PASSIVE VIBRATION CONTROL OF FRAMED STRUCTURES BY BASE ISOLATION METHOD USING LEAD RUBBER BEARING

Kisan Jena

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA
ORISSA – 769 008

PASSIVE VIBRATION CONTROL OF FRAMED STRUCTURES BY BASE ISOLATION METHOD USING LEAD RUBBER BEARING

A Project Report

Submitted by

Kisan Jena
Roll No: 209CE2041

In partial fulfilment of the requirements for
the award of Degree of

MASTER OF TECHNOLOGY

in

STRUCTURAL ENGINEERING

Under the supervision of
Dr. Shishir Kumar Sahu, Professor
Dr. Pradip Sarkar, Associate Professor

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA
ORISSA – 769 008

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY,
ROURKELA, ORISSA, INDIA – 769 008,
mail id – www.nitrkl.ac.in

CERTIFICATE

This is to certify that the thesis entitled **"Passive vibration control of framed structures by base isolation method using lead rubber bearing"**, submitted by Mr. Kisan Jena, in partial fulfilment of the requirements for the award of Master of Technology Degree in Civil Engineering with Specialization in Structural Engineering at the National Institute of Technology Rourkela are an authentic work carried out by him under my supervision.

The candidate has fulfilled all the prescribed requirements.

The thesis, which is based on candidate's own work, has not been submitted elsewhere for a Degree/Diploma.

In my opinion, the thesis is of standard required for the award of a Master of Technology Degree in Civil Engineering.

To the best of my knowledge, he bears a good moral character and decent behaviour.

Prof. S.K. Sahu
Professor
Department of Civil Engineering
National Institute of Technology
Rourkela-769 008 (INDIA)
Email: sksahu@nitrkl.ac.in

Prof. P. Sarkar
Associate Professor
Department of Civil Engineering
National Institute of Technology
Rourkela-769 008 (INDIA)
Email: sarkarp@nitrkl.ac.in

ACKNOWLEDGEMENT

The satisfaction and euphoria on the successful completion of any task would be incomplete without the mention of the people who made it possible whose constant guidance and encouragement crowned my effort with success.

I am grateful to the Dept. of Civil Engineering, NIT Rourkela, for giving me the opportunity to execute this project, which is an integral part of the curriculum in M.Tech programme at the National Institute of Technology, Rourkela. I would like to take this opportunity to extend my deepest gratitude to my teacher and supervisor, Prof. S.K Sahu, Prof. P Sarkar for their continuous encouragement and active guidance. I am indebted to them for their valuable time they have spared for me during this work. They are always there to meet and talk about my ideas, to proofread and mark up my research papers and chapters, and to ask me good questions to help me think through my problems.

I am also very thankful to all the faculty members of the department, especially Structural Engineering specialization for their constant encouragement, invaluable advice, Encouragement, inspiration and blessings during the project. Also, I am thankful to all the non-teaching staffs of Civil Engineering Department for their kind cooperation. I am very much thankful to Prof. N. Roy, Head, Department of Civil Engineering, for his continuous encouragement. Also, I am indebted to him who provided me all official and laboratory facilities. During the course of this work, I am supported in part by the Earthquake Engineering Literature Survey Workshop for Post Graduate Students from Engineering Colleges across India held at IIT Kanpur during July 10-10, 2010. I am really thankful to them.

Last but not the least, I take this opportunity to express my regards and obligation to my parents, friends and family members for educating me in all aspects. I can never forget for their unconditional support and encouragement to pursue my interests.

Kisan Jena

ABSTRACT

In recent years considerable attention has been paid to research and development of structural control devices with particular emphasis on mitigation of wind and seismic response of buildings. Many vibration-control measures like passive, active, semi-active and hybrid vibration control methods have been developed. Passive vibration control keeps the building to remain essentially elastic during large earthquakes and has fundamental frequency lower than both its fixed base frequency and the dominant frequencies of ground motion. Base isolation is a passive vibration control system.

