ANALYTICAL MODELLING OF STUDIED TIME HISTORY RESPONSE OF STEEL FRAME BUILDING MODEL

FINAL YEAR PROJECT UG 2014

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It is certified that the

Final Year Project titled

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has been accepted towards the requirements

for the undergraduate degree

in

CIVIL ENGINEERING

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Prima facea, I am grateful to the God for the good health and wellbeing that were necessary to complete this document

We are highly grateful to our Project advisor Dr. Muhammad Rizwan Siddique for his continuous and effective guidance and support throughout the course of this project.

As our teacher and advisor, he has taught us more than we could ever give him credit for here. He has shown us, by his example, what a good project advisor (and person) should be.

We are highly obliged to mention Mr. Muhammad Qasim, MS Structural Engineer,

He has been a complete and efficient guide throughout our project.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Lateral loading due to bomb blast is instantaneous and more destructive in comparison to other lateral forces, it causes majority of the human losses and casualties [1]. Usually, when large displacements occur in a structure, severe damage may occur to dissipate the energy. Considering the frame as a primary structure lateral load resisting design endeavours to enhance ductility of the structure. Beams and columns remain the major contributing elements in this regard. The structure must be ductile enough to allow load capacity to be reached by beams and columns. From the extensive review of literature carried out, it has been found that comparison of experimental work with analytical work for the response of building to the instantaneous lateral displacement or forces has been carried out on a very small scale.

Several design philosophies are used by making use of computer simulations, experimental results and observations from past cases of lateral loadings to offer the required performance for the terroristic threats at the site of interest. In computer simulations, the detailed structural model subjected to a lateral load produces estimates of deformations in the model. The properties of the building response depend on the severity of the shaking due to loading. Among the two general methods of structural dynamic analysis (in time domain and frequency domain), time-series analysis provides a more rational option, with possible modifications for variable stiffness depending on load and deformation. [1]

1.2 PROBLEM STATEMENT

During an instantaneous loading, severe damage may occur in a moment resisting frame structure to dissipate the energy. Before designing the structural elements of the lateral load resisting system, a comprehensive analysis should be performed for the calculation of stresses in the elements under the instantaneous loading. This analysis can be performed using Finite Element Method (FEM) softwares. However, acceptability of commercially available FEM software is still under consideration. For this reason, comparison of experimental results and analytical results for a steel structure has been performed in this study. This study investigates the response of structure to instantaneous loading.

1.3 DOMAIN OF RESEARCH

The domain encompasses analysis of, how building structures exhibit dynamic response upon application of instantaneous loading. The major response parameter is top roof displacement. The scope of this investigation is as follow:

-Perform Experimental work to analyse as to how instantaneous loads create dynamic response in a steel structure

- Perform Analytical work with ETABS to compare the analytical results with experimental results for the dynamic response of steel structure to an instantaneous loading.

- Thorough collaboration of the results from ETABS and the steel structure will dictate the viability is this software to predict the actual behaviour of structures to instantaneous loading.

1.4 RESEARCH OBJECTIVES

Purpose of this study revolves around generating results which would help in the prediction of moment resisting steel structure’s performance using FEM software.

Additional objectives are:

- Create actual model of steel structure to investigate the actual response.
- Study the effects of instantaneous loading on important parameter such as top roof displacement.
- Use the research findings to establish the feasibility of using ETABS software for predicting the dynamic response of structure.

1.5 THESIS LAYOUT

The study during this thesis spans over six divisions or chapters as under:

Chapter 1 Introduction

Introduces the topic by defining the research problem, mentions the goals and aims and outlines the research scope.

Chapter 2 Literature Review

Gives a brief review on what is free-vibration, how damping effects the free-vibration after the release of loading and how can we develop numerical model of a damped system. It gives a gist of literature already published on the comparison of analytical work with experimental work.

Chapter 3 Research Methodology

This chapter provides step by step brief description about the methodology of research.

Chapter 4 Model Development

This chapter sheds light on the design of model using finite element method, for the analysis of steel structure. Material and size details for the structural elements have been provided in this chapter.

Chapter 5 Analysis and Results

In this chapter top roof displacements and storey displacements during each analysis was presented analytical results are compared with experimental results.

Chapter 6 Conclusion and Recommendations

Conclusions drawn from this study have been presented in this chapter. On the basis of conclusions, recommendations have been given on the use of FEM softwares to predict the actual behaviour of the structure.

CHAPTER 2

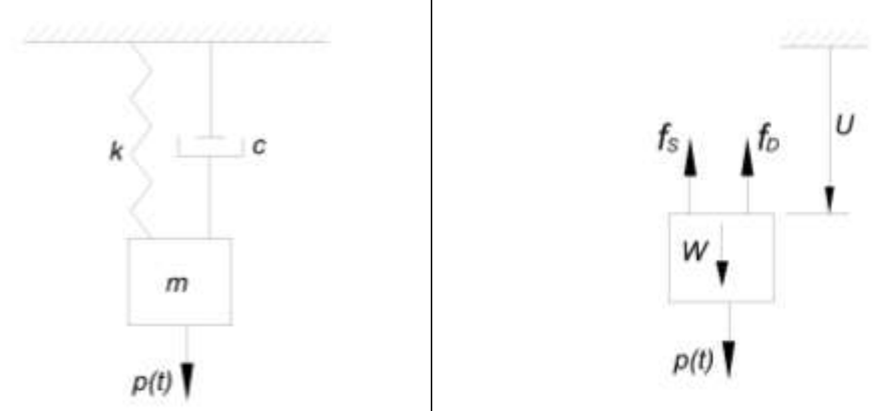
LITERATURE REVIEW

2.1 RESPONSE OF SDOF SYSTEMS

2.1.1 DAMPED SDOF SYSTEM

In the spring – mass system shown in the diagram below energy is being dissipated as a result of a damping force. Considering a damping constant c, it is assumed that damping force f_d and velocity of the mass are proportional. Upon application of' 2nd Law of motion the model depicting damping force leads to a linear equation (differential) as

$$m\ddot{u} + c\dot{u} + ku = p(t) \quad (2.1)$$



(1) (II)

Figure 2.2 : *(a) Mass-spring-dashpot SDOF system; (b) Free-body diagram of a mass-spring -dashpot SDOF system.*

2.1.2 VISCOUS-DAMPED SDOF SYSTEM RESPONSE TO FREE VIBRATION

When SDOF is subjected to excitation p(t) and to initial conditions, the velocity and displacement at time *t=0*, given by

$$u(0) = u_0 \qquad \dot{u}(0) = v_0 \quad (2.2)$$

where u_0 and v_0 are the specified initial displacement and initial velocity, respectively.

It will be easier for us if we divide Eq 2.1 by m and to write SDOF equation as fo

$$\ddot{u} + 2\zeta\omega_n\dot{u} + {\omega_n}^2 u = {\omega_n}^2 \frac{p(t)}{k} \tag{2.3}$$

where □$_n$ is defined by

$$\boxed{{\omega_n}^2 = \frac{k}{m}} \tag{2.4a}$$

and ζ is defined by

$$\boxed{\zeta = \frac{c}{c_{cr}}} \tag{2.4b}$$

where

$$c_{cr} = 2m\omega_n = 2\sqrt{km} \tag{2.4c}$$

The above equation defines how the vibration of linear SDOF systems is based on two important factors.

- ω_n is called the *un damped circular natural frequency* ((radians per seconds (rad/s)).
- ζ is viscous damping factor, it is dimensionless
- c_{cr}, is the *critical damping coefficient.*

Equation 3.3 is a linear ordinary differential equation having coefficients that are constant. The response given by above equation is based on two parts

- a forced motion $u_p(t)$, related directly to $p(t)$,
- Natural motion $u_c(t)$, so that arbitrary initial conditions can be satisfied. So,

$$u(t) = u_p(t) + u_c(t) \tag{2.5}$$

In mathematics, the general solution of the differential equation consists of following;

- *particular solution* $u_p(t)$ and
- a *complementary solution* $u_c(t)$.

