEM	STAAD Pro.
1	11.57	11
2	20.67	20.5

Figure 4.36 shows Absolute Maximum Storey Acceleration (m/s^2) vs Storey Height (m) for Present FEM and STAAD Pro. model

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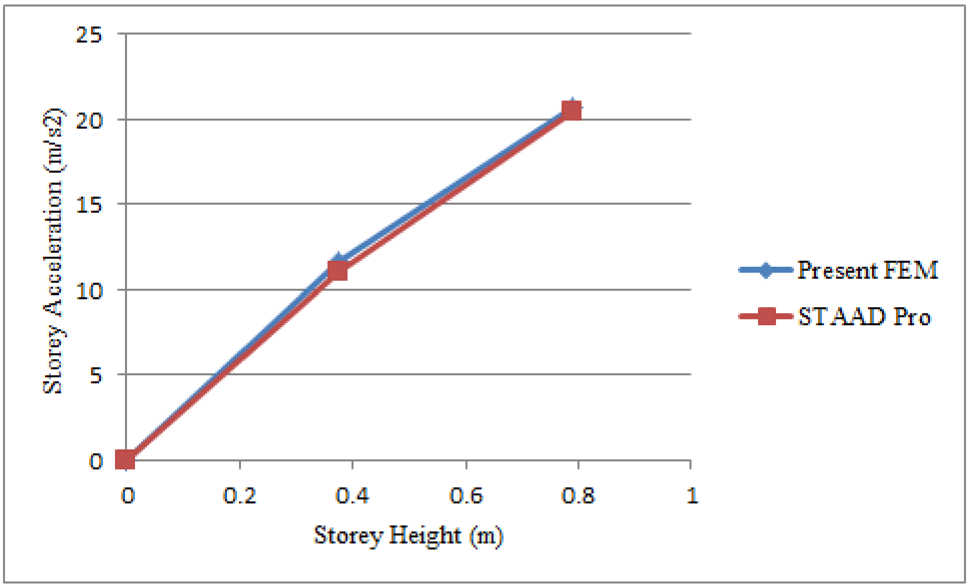
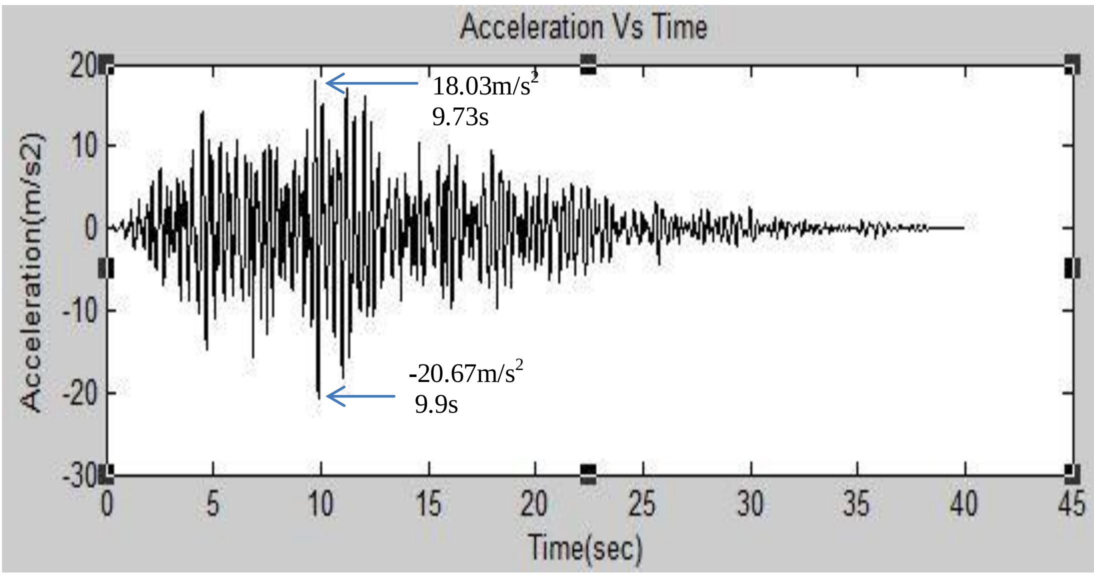


Figure 4.36: Storey Acceleration vs Storey Height for 25° slope

Figure 4.37 (a) and (b) and 4.38 (a) and (b) are the four plots shown for time history of top storey (roof) acceleration and acceleration of storey of 1st floor obtained in the numerical i.e., Finite Element model and STAAD Pro. model.