Free vibration and forced vibration analysis was carried out on the framed structure by the use of computer program SAP 2000 v12.0.0 and validating the same experimentally. The results of the free vibration analysis like time period, frequency, mode shape and modal mass participating ratios of the framed structure were found out. From modal analysis the first mode time period of fixed base building is found to be 0.56 sec whereas the first mode period of isolated building is found to be 3.11s (approximately 6 times the fixed-base period!). This value is away from the dominant spectral period range of design earthquake. Forced vibration analysis (non-linear time history analysis) was done to determine the response of framed structures and to find out the vibration control efficiency of framed structures using lead rubber bearing. Isolation bearings in this study are modelled by a bilinear model. Under favourable conditions, the isolation system reduces the interstorey drift in the superstructure by a factor of at least two and sometimes by a factor of at least five. Acceleration responses are also reduced in the structure by an amount of 55-75% although the amount of reduction depends upon the force deflection characteristic of the isolators. A better performance of the isolated structure with respect to the fixed base structure is also observed in floor displacements, base shear (75-85% reduction), floor acceleration relative to the ground(less acceleration imparted on each floor and their magnitude is approximately same in each floor), roof displacement. Introduction of horizontal flexibility at the base helps in proper energy dissipation at the base level thus reducing the seismic demand of the super structure to be considered during design.

***Keywords**: Passive vibration control, Time history analysis, interstorey drift, yielded stiffness, Design basis earthquake.*

TABLE OF CONTENTS

PAPERS PUBLISHED IN INTERNATIONAL CONFERENCE:

Passive vibration of framed structure by base isolation technique using rubber bearing, Advance in materials and techniques for infrastructure development (AMTID) 2011, S077, NIT Calicut.

PAPERS PUBLISHED IN NATIONAL CONFERENCE:

Vibration measurement and Passive vibration control of framed structure using SAP 2000, Advance in Civil Engineering, STHAPNA 2011, VSSUT, Burla.

WORKSHOP ATTENDED:

Attended Earthquake Engineering Literature Survey workshop for post-graduate students from Engineering Colleges across India and at IIT Kanpur by NICEE during July 10- 19, 2010 at IIT, Kanpur.

List of Figures

List of Tables

CHAPTER
1

INTRODUCTION

1.1 Background

For seismic design of building structures, the traditional method, *i.e.*, strengthening the stiffness, strength, and ductility of the structures, has been in common use for a long time. Therefore, the dimensions of structural members and the consumption of material are expected to be increased, which leads to higher cost of the buildings as well as larger seismic responses due to larger stiffness of the structures. Thus, the efficiency of the traditional method is constrained. To overcome these disadvantages associated with the traditional method, many vibration-control measures, called structural control, have been studied and remarkable advances in this respect have been made over recent years. Structural Control is a diverse field of study. Structural Control is the one of the areas of current research aims to reduce structural vibrations during loading such as earthquakes and strong winds.

In terms of different vibration absorption methods, structural control can be classified into active control, passive control, hybrid control, semi-active control and so on (Appendix-VI). The passive control is more studied and applied to the existing buildings than the others. Base isolation is a passive vibration control system that does not require any external power source for its operation and utilizes the motion of the structure to develop the control forces. Performance of base isolated buildings in different parts of the world during earthquakes in the recent past established that the base isolation technology is a viable alternative to conventional earthquake-resistant design of medium-rise buildings. The application of this technology may keep the building to remain essentially elastic and thus ensure safety during large earthquakes. Since a base-isolated structure has fundamental frequency lower than both its fixed base frequency and the dominant frequencies of ground motion, the first mode of vibration of isolated structure involves deformation only in the isolation system whereas superstructure remains almost rigid. In this way, the isolation becomes an attractive approach where protection of expensive sensitive equipments and internal non-structural components is needed. It was of interest to check the difference between the responses of a fixed-base building frame and the isolated-base building frame under seismic loading. This was the primary motivation of the present study.

1.2 Importance of present study:

Civil Engineers are still unable to rigorously predict even in a probabilistic way the loads which structures may have to withstand during their useful life. All structures are subjected to vibration. Recent destructive earthquakes in California and Japan have shown how vulnerable our structures and societies remain to natural phenomena. The enormous losses inflicted by such catastrophes have motivated ever more stringent requirements on the performance of structural systems, in an effort to reduce the cost of repair and disruption. The cost and performance requirements for both buildings and equipment have motivated advances in the field of Structural Control, which deals with methodologies for the protection of high performance structural systems. The vibration isolator is a device that is designed to effectively isolate such structures from harmful vibrations.