When only the free vibrations are considered the natural motion $u_c(t)$ that is $(t) = 0$.
Resultantly the equation of motion for free vibration of a viscous-damped SDOF system becomes

$$\ddot{u} + 2\zeta\omega_n\dot{u} + {\omega_n}^2 u = 0 \tag{2.6}$$

For solving Eq 2.6 is to be assumed that the solution would be of the form

$$u(t) = \bar{C}e^{\bar{s}t} \tag{2.7}$$

Putting Eq. 2.7 into Eq. 2.6, we get

$$(\bar{s}^2 + 2\zeta\omega_n\bar{s} + {\omega_n}^2)\bar{C}e^{\bar{s}t} = 0 \tag{2.8}$$

To make Eq. 2.8 valid for all values of *t*, we must set

$$\bar{s}^2 + 2\zeta\omega_n\bar{s} + {\omega_n}^2 = 0 \tag{2.9}$$

Equation 2.9 is called the **characteristic equation.**

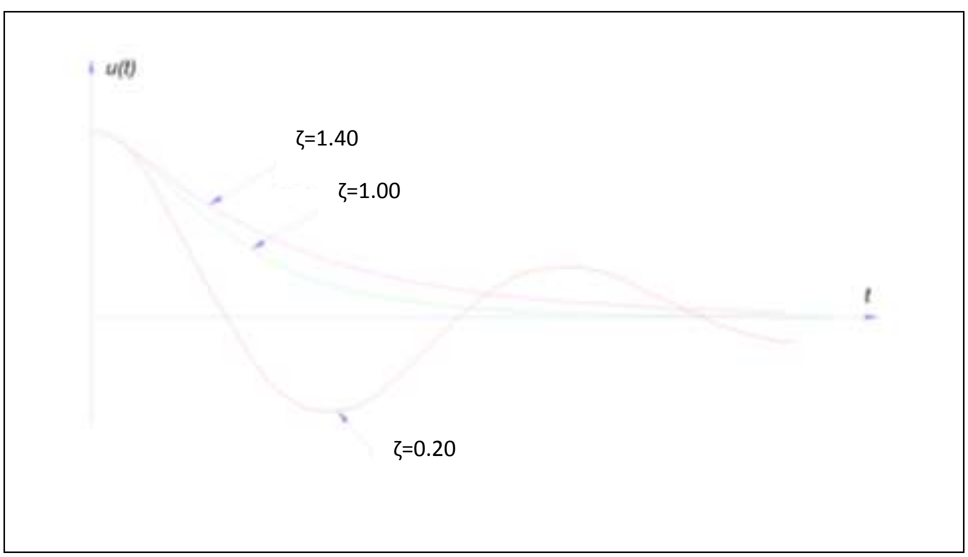


__Figure 2.3__: Response of a viscous-damped SDOF system with various levels of damping.

Roots $\bar{s}_1$ and $\bar{s}_2$ are given by

$$-\zeta\omega_n \pm \omega_n\sqrt{\zeta^2 - 1} = \begin{cases}\bar{s}_1\\ \bar{s}_2\end{cases} \tag{2.10}$$

Damping factor ζ has a certain magnitude. This magnitude can be utilized to distinguish between three cases:

- underdamped ($0<\zeta<1$),
- critically damped ($\zeta = 1$), and
- Over damped ($\zeta > 1$).

Above figure explains all three cases. The oscillatory movement for the underdamped case with a decaying amplitude. No oscillations found in over damped case, and the oscillations wither away slowly. Negligible oscillations for the critically damped system and, dilution of amplitude is more abrupt than the other two cases.+

Case 1: Underdamped SDOF System (ζ<1)

The underdamped case is the most important of the three cases. For ζ<1 Eq.2.10 must be written in the form

$$-\zeta\omega_n \pm i\omega_d = \begin{cases}\bar{s}_1\\ \bar{s}_2\end{cases} \tag{2.11}$$

Where $\omega_d$is the damped circular natural frequency, given by

$$\omega_d = \omega_n\sqrt{\zeta^2 - 1} \qquad \text{(rad/s)} \tag{2.12a}$$

the damped period T_d, corresponding to this is given by

$$T_d = \frac{2\pi}{\omega_d} \qquad \text{(sec)} \tag{2.12b}$$

Taking help from Euler's formula, the general solution $u(t)$ takes the form

$$u(t) = e^{-\zeta\omega_n t}(A_1 \cos\omega_d t + A_2 \sin\omega_d t) \tag{2.13}$$

where $A_1$and A_2 are constants which can be found out from the initially designated conditions. For free vibration initial conditions are u_0 and v_0. Thus the Eq. 2.13 will be given by

$$u(t) = e^{-\zeta\omega_n t}(u_0 \cos\omega_d t + \frac{v_0+\zeta\omega_n u_0}{\omega_d} \sin\omega_d t) \quad (2.14)$$

Since $u_0 = 0$, we see that

$$u(t) = e^{-\zeta\omega_n t}(\frac{v_0}{\omega_d} \sin\omega_d t) \quad (2.15)$$

The value of ζ does have an effect on the frequency ω_d, the most prominent effect of the damping is on motion as to how it dies out, i.e., on the $e^{-\zeta\omega_n t}$ term.

Case 2: Critically Damped SDOF System (ζ=1)

When □=1 Eq. 2.9 gives only one solution,

$$\bar{s} = -\omega_n \quad (2.16)$$

So it turns into

$$u(t) = (C_1 + C_2 t)e^{-\omega_n t} \quad (2.17)$$

Considering the initial conditions, critically damped SDOF's behaviour is given as

$$u(t) = e^{-\omega_n t}[u_0 + (v_0 + \omega_n u_0)t] \quad (2.18)$$

Examples of the same type of non-oscillatory response can be seen in Figure 2.2 and Figure 2.3

Case 3: Overdamped SDOF System (ζ>1)

When ζ>1, Eq. 2.10 describes roots which are negative. Let

$$\omega^* = \omega_n\sqrt{\zeta^2 - 1} \quad (2.19)$$

Then the response of an over damped system can be written in the form

$$u(t) = e^{-\zeta\omega_n t}(C_1 \cosh\omega^* t + C_2 h \sin\omega^* t) \quad (2.20)$$

Where C_1 and C_2 alter with the initial conditions. In the end, with initial condition u_0 and v_0, Eq. 2.20 will be

$$u(t) = e^{-\zeta\omega_n t}\left(u_0 \cosh\omega^* t + \frac{v_0+\zeta\omega_n u_0}{\omega_d}\sinh\omega^* t\right) \qquad (2.21)$$

Figure 2.4 depicts the influence damping level has on the response, which highlights for the smaller damping levels initial overshoot is greater, as it is approaching ζ=1,the final die down comes out to be more rapid.

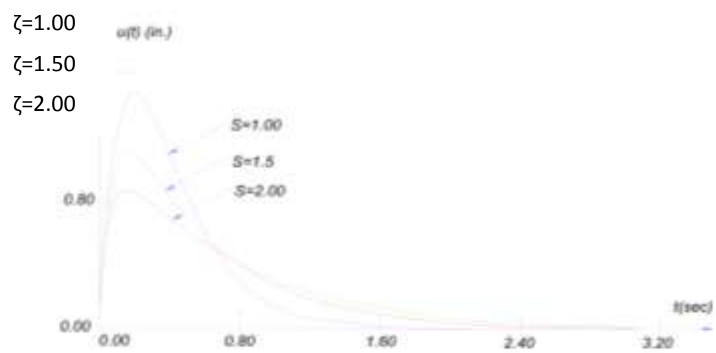


Figure 2.4 :*Response of overdamped systems*

2.2 PREVIOUS RESEARCH

Various literature was consulted to develop an understanding of already conducted research and determine an understanding of the shortcomings. Tianyi Yi at GIT studied an Unreinforced Masonry (URM) structure in 2004. The URM Building had flexible diaphragms, was 22 ft high and had planar dimensions of 24 x 24 ft. Constant experimentation and numerical simulation revealed that damage resulted because of:

(a) Masonry walls developing large discrete cracks and

(b) Sliding and rocking was experienced in URM piers

Previous research results also verified these findings. Additional phenomenon was also seen, including overturning moment effects, flange effects, and different effective piers being formed in a perforated wall. The response of the URM building tested was greatly affected by this global behaviour.

A series of analytical studies - at the material, component and structural level respectively - were conducted to determine the reason behind nonlinear behaviour of the URM building. Firstly, in order to explain why URM assemblages failed when subjected to stress biaxially, a mechanical key model was proposed. Secondly, to study the URM pier's mixed failure modes and a nonlinear relationship of force-deformation, an effective pier model was developed. Thirdly, employing the mechanical models made in previous two levels, a nonlinear pushover model was developed to explain the URM building's nonlinear properties. A three-dimensional finite element model and a nonlinear pushover model and were used to ponder upon the test structure. The results coincided with the test data. Yi also suggested improvements on how to evaluate already existing masonry structures in comparison to existing state in this study. [2]

S.J. Hwang et al. in 2006 proposed an integrated experiment and analysis research program to address the behaviour of reinforced concrete buildings subjected to EQ (earthquake) loading and the subsequent interactions resulting from the nonlinear response of individual components of the structures that further complex the multi-directional effect of the motion of the ground. Due consideration was placed on using simulation response histories to show actuation forces applied to the RC building structures under action of reversed cyclic loading. Analytical simulation studies of RC Buildings were conducted using OpenSees including nonlinear elements recently calibrated at the University of Houston. Analytical tools and the new design methodologies were correlated by the results. Novel wireless telemetry for data collection and distributed data interrogation was used. [3].