(a)

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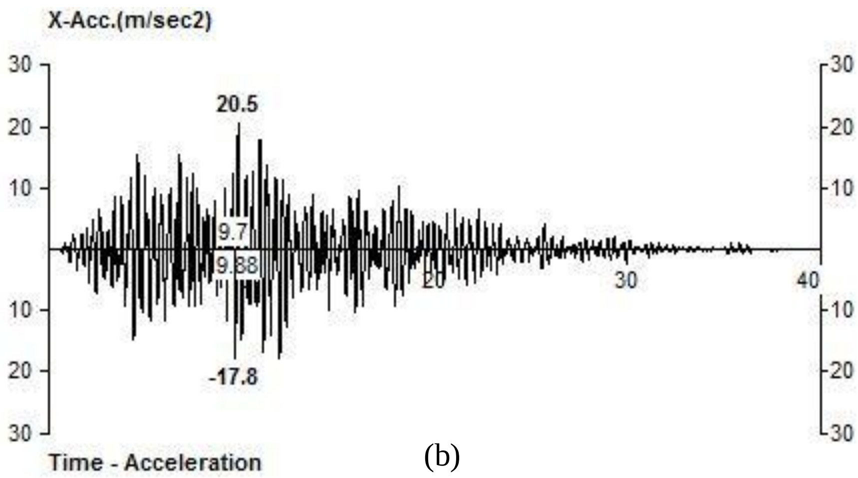
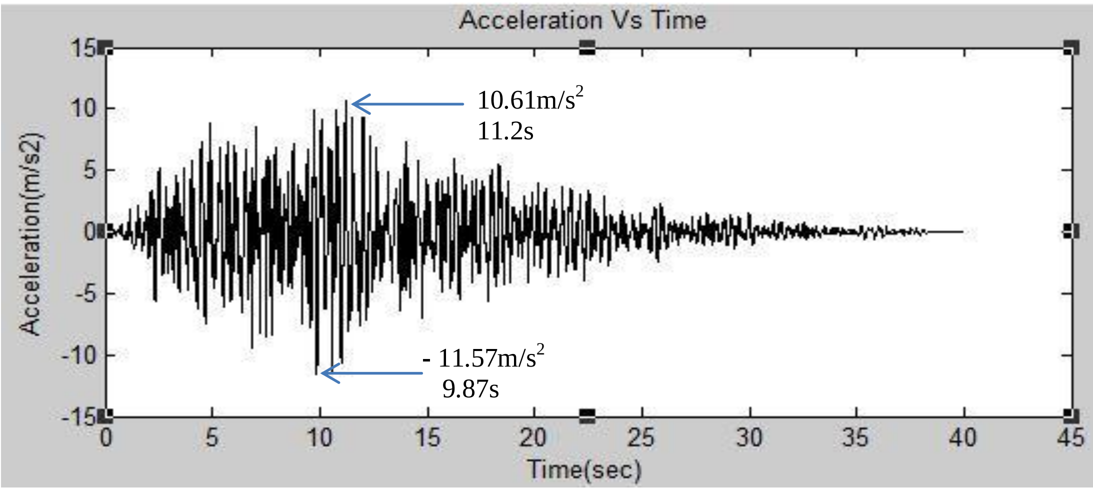
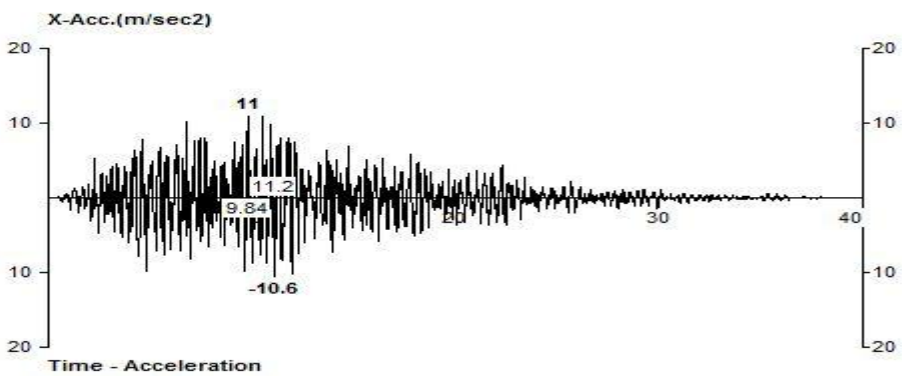


(b)

Figure 4.37: Time History of Top Storey Acceleration (a) Present FEM (b) STAAD Pro for 25° slope



(a)



(b)

Figure 4.38: Time History of Storey (1st Floor) Acceleration (a) Present FEM (b) STAAD Pro for 25° slope

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Table 4.17 shows Maximum Base Shear (Absolute) of frame with respect to Finite Element and STAAD Pro. model