1.3 Vibration Control

- Vibration control is the mechanism to mitigate vibrations by reducing the mechanical interaction between the vibration source and the structure, equipment etc. to be protected.
- Structural control relies on stiffness (i.e. energy storage) and damping (i.e. energy absorption/dissipation) devices in a structure to control its response to undesirable excitations caused by winds and moderate earthquakes. This control has, in most cases, been achieved passively by means of bracing systems and shear walls, which do not require any additional external energy input. More recently, we have seen the emergence of more modem passive structural control systems. The tuned mass damper and base isolation systems are examples of such relatively modern passive systems.

1.4 Base isolation of structures

1.4.1 Concept of base isolation

Seismic base isolation of structures such as multi-storey buildings (Appendix-V), nuclear reactors, bridges, and liquid storage tanks are designed to preserve structural integrity and to prevent injury to the occupants and damage to the contents by reducing the earthquake-induced forces and deformations in the super-structure. This is a type of passive vibration control. The performance of these systems depends on two main characteristics:

(1) The capacity of shifting the system fundamental frequency to a lower value, which is well remote from the frequency band of most common earthquake ground motions.

(2) The energy dissipation of the isolator.

1.4.2 Types of Bearings:

Following types of bearings are available as per literature as per their materials:

a) Flexible Columns.

b) Rocking Balls.

c) Springs.

d) Rubbers.

e) Other materials than rubber.

Rubbers are further divided into four categories,

- a) Rubber Bearing
- b) Steel laminated rubber bearing (RB).
- c) Lead rubber Bearing (LRB).
- d) High damping rubber bearing (HDRB).

1.5 Response of the building under Earthquake.

1.5.1 Building frequency and period:

The magnitude of Building response mainly accelerations depends primarily upon the frequencies of input ground motions and Buildings natural frequency. When these are equal or nearly equal to one another, the buildings response reaches a peak level. In some cases, this dynamic amplification level can increase the building acceleration to a value two times or more that of ground acceleration at the base of the building. Generally buildings with higher natural frequency and a short natural period tend to suffer higher accelerations and smaller displacement. Buildings with lower natural frequency and a long natural period tend to suffer lower accelerations and larger displacement. When the frequency content of the ground motion is around the building's natural frequency, it is said that the building and the ground motion are in resonance with one another. Resonance tend to increase or amplify the building response by which buildings suffer the greatest damage from ground motion at a frequency close to its own natural frequency.

1.5.2 Building stiffness:

Taller the building, longer the natural period and he building is more flexible than shorter building.

1.5.3 Ductility:

Ductility is the ability to undergo distortion or deformation without complete breakage or failure. In order to be earthquake resistant the building will possess enough ductility to withstand the size and type of earthquake it is likely to experience during its lifetime.

1.5.4 Damping:

All buildings possess some intrinsic damping. Damping is due to internal friction and adsorption of energy by buildings structural and non- structural components. Earthquake resistant design and construction employ added damping devices like shock absorbers to supplement artificially the intrinsic damping of a building.

1.6 Methodology

a) A thorough literature review to understand the seismic evaluation of building structure, application of time-history analysis and Modal analysis.

b) Select an aluminium frame for the analytical and experimental case study.

c) The selected aluminium frame was modelled in computer software SAP2000.

d) Modal analysis of a Benchmark Problem was carried out to validate the accuracy of steps involved in SAP 2000 v12.0.0 software package and then modal analysis of the selected frame was carried out to obtain the dynamic properties of the frame.

e) Modal analysis result of the selected frame is to be validated experimentally with the help of FFT Analyser and shake table test.

f) Time-history analysis of a fixed base structure and similar base isolated structure was carried out.

1.7 ORGANISATION OF THESIS

This introductory chapter presents the background; objectives and methodology of the project. The first part of **Chapter 2** discusses details about time history analysis procedures and different improvements of this procedure available in literature. The second part of this chapter presents effectiveness of base isolation as present in previous researches. **Chapter 3** presents theoretical formulations of base isolation, modal analysis and time history analysis for framed structure. **Chapter 4** presents the details of the results obtained in experimental programmes, analytical programme and finally the results obtained. **Chapter 5** presents the conclusions obtained from the above study. It also presents the future scope of the work which can be extended further. **Chapter 6** presents the references considered during the work. Finally, in **Chapter 7** appendices have been given for assessment of certain information.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction: - Thus the modal analysis of framed Structure is of great technical importance for understanding the behaviour of the framed Structure under applied dynamic loading. The study of response analysis methodology (Experimental or Analytical) of a base isolated framed structure with a fixed base otherwise similar framed structure is essential to conclude the effectiveness of base isolation using rubber bearing.