H.Chang and J.Xia used finite element method (2009) to determine the difficulties associated with presence of multiple coexistent frictional contacts, and reconfirmed results through laboratory test. They carried out the simulation of a steel frame joined with bolts of high strength splicing and a spliced friction component. This setup resulted in a large number of frictional contacts:

1. The web splicing to the beam web

2. Flange splicing to the beam flange

3. The bolt shank to the bolt hole and

4. The bolt head to the splicing

Post analysis, Chang and Xia found that the finite element simulation predicted results in coherence with the experimental results. Thus it was concluded that finite element method can be applied where complex coexistent contact is present. [4]

A. Penna et al. performed a study based on the outcomes of a project conducted at the European Centre for Training and Research in Earthquake Engineering on unreinforced stone masonry. The seismic response exhibited by shaking table tests simulated on 2 x large-scale building prototypes as per an existing equivalent frame modelling approach involving nonlinear macro elements was conducted. Both buildings differed in the planar stiffness of timber flooring and roof diaphragms as different means were employed to strengthen them. This research addressed many issues involved when seismic response of masonry construction is numerically modelled mainly its effect upon assessing the global response of the discretization and geometry of the equivalent frame model and also on definition of model parameters based on tests of material properties and lateral response that the structural members exhibited. The results of pushover analysis of the calibrated models showed a fair approximation for both damage pattern and envelope even with flexible diaphragms. Results obtained from a time history analysis assessing cumulative damage also suggested good simulation especially with regards to hysteretic response .[5]

In 2016 B. M. Ricles and Dong performed an experiment to assess the seismic response of a three-story, 0.6-scale seismic resistant building structure. It comprised of “a frame with nonlinear viscous dampers and associated bracing (called the DBF), and moment resisting frame (MRF) with reduced beam sections (RBS).” These experiments mainly considered the maximum considered earthquake (MCE) and design basis earthquake (DBE). MRF designs for 100%, 75%, and 60%, determined by ASCE 7-10, for the necessary base shear design strength were studied. The DBF and nonlinear viscous dampers were designed to limit the lateral drift demands. Real-time hybrid method of simulation was employed to replicate MCE and DBE ground motions. As a result, the damage that happens in an MRF when seismically loaded and the drift demand that occurs was seen. Thus we see that even for structures without dampers but with 60% base shear design strength as necessitated by ASCE 7-10, a good level of seismic performance can be achieved under DBE and MCE ground motions,.[6]

What we have seen so far that diluting or dissipating capacity of shear walls made of reinforced concrete, presently in use as resisting mechanisms for laterally transferred loads is not upto mark and instead use of brace system is much preferred as they provide satisfactory solutions. Hadad and Ibrahim in 2017 researched the brace types on loads on frames which were transferred laterally. Analysis was also done for comparison frames which braced and which in filled as well. The study involves four type of frames;

a. Frames which were bare

b. Study with two frames out of which one was reinforced

c. Out of two the second one had steel bracing

d. Solid cement bricks were used to fill in the fourth frame

Cyclic loading was applied on all types of frames. The results achieved were as follows; Frames which had any kind of bracing or which were filled in increased the lateral strength of the bare frame. The difference in the type of infill and the type of bracing has an increasing effect over bare frame's strength that too lateral one. Up to failure the energy that is dissipated for the frames which are braced and filled in is always greater than those which are only braced. As numerical simulation was also carried out, a favourable comparison was achieved between analytical and experimental results. [7]

Cyclic loading tests were conducted on shear walls which were made of reinforced concrete and which were laminated as well, by J. Li et al in 2017. To evaluate their seismic performance, mode of failure, deformation analysis, hysteresis curve, degradation of stiffness, and energy dilution capacities were also used. In addition to all this, 2 x types of construction for achieving reference for construction of shear walls which were laminated as well as reinforced were also established. ABAQUS was used to carry out the numerical simulation of the specimen which was in accord with the experimental results. [8]

Very serious amount of dedication had been applied because of difficulty involved in testing for experiments and numerical simulations which were very unorthodox to calculate the ultimate limit of components of structural members using analytical solutions in use pertaining to complex load- structure iteration and behaviours of materials. As we are aware that loss of column which is sudden is a dynamic process, and quasi-static loads were used primarily for calculation of results on specimens which were either full scale or scaled down. Thus a research was conducted by I. Marginean and Dinu F. in 2018, to study steel frame's response after a column has been lost by them. Numerical models which were very advanced by use of test results which were experimental and factors were studied which caused dynamic increase. Many full-scale structures have been studied for a loss of column in a sudden condition.[9]

CHAPTER 3

RESEARCH METHODOLOGY

Realizing the objectives defined in chapter 1, Research methodology will be comprised of following tasks and subtasks:

<u>Task 1: Conduct Literature Review</u>

- Give emphasis to study comparison of analytical and experimental research work on response of frame structures to lateral loadings.
- Discuss the conclusions drawn from previous works.

<u>Taks 2: Shake Table Test</u>

- Develop a simple model of a steel frame for shake table test.
- Develop an arrangement of weight to apply lateral loading.
- Conduct Time History Analysis to study the free vibration response.

<u>Taks 3: Develop Numerical Model</u>

- Develop a simple model of a reinforced concrete frame with a lumped mass at the top of each story.
- Using ETABS Software for numerical modeling of the selected building.
- Conduct Time History Analysis to compare the free vibration response results with experimental results.

<u>Task 4: Interpret the Results</u>

- Perform comparison of the analytical results with experimental results for free-vibration motion.
- Investigate the differences in analytical and experimental results.

CHAPTER 4

MODEL DEVELOPMENT

4.1 FRAME DESCRIPTION:

The structure under consideration is a steel structure, having 3-stories, 3-bays in longitudinal direction and 3-bays in transverse directions. It is a small scale frame structure. Lateral load resisting system for this structure is Moment Resisting Frame. The height of each story is 11 inches. Total height of the building is 33 inches. 3D Structural Analysis of this Frame was performed on ETABS 2016. Beams and Columns sizes were No.2 steel reinforcement bars in all stories. These bars have 40,000 psi yield strength. Slab is a steel sheet, having 22 gauge thicknesses. 3D view of this building has been shown in figure 4.1. Beam framing Plan and Front Elevation of this building are as per figures 4.2 and 4.3, respectively.

Figure 4.1: *3D view of the Steel frame used for Experimental work*

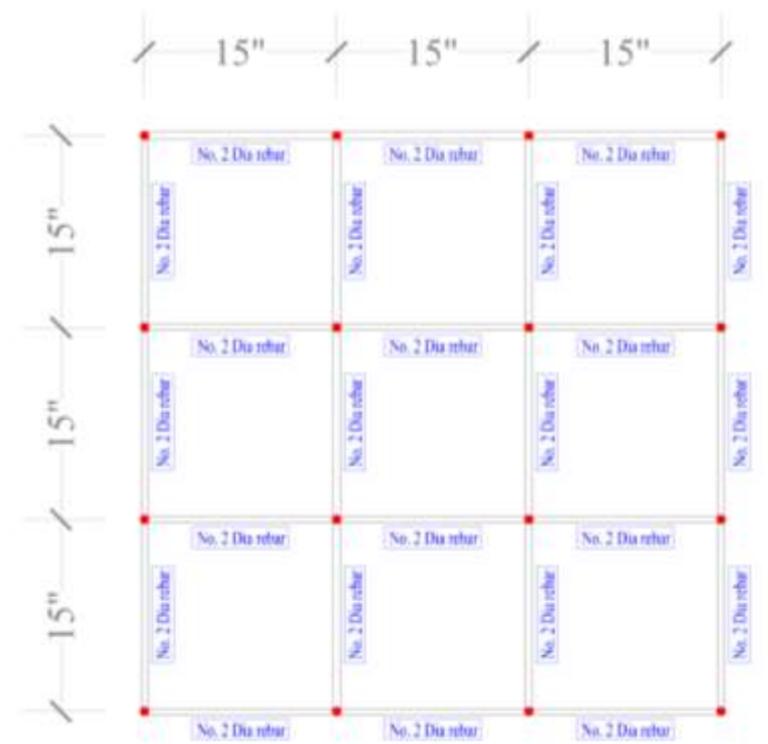


Figure 4.2 : *Typical beam framing plan of the steel frame structure*

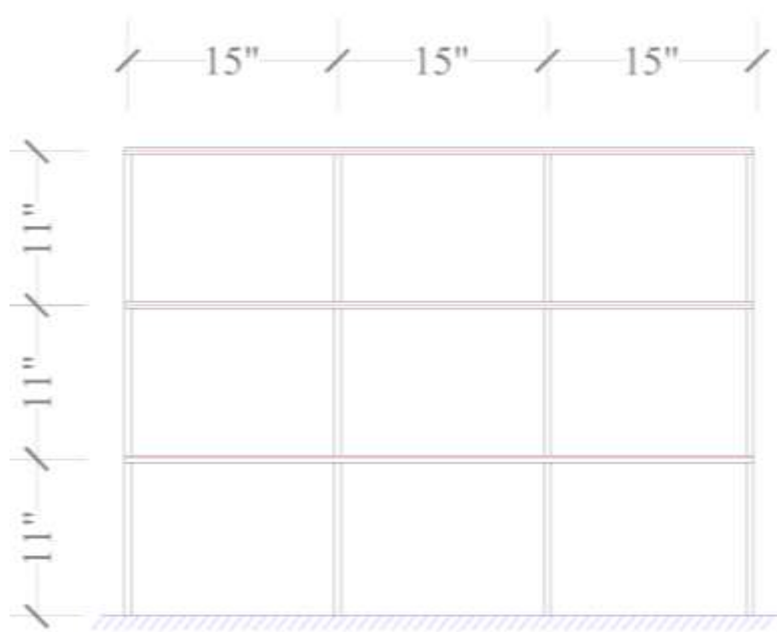


Figure 4.3 : *Front elevation of the steel frame structure*

4.2 GRAVITY LOADING

Only Self wieght of the structure as Dead Load was considered during the exeriment and modelling in ETABS.