Table 4.17: Maximum Base Shear (N) (Absolute) for Present FEM and STAAD model

Model	Maximum Base Shear (N)
Present FEM	400.7
STAAD Pro.	387.21

Figure 4.39 shows time history of base shear for FEM model for 25° slope

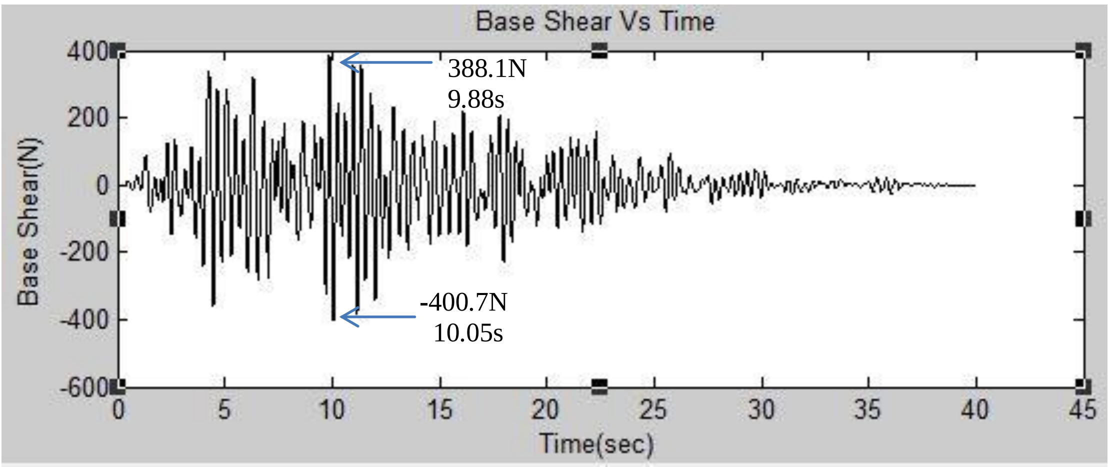

Figure 4.39: Time History of Base Shear

4.13 Mass Participation factor of both modes for considered slope angles

In the analysis of structures, the number of modes considered should have at least 90% of the total seismic mass as per IS 1893-2002 (Part I). Table 4.18 shows that the number of modes considered here are satisfying the criteria. The Mass participation factor (%) for both modes 1 and 2 and all the three slope inclination is tabulated and it is observed that the mass participation factor decreases with increase in slope inclination.

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Table 4.18: Mass Participation Factor (%) of both modes for different slope angle

Slope angle	Mass Participation Factor (%)	
	Mode 1	Mode 2
15°	96.40	3.60
20°	95.08	4.92
25°	91.33	8.67

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Chapter 5

SUMMARY

AND

CONCLUSIONS

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SUMMARY AND CONCLUSIONS

5.1 Summary

Earthquake is caused when it is subjected to the ground motion and due to which structures suffers damage and to take care of such effects it is important to know the properties of earthquake and predicts its possible response which can incur on the buildings. These properties are base shear, maximum storey displacement, velocity and acceleration, etc.

In this study, such analysis has been done experimentally with validation in structural analysis tool and finite element modeling to know the response of building mentioned above. The responses for each slope angle is studied and compared.

5.2 Conclusions

Following conclusions can be drawn for the three sloped frame model from the results obtained in analysis:

- 15 degree sloped frame experiences maximum storey displacement due to low value of stiffness of short column while the 25 degree frame experiences minimum storey displacement.
- 15 degree sloped frame experiences nearly the same storey velocity as of 20 degree and 25 degree in the top storey but the velocity is maximum for the storey level of first floor while for 25 degree frame velocity is minimum for level of first floor.
- 15 degree sloped frame experiences maximum storey acceleration for the top floor with little variations with the 20 degrees and 25 degrees model but for the storey level of the first floor, acceleration is maximum and is minimum for the storey level of the first floor for 25 degrees frame.

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- The natural frequencies of the sloped frame increases with the increase in the slope angle.
- The number of modes considered in the analysis is satisfying the codal provisions. The modal mass participation of the sloped frame model are decreasing for the first mode and increasing for the second mode with the increase in slope angle.
- For all the three frame models, time history response of the top floor acceleration is maximum at resonance condition i.e., when excitation frequency matches with fundamental frequency.
- The base shear of all the buildings are nearly the same with little variations but their distribution on columns of ground storey is such that the short column attracts the majority (75% approx.) of the shear force which leads to plastic hinge formation on the short column and are vulnerable to damage. Proper design criteria should be applied to avoid formation of plastic hinge.

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5.3 Future work

There is a scope for future work in this area of study. The analysis can be performed for varying frequency content i.e., for low, intermediate and high frequency content. In this study linear time history analysis is performed, one can also perform non-linear time history analysis for the sloped frame model.

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