"Earthquake proof structures" generally mean the structures which resist the earthquake and save and maintain their functions. The key points for their design includes select good ground for the site, make them light, make them strong, make them ductile, shift the natural period of the structures from the predominant period of earthquake motion, heighten the damping capacity.

Izumi Masanory [1] studied on the remained literature, the first base isolated structure was proposed by Kawai in 1981 after the Nobi Earthquake (M=8.0) on journal of Architecture and building Science. His structure has rollers at its foundation mat of logs put on several steps by lengthwise and crosswise manually. After the San Francisco Earthquake (M=7.8) an English doctor J.A. Calantarients patented a construction by putting a talc between the foundations in 1909. The first base isolated systems actually constructed in the world are the Fudo Bank Buildings in Himeji and Simonoseki, Japan designed by R. Oka. After the world War-II, the U.S took a leading part of Earthquake Engineering. **Garevski** A *et al.* [2] The primary school "Pestalozzi" in Skopje, built in 1969, is the first building in the world for which natural rubber isolators were used for its protection against strong earthquakes. The first base isolated building in the United States is the Foothill Communities of Law And Justice Centre completed in 1985 having four stories high with a full basement and sub-basement for isolation system which consists of 98 isolators of multilayered natural rubber bearings reinforced with steel plates. The Superstructure of the building has a Structural Steel frame stiffened by braced frames in some Bays [3]. In India, base isolation technique was first demonstrated after the 1993 Killari (Maharashtra) Earthquake [EERI, 1999]. Two single storey buildings (one school building and another shopping complex building) in newly relocated Killari town were built with rubber base isolators resting on hard ground. Both were brick masonry buildings with concrete roof. After the 2001 Bhuj (Gujarat) earthquake, the four-storey Bhuj Hospital building was built with base isolation technique [4]. The Base isolation system has been introduced in some books of dynamic Engineering and the number of scholars has been increasing in the world.

2.2 General categories of literature review: The studies presented for literature review are categorized as:

(i) Non linear dynamic Analysis of framed structure.

(ii) Relative performance of Fixed-base and Base-isolated concrete frames.

(iii) Base Isolated Structures subjected to near-fault earthquakes

(iv) Effect of Superstructure Stiffening on base isolation.

(v) Seismic response of torsionally coupled Base Isolated Structures

2.2.1 Non linear dynamic Analysis of framed structure

Constantinou *et al.* [5] described in this paper an analytical model and an algorithm to analyze multiple buildings on a common isolation system and the results are used to demonstrate the importance of analyzing the combined system as against analyzing individual buildings. Jain and Thakkar [6] explored the idea of superstructure stiffening is to enhance the effectiveness of base isolation for 10 to 20 storeys range of buildings. The superstructure stiffening may result in reduced fixed base period and such buildings, if base isolated may develop smaller seismic response. Jangid and Kulkarni [7] made a comparison in this study of the seismic response of a multi-storey base-isolated building by idealizing the superstructure as rigid and flexible. The top floor acceleration and bearing displacement of the system are plotted for different system parameters and compared with the corresponding response under rigid superstructure conditions to study the influence of superstructure flexibility. Naharajaiah and Sun [8] carried out a study aimed to evaluate the seismic response of base isolated USC Hospital Building and Fire Command Control Building in Los Angles during the 1994 Northridge earthquake. Hang *et al.* [9] presented the earthquake responses of the multifunctional vibration-absorption RC mega frame structures decrease significantly in comparison with the normal megaframe structures, namely 60-80 per cent decrease of the earthquake responses of the major frames and 70-90 per cent decrease of the ones of the minor frames.

Mazza and Vulcano [10] compared different base-isolation techniques, in order to evaluate their effects on the structural response and applicability limits under near-fault earthquakes. Palazzo and Petti [11] described rational methodology to evaluate behaviour factors for base isolated structures (BIS) code. The method used in this study is to directly derive q-factors by the nonlinear response analysis of 2 d.f. isolated models subject to strong seismic excitations artificially generated according to EC8 design spectra. Dutta and Jangid [12] investigated the reliability against the first passage failure of base-isolated and fixed base steel buildings frames due to earthquake ground motion and found base isolated structures are more reliable than fixed base frame. Mei [13] applied Classical theories in modelling in-plane vibrations in planar frame structures. Analytical solution has been obtained using a wave vibration approach. The propagation, reflection, and transmission matrices, as well as the matrix relations between the injected waves and externally applied forces and moments are obtained using classical vibration theories.