4.3 LATERAL LOADING

4.3.1 Lateral Loading in Experimental Work

During Experimental work, due to unavailability of shake table at lab, this frame was pushed to a target displacement of 0.23” without instantaneous force. For this reason, an arrangement of pulley and weights, as shown in figure 4.6, had been adopted. This load was increased incrementally. Addition of loading was stopped when the top displacement was reached to 0.15”. This frame was then released for free-vibration. Free-vibration response of the structure was recorded at each storey level through digital accelerometer.

4.3.2 Lateral Loading in Analytical Work

During Analytical work, Instantaneous force has been provided to reach the target displacement of 0.23 inches. Then the time history record for the response of structure under free-vibration has been recorded at each storey level. These results have been compared with experimental results.

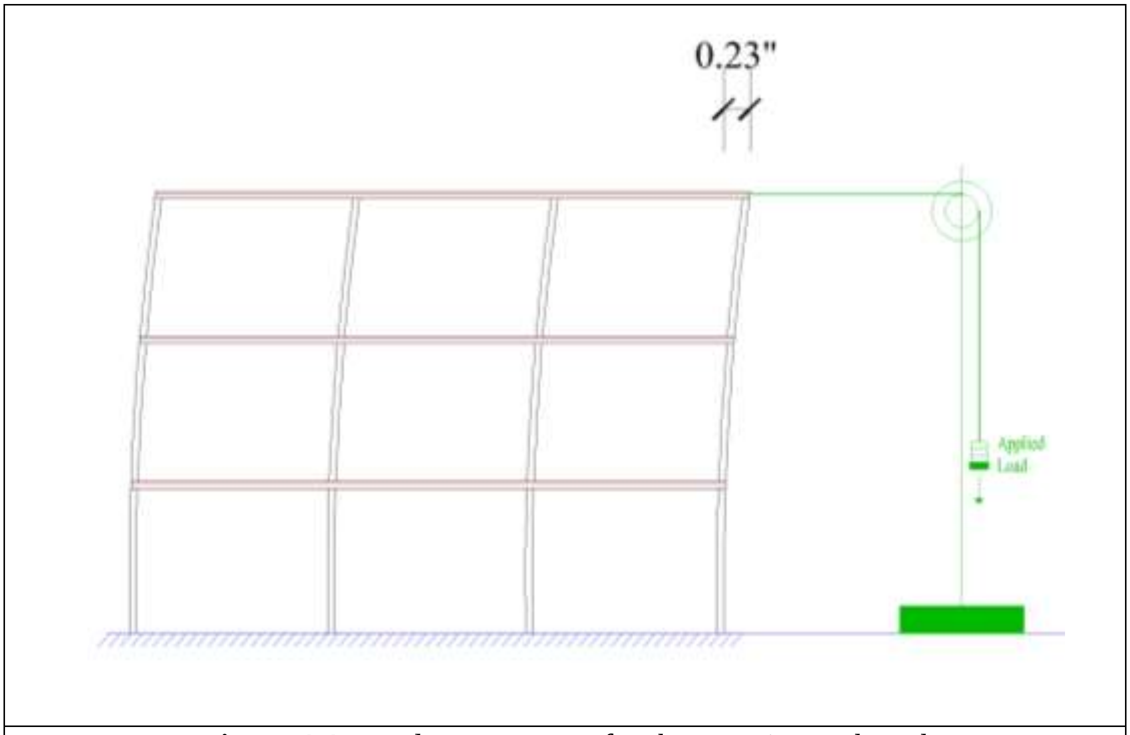


***Figure 4.4**: Load arrangement for the experimental work*

4.4 MODELLING DESCRIPTION:

Analytical model for this frame has been defined in ETABS 2016. ETABS 2016 software is based on Finite Element Method. Columns and Beams were defined as line element. Slabs were defined as shell elements.

4.4.1 Material Definition:

Steel with 40,000 psi yield strength and 60,000 psi ultimate strength has been defined for the modelling of all the structural members of the frame such as Beams and Columns. A stress-strain curve for this material has been provided in the Figure 4.5.

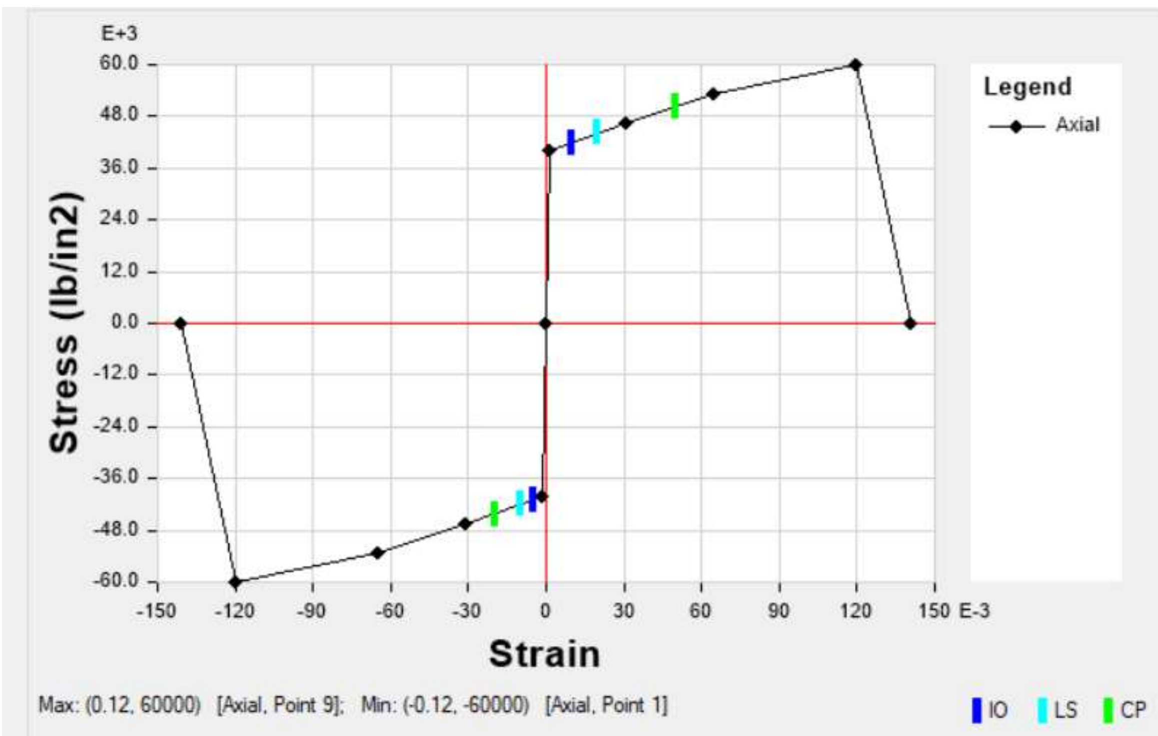


Figure 4.5: *Stress-strain curve defined in ETABS Model for structural elements material*

For Slabs material property, Grade 36 steel as per ASTM has been defined in ETABS. Stress-strain curve for this material has been shown in figure above.

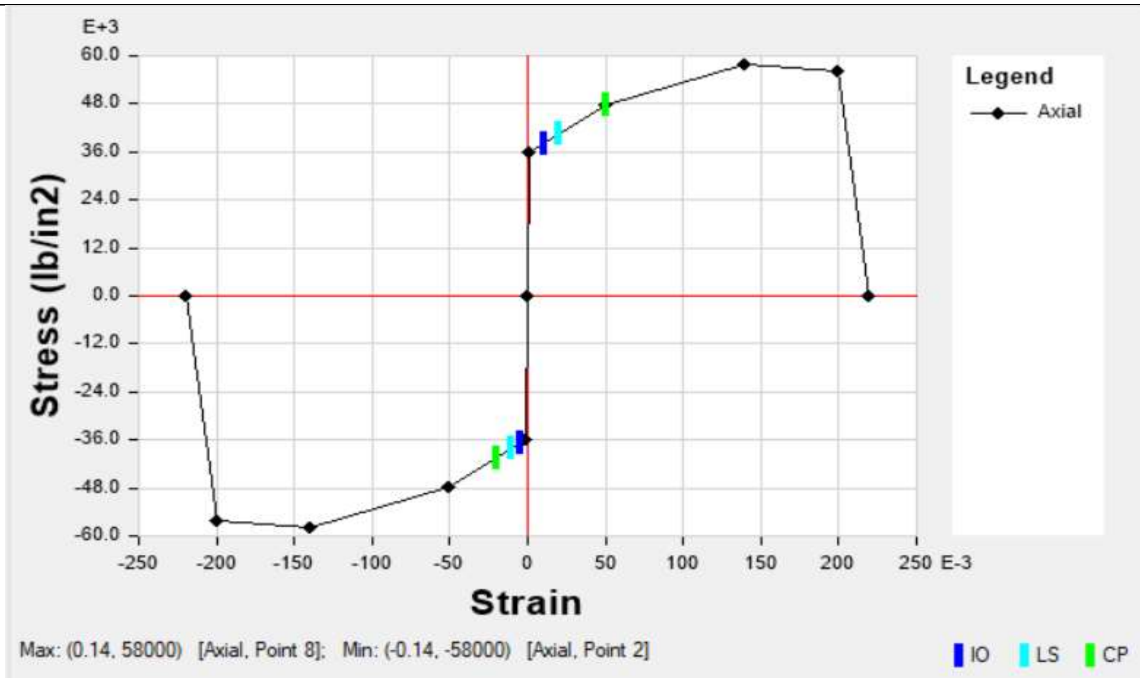


***Figure 4.6**: Stress-strain curve defined in ETABS Model for slab element material*

4.4.2 Columns Definition:

Columns have been defined as line elements. These columns are circular in shape having diameter of 2/8”. These columns have steel properties with yield strength of 40,000 psi. Definition of these columns has been provided in Figure 4.7.

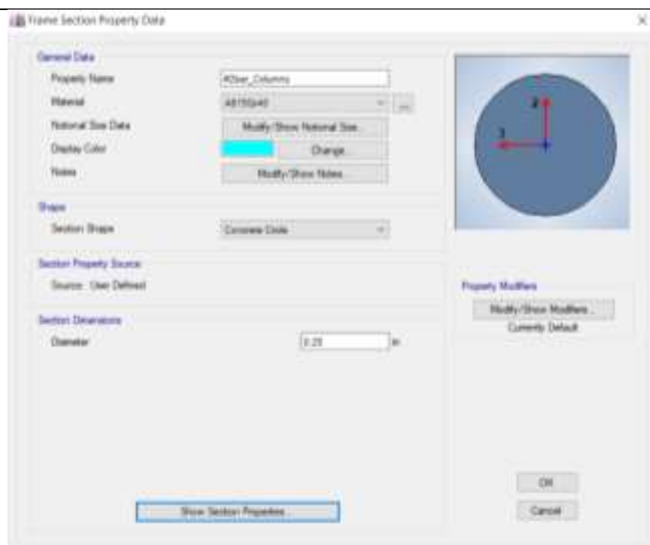


***Figure 4.7**: Frame Section Property Data for columns defined in ETABS Model*

4.4.3 Beams Definition:

Beams have been defined as line elements. These columns are circular in shape having diameter of 2/8". These columns have steel properties with yield strength of 40,000 psi. Definition of these columns has been provided in Figure 4.8.

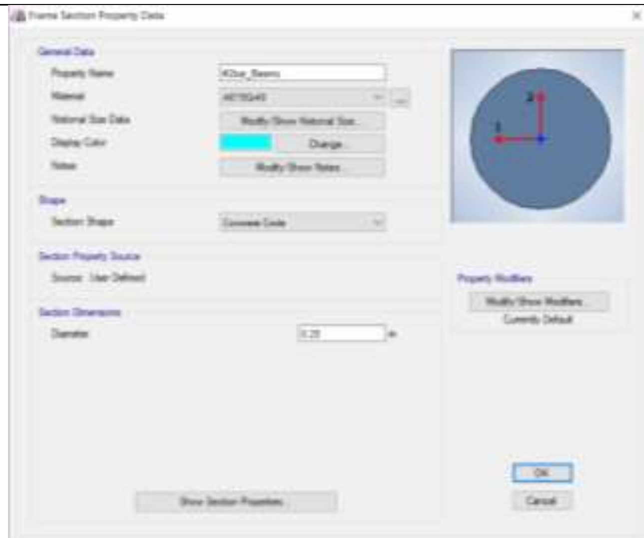


Figure 4.8*: Frame Section Property Data for columns defined in ETABS Model*

4.4.4 Slabs Definition:

Slabs have been defined as shell elements. These slabs have thickness of 0.8 mm (0.029 inches). These slabs have steel properties with yield strength of 36,000 psi. Definition of these slabs has been provided in Figure 4.9.

Figure 4.9*: Slab Section Property Data defined in ETABS Model*

4.5 MODELLING ASSUMPTIONS

Following assumptions were made while modelling the frame structure in ETABS:

1. Beam Column joints were modelled as rigid joints as these are welded connections in actual frame.
2. All supports were assumed to be fixed support.
3. For calculation of mass, dead load was lumped at each storey level in ETABS model as shown in figure 4.10.
4. Damping ratio for steel material was considered as 0.02.

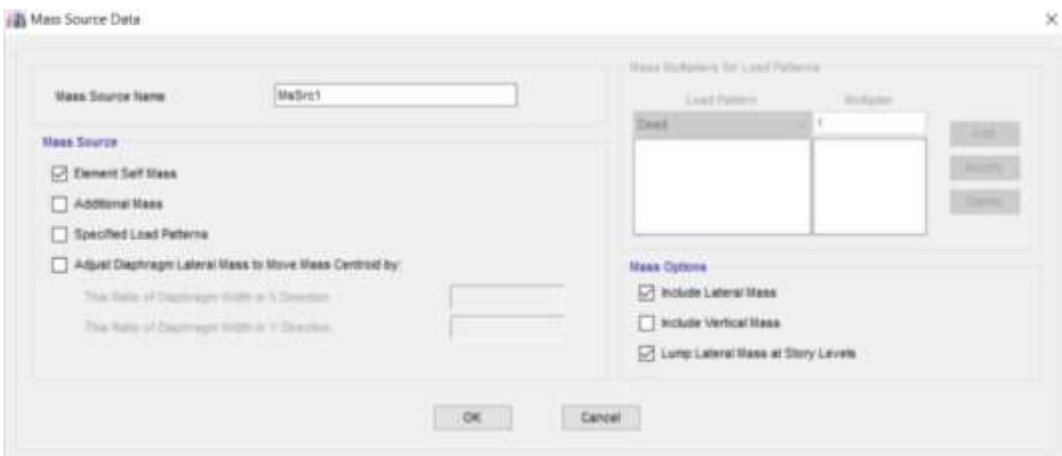


***Figure 4.10**: Frame Section Property Data for columns defined in ETABS Model*

CHAPTER 5

ANALYSIS AND RESULTS

5.1 PUSHOVER ANALYSIS

Steel frame, described in chapter 4, was pushed to immediate occupancy (IO) limit as per FEMA 356. This limit is 0.7 % for Steel moment resisting frame. [10] Thus IO would be 0.231 inches. Life safety (LS) limit for this frame is 2.5% transient i.e. 0.825 inches. Collapse prevention (CP) limit for this frame is 5% transient i.e. 1.65 inches. Pushover Curve has been shown in figure 5.1. This curve shows the IO, LS and CP limits and corresponding forces. The Blue line shows the linear range which was achieved during experimental as well as analytical work. The red line which is also dashed line is interpolated nonlinear range for this frame. Interpolation was performed based on FEMA 356. [10] After reaching the IO limit the structure was released and time history response was recorded.

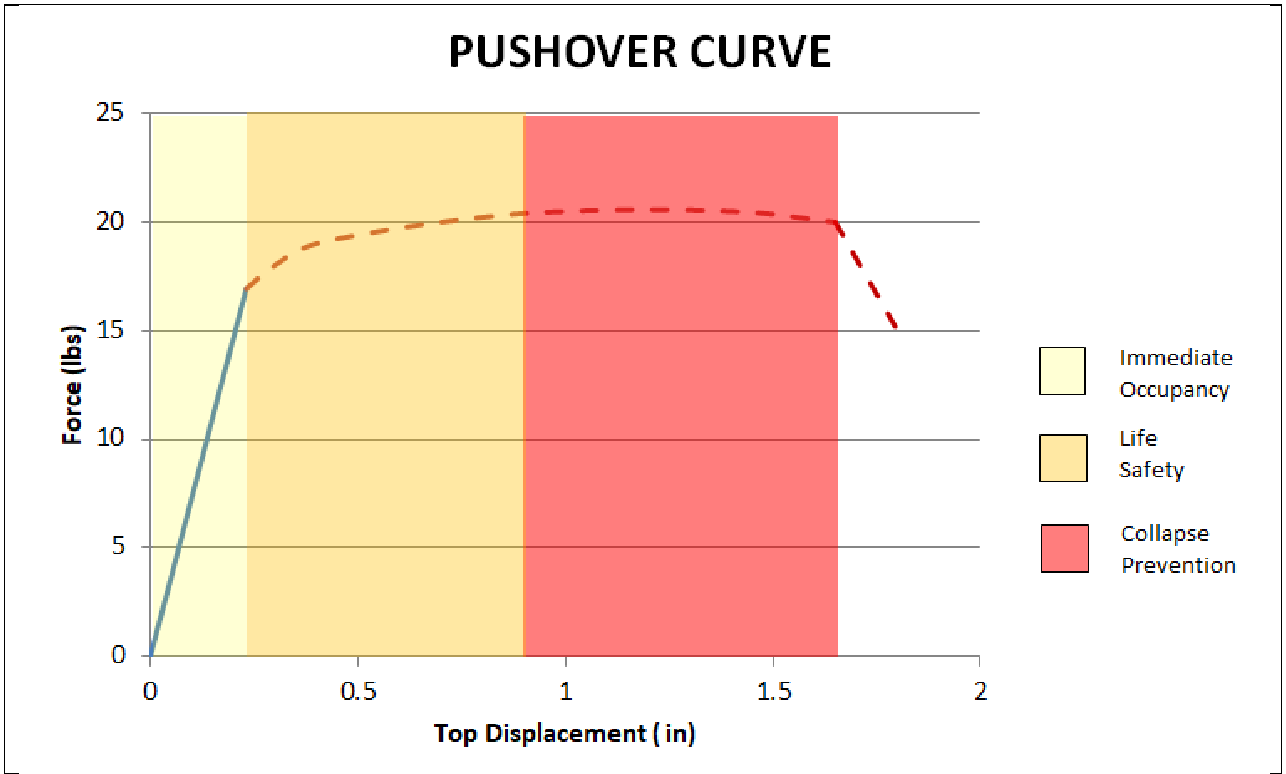


Figure 5.1 : *Pushover Curve for the Steel frame*

5.2 EXPERIMENTAL RESEARCH

Experiment has been performed at Structural Lab of National University of Sciences and Technology (NUST), Risalpur campus. The equipment arrangement has been given in chapter 4. Pushover Analysis was performed to reach the IO limit, and then the structure was released to free-vibration. Acceleration at each storey level was recorded using digital accelerometer. Acceleration at top floor level, second floor level and first floor level was calculated. Figure 5.2 shows the experiment performed to push the structure to target displacement limit i.e. Immediate Occupancy.