Nagarajaiah *et al.* [14] presented an analytical model and a solution algorithm developed for nonlinear dynamic analysis of three-dimensional reinforced-concrete-base-isolated structure with elastomeric and/or sliding isolation systems. Biaxial and uniaxial models for both elastomeric and sliding isolation bearings have been presented. Deb [15] analysed 3D nonlinear analysis procedure of base isolated building. Important issues like effects of soft soil on performance of base isolated building, effects of near fault motion, soil-base isolated building interaction have also been discussed. Kitagawa *et al.* [16] discussed the results of vibration test of the Oiles Technical Centre Building and the results of experiments on the characteristics LRBs of the building. Fundamental vibration characteristics of the base-isolated building were investigated by free vibration and forced vibration testing. Agrawal et al. [62] Controlling of seismic responses in Structures, Systems and Components was performed using active, semi-active, passive dampers and Seismic Base Isolators. Sorace and Terenzi [63] developed a final experimental campaign to access the interference campaign of the dissipative actions of a base isolation system having pressurized fluid viscous spring dampers

are coupled to steel-Teflon sliders,

2.2.2 Relative performance of Fixed-base and Base-isolated concrete frames

Shenton and Lin [17] compared the performance of code designed fixed-base and base-isolated concrete frames in a quantitative manner. Time-history analyses were conducted for three ensembles of recorded earthquakes. Analysis considered the nonlinear behaviour of the isolation system and superstructure. Base-isolated concrete moment frame designed to between 25% and 50% of the code-recommended base shear performed comparably to the fixed-base design, when based on: (1) Extent of superstructure yielding; (2) average relative roof displacement; (3) average first-story drift; and (4) average time of first yielding in the superstructure. Bezerra and Carneiro [18] presented a paper which deals with numerical evaluation of the efficiency of anti-vibration mechanisms applied to typical frame structures under earthquake. The building structure is modelled by finite elements, an anti-vibration mechanism is placed at the building base with special finite element, and an artificial earthquake equivalent to El Centro is generated and applied at the building base. The behaviour of the frame, with and without anti-vibration mechanisms, is compared.

Kang *et al.* [19] performed systematic dynamic response analyses for three different models such as a fixed based, an SREI based and an FREI based low-story building structures in terms of displacement, drift, acceleration and shear force. The SREI and the FREI based structures are proven to be the more effective isolation systems against seismic events by comparing with the fixed based one. Ibrahim [20] described an article which deals with the comprehensive assessment of recent developments of nonlinear isolators in the absence of active control means. Base isolation utilizes friction elements, laminated-rubber bearings, and the friction pendulum. Nonlinear viscoelastic and composite material springs, and smart material elements are described in terms of material mechanical characteristics and the dependence of their transmissibility on temperature and excitation amplitude. Providakis [21] carried out nonlinear time history analyses using a commercial structural analysis software package to study the influence of isolation damping on base and superstructure drift. The efficiency of providing supplemental viscous damping for reducing the isolator displacements while keeping the substructure forces in reasonable ranges is also investigated. Aiken *et al.* [22] documented in their paper the seismic behaviour of four seismically isolated buildings from their recorded response for earthquakes producing various amplitudes and durations of shaking. It considers the responses of multiple buildings to multiple earthquakes, using consistent procedures, evaluates soil-structure interaction effects, and achieves new insights into isolation system behaviour by examining temporal variations in system properties. Hasebe *et al.* [23] has presented design experience of the Base-Isolated new Computer Centre of the Tohoku Electric Power Co. This is located in Sendai City, Miyagi Prefecture. High damping laminated rubber bearings are used as base-isolation device. Through the response analyses, it was confirmed that the responses of the building and the computer equipment were reduced significantly by adopting the base-isolation system.