Figure 5.2 : *Experiment performed to push the frame to reach the target displacement*

5.2.1 Response Curves

Response of the structure was observed at each storey level, after pushing the structure to immediate occupancy level. As in the figure 5.3, the free-vibration response of the top level of the structure was observed for 20 seconds. Top floor level was displaced to 0.23 inches. Very low amplitude of free vibration has been observed after the 20 seconds which can be ignored while recording the free-vibration response. Response curve for displacement at top level shows that structure is under-damped system.

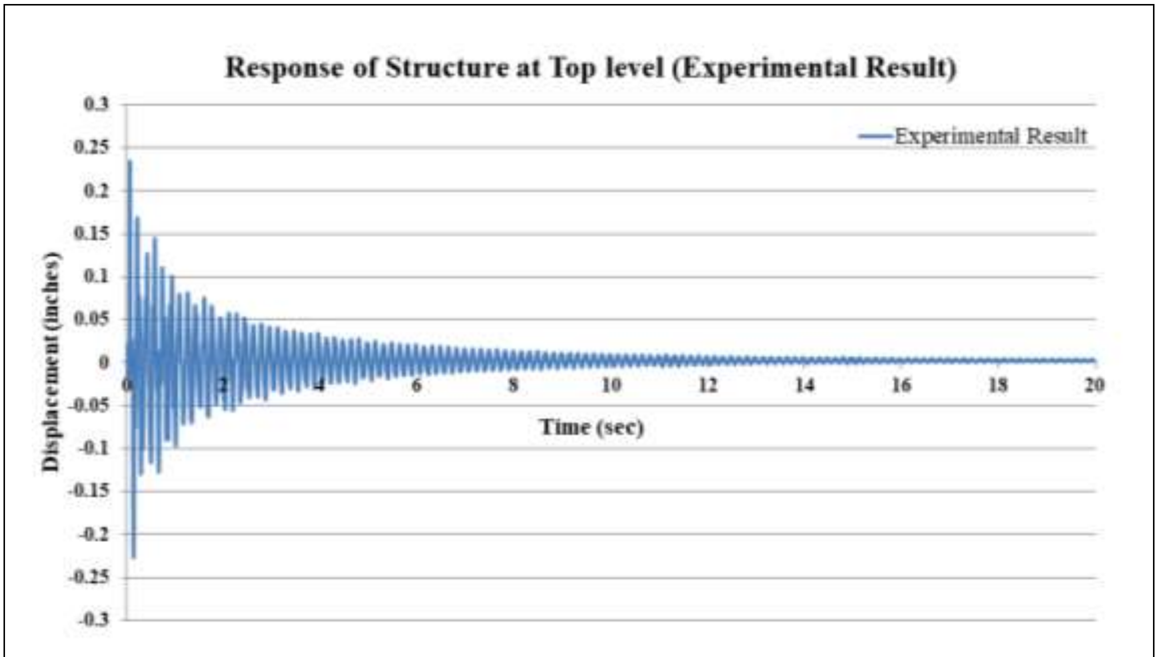


Figure 5.3 : *Displacement curve at top level during experiment work*

As in the figure 5.4, the free-vibration response of the 2nd level of the structure was observed for 20 seconds. 2nd floor level was displaced to 0.18 inches. Very low amplitude of free vibration has been observed after the 20 seconds which can be ignored while recording the free-vibration response. Response curve for displacement at 2nd level shows that structure is under-damped system.

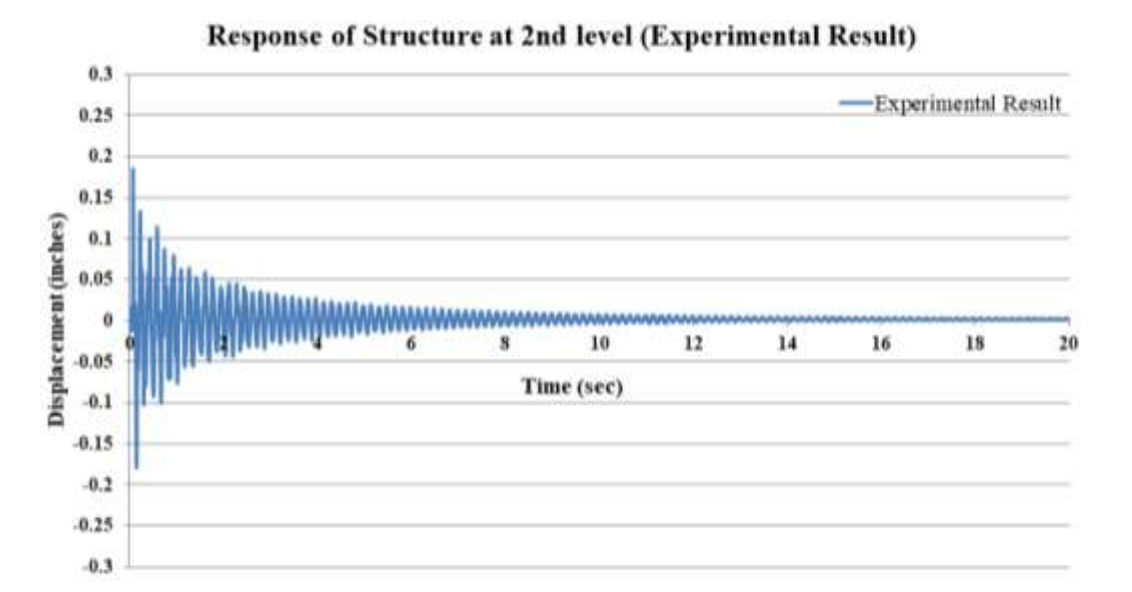


Figure 5.4 : *Displacement curve at 2nd level during experiment work*

As in figure 5.5, the free-vibration response of 1st level of the structure was observed for 20 seconds. 1st floor level was displaced to 0.07 inches. Very low amplitude of free vibration has been observed after the 20 seconds which can be ignored while recording the free-vibration response. Response curve for displacement at 1st level shows that structure is under-damped system.

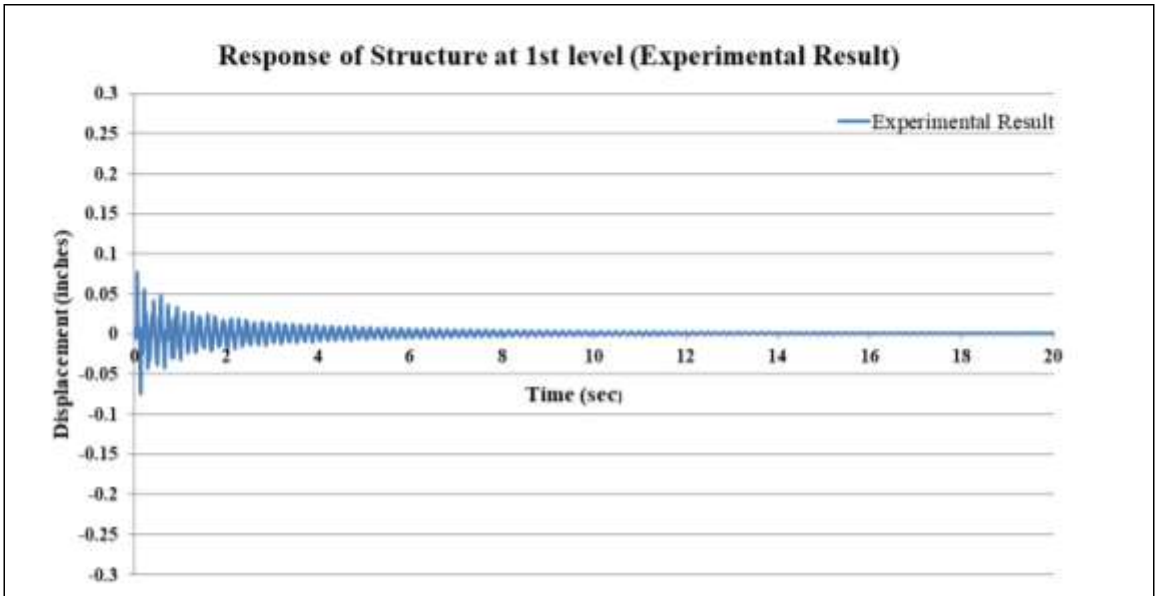


Figure 5.5 : *Displacement curve at 3rd level during experiment work*

5.3 ANALYTICAL RESEARCH

Analytical research has been performed through structural modelling of the actual frame on ETABS 2016. The structure was pushed through a force of 16.94 lbs to reach the IO limit, and then the structure was released to free-vibration. It was observed that same force has been required during experimental and analytical research to reach the IO level. Acceleration at each storey level was recorded using digital accelerometer. Accelaration at top floor level, second floor level and first floor level was observed in ETABS.

5.3.1 Response Curves

As shown by the figure 5.6 that the response for free-vibration of the top level of the structure during analytical research was observed for 20 seconds. Top floor level was displaced to 0.23 inches. Very low amplitude of free vibration has been observed after the 20 seconds which can be ignored while recording the free-vibration response. Response curve for displacement at top level for analytical result also shows that structure is under-damped system. Damping ratio was considered as 0.02 in ETABS.

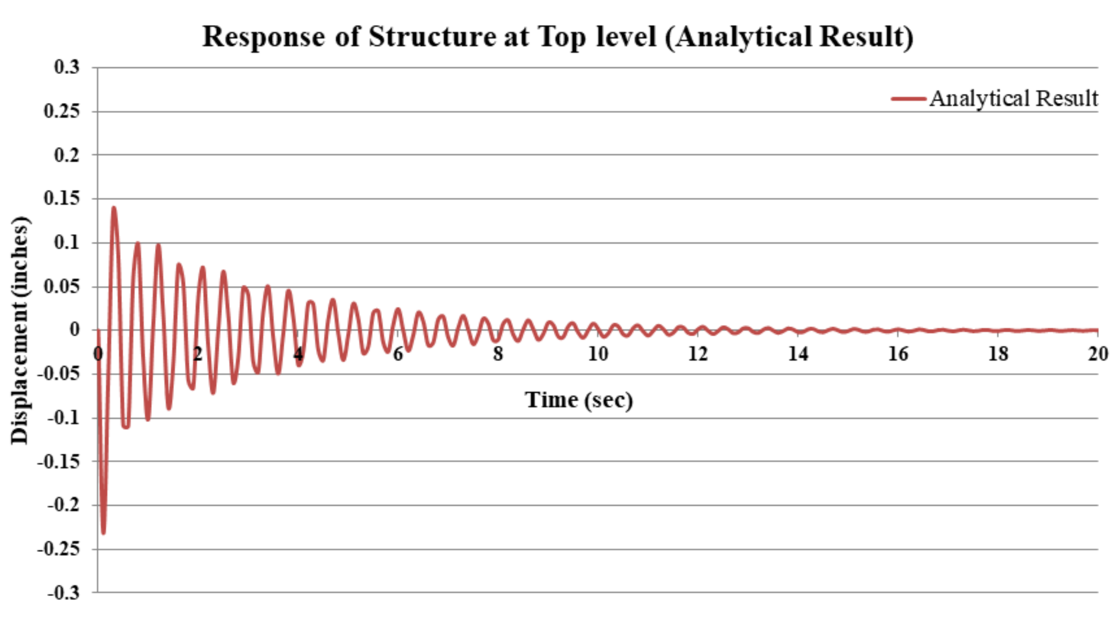


Figure 5.6 : *Displacement curve at top level during analytical work*

It can be seen from the figure 5.7 that the free-vibration response of the 2nd level of the structure during analytical research was observed for 20 seconds. 2nd floor level was displaced to 0.18 inches. Very low amplitude of free vibration has been observed after the 20 seconds which can be ignored while recording the free-vibration response. Response curve for displacement at top level for analytical result also shows that structure is under-damped system. Damping ratio was considered as 0.02 in ETABS.

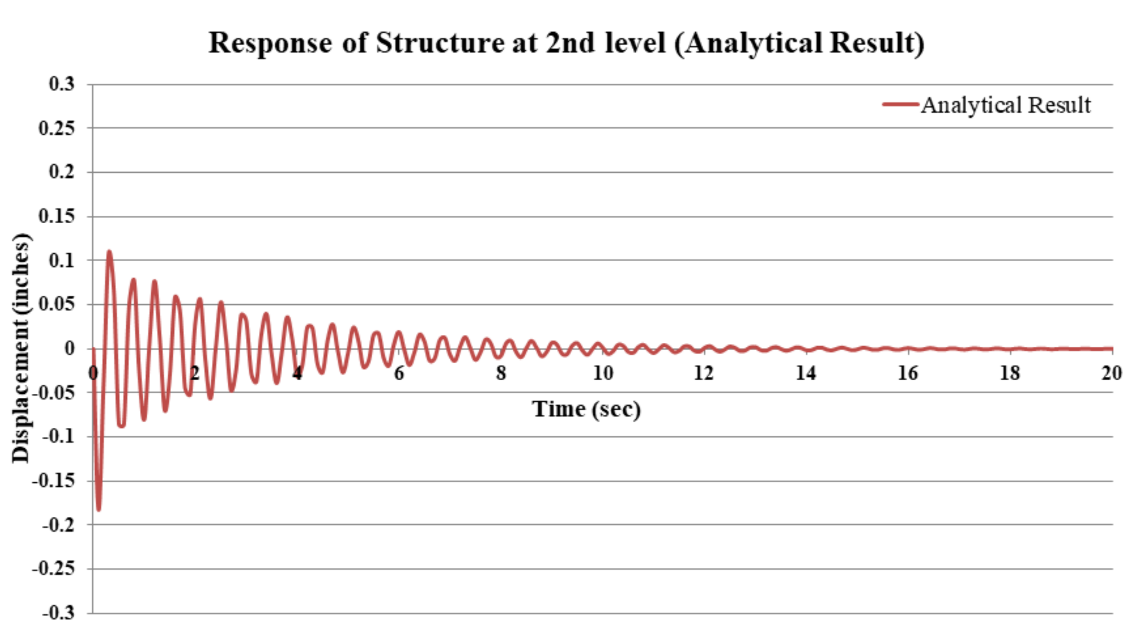


***Figure 5.7** : Displacement curve at 2nd level during analytical work*

It can be seen from the figure 5.8 that the free-vibration response of the 1st level of the structure during analytical research was observed for 20 seconds. 1st floor level was displaced to 0.077 inches. Very low amplitude of free vibration has been observed after the 20 seconds which can be ignored while recording the free-vibration response. Response curve for displacement at top level for analytical result also shows that structure is under-damped system. Damping ratio was considered as 0.02 in ETABS.

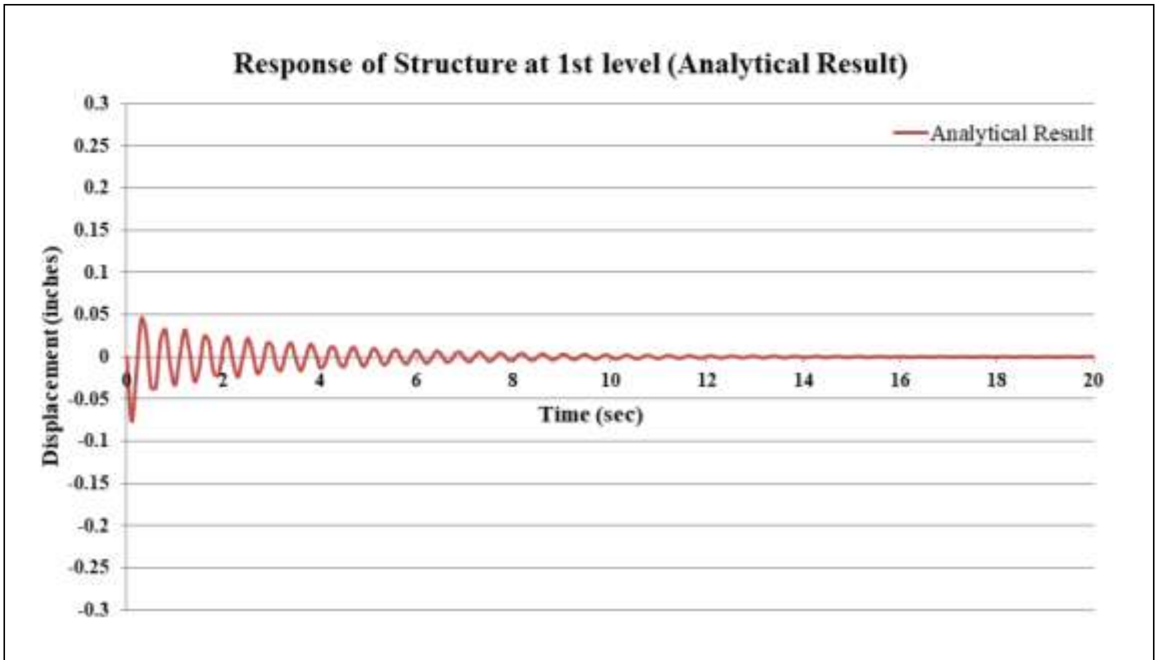


Figure 5.8 : *Displacement curve at top level during analytical work*

5.4 DISCUSSION ON RESULTS

It was assumed in the start of the research that damping ratio would be 0.02 during analytical research. While comparing the damping pattern of displacement at top, 2nd and 1st levels during experimental and analytical research, it can be seen that the assumption of 0.02 of damping for steel material is correct. Decaying pattern of the displacement at each floor level is same in experimental and analytical research. Time period varies in experimental and analytical results. Structure was pushed to mode 1 shape, and then it was released to free-vibration. Figure 5.8, 5.9 and 5.10 shows displacement at each floor in mode 1 shape is same in experimental research and analytical research. However, amplitude of each oscillation during free-vibration is different in experimental and analytical results. Analytical result overestimates the displacement during free-vibration response. Thus designing of the structural elements using ETABS software will provide conservative design of the structure. The results give the detailed comparison and insight of vibration response of the structure. The agreement in the results will increase confidence level for use of FEM software.

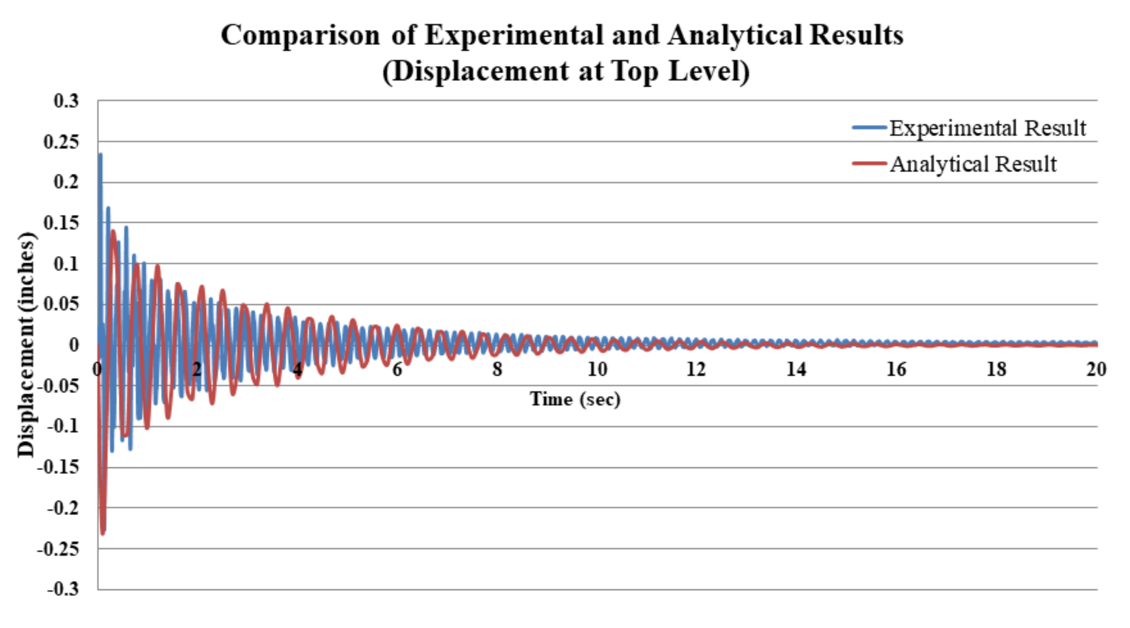


Figure 5.9 : *Comparison of displacement curves at top level during experimental and analytical work*

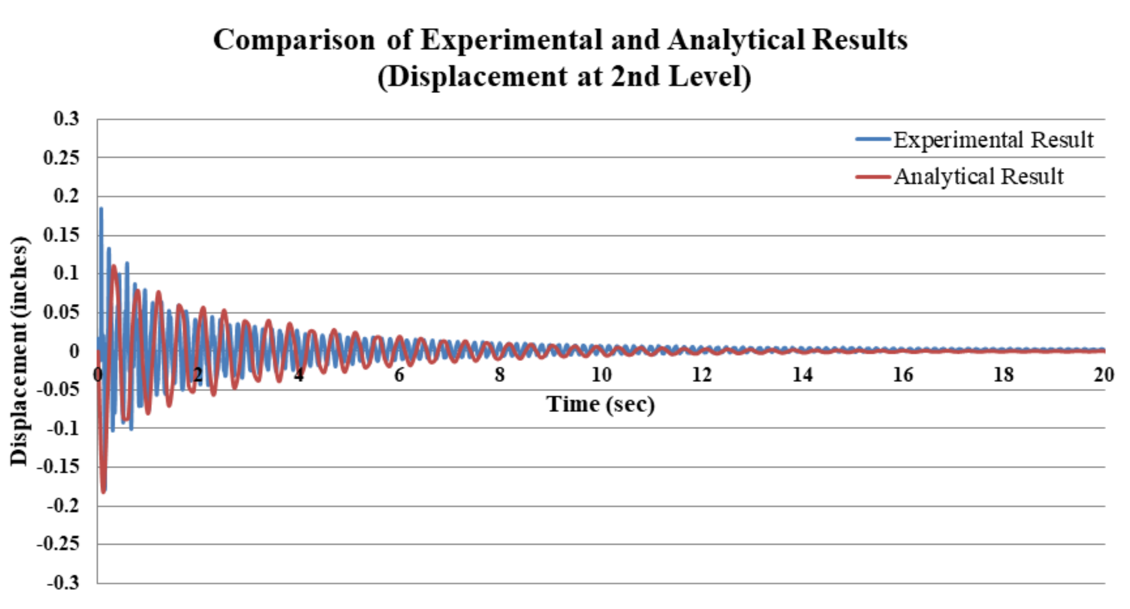


Figure 5.10 : *Comparison of displacement curves at 2nd level during experimental and analytical work*

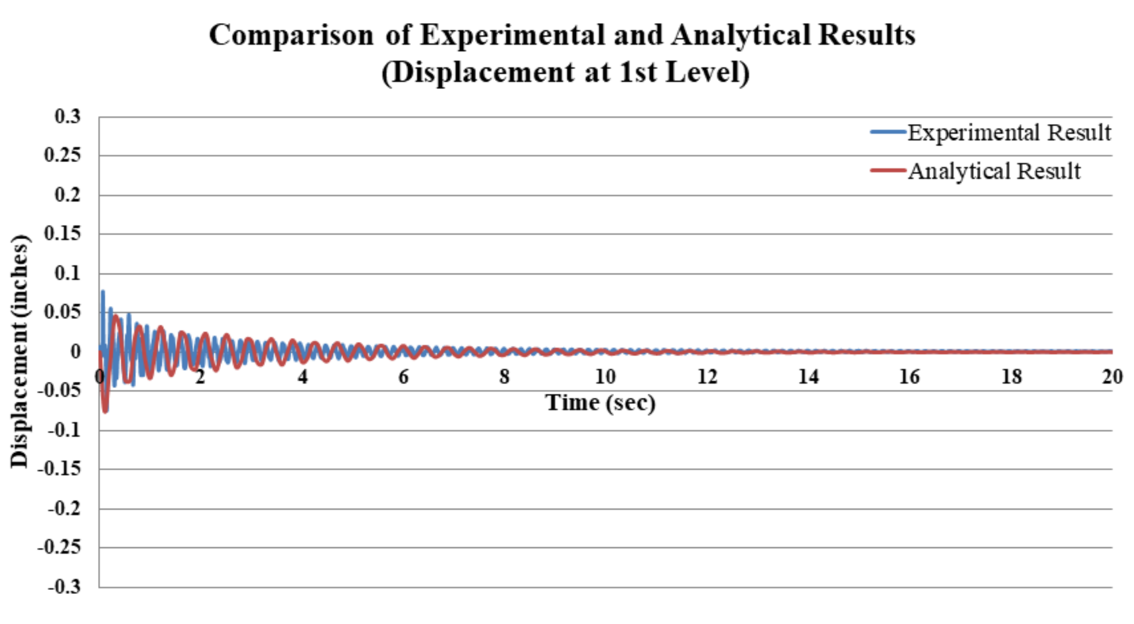


Figure 5.11 : *Comparison of displacement curves at 1st level during experimental and analytical work*

CHAPTER 6

CONCULSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

This study compared the experimental research and analytical research on free-vibration response of a steel moment resisting frame. It was concluded that:

1. It was assumed that damping ratio for steel moment resisting frame would be 0.02. This damping ratio predicts the actual behaviour of the material.

2. Storey Accelerations at each level was same in experimental and analytical research after pushing the top storey to immediate occupancy level of the frame. Thus, mode shape for first mode is same during experimental and analytical results.

4. Time period of the steel frame may vary while comparing the free-vibration response in experimental results with analytical result.

5. Analytical research provides conservative results in terms of amplitude of each oscillation during free-vibration response.

6. ETABS 2016 predicts the response of structure which is in agreement with response during experimental research.

6.2 RECOMMENDATIONS

Following Recommendations can be drawn from this study:

1. A similar study is recommended to be carried out for RC moment resisting frame. Such study is now possible in MCE with addition of structural Engineer Lab