Hang *et al.* [24] presented the multifunctional vibration-absorption RC megaframe structures, which act as tuned mass dampers, base isolators and damping energy-dissipaters. Significant decrease of the earthquake responses of the multifunctional vibration-absorption RC megaframe structures is shown in comparison with the normal megaframe structures. Chang and Mo [25] described practical system that combines a flexible first story with sliding frictional interfaces. Energy dissipation is provided by the first story ductile columns and by the Teflon sliders which is placed at the top of the first story reinforced concrete framed shearwalls. Jangid [26] investigated stochastic response of buildings isolated by lead–rubber bearings (LRB). The stochastic response of isolated building frames is obtained using the time-dependent equivalent linearization technique as the force–deformation behaviour of the LRB is highly non-linear. Dolce and Telesca [27] presented a parametric study on RC base isolated building structures with different type of nonlinear devices. The seismic response is evaluated by means of non-linear dynamic analysis using artificially generated accelerograms (EC8, type B). Jangid [28] obtained response of a one-storey model of a torsionally coupled (asymmetric) building with sliding support to two component random ground motions. Force-deformation behaviour of the sliding blocks is modelled as elasto-plastic with very high initial stiffness. The response of the system is analyzed under different parametric variations to investigate the effectiveness of the sliding support like eccentricity of superstructure, uncoupled torsional to lateral frequency ratio, mass ratio and coefficient of friction of sliding support. Nagarajaiah and Sun [29] carried out a study with an objective to evaluate seismic performance of base isolated USC hospital building and fire command control building in Los angles during 1994 Northridge Earthquake. Hur *et al.* [30] evaluated isolation story displacement of base isolated residential building according to earthquake records selection method. Non-linear time history analysis is done with variables such as earthquake ground motion records. Rao and Jangid [31] carried out an experimental Shake table study for the response of the structures supported on base isolation systems. It was found that isolation is quite effective in reducing acceleration the response of the System.

Hasebe *et al*. [32] discussed the detailed design approach to the base-isolated building, in which the computer equipment is installed along with the results of the response analyses. The responses of the building and the computer equipment were reduced significantly by adopting the base-isolation system, and that the building and the computer equipment is safe against large earthquake motions. Colunga and Jiménez [33] presented a methodology for the design of base isolated structures located in the Mexican Pacific Coast using dynamic principles. The dynamic method is based on the one proposed by the UBC-97 code, but including several modifications to take into account the philosophy of local codes and regional seismicity. Constantinou [34] presented an analytical model and an algorithm to analyze multiple buildings on a common isolation system. Importance of analyzing the combined system as against analyzing individual buildings is demonstrated. Ferrell and Nagarajaiah [35] presented a linear model is based on small displacements and rotations and predicts stable post-critical behaviour or increasing critical load with increasing horizontal displacement. However, unstable post-critical behaviour is observed in the bearing test results presented in this study. Nagarajaiah and Xiaohong [36] evaluated the seismic performance of the base-isolated FCC building during the 1994 Northridge earthquake and the effect of impact. Nagarajaiah and Xiaohong [37] carried out a study with an objective is to evaluate the seismic performance of the base-isolated USC hospital building during the 1994 Northridge earthquake. A nonlinear analytical model of the USC hospital building is developed. Structural behaviour during the Northridge earthquake is evaluated in detail. The base-isolated USC hospital building performed well and reduced the response when compared to a fixed-base structure. Aiken *et al.* [38] used strong motion recordings from four seismically isolated buildings in time-invariant and time-variant modal identification analyses. Kareem and kijiwski [61] presented an overview of the measures that reduce the structural response of buildings including a summary of research work.

2.2.3 Base Isolated Structures subjected to near-fault earthquakes:

Mazza and Vulcano [39] studied the nonlinear seismic response of base-isolated framed buildings subjected to near-fault earthquakes to analyze the effects of supplemental damping at the level of the isolation system, commonly adopted to avoid overly large isolators. Aiken *et al.* [66] described the results of a study of an existing seismically isolated building in Southern California which are located near San Andreas fault, san Jacinto fault and south frontal fault zone. Analysis result for three levels of earthquake were presented and recommendation s are made.

2.2.4 Benchmark problem:

The U.S. Panel on structural control and monitoring (Currently chaired by Professor Satish Nagarajaiah, Rice University, Houston, TX), IASCM, and the ASCE structural control and monitoring committee [40] have developed a new benchmark study to compare control strategies designed for a base-isolated building subjected to strong near-fault pulse-like ground motions. Phase II part focuses on smart base-isolated building benchmark problem with nonlinear isolation systems—friction or elastomeric system. Gavin *et al.* [41] carried out a benchmark study to provide a well defined base isolated building with a broad set of carefully chosen parameter sets, performance measures and guidelines to the participants. Erkus and Johnson [42] presented a sample control design for the base-isolated benchmark building with bilinear hysteretic bearings (e.g. lead–rubber bearings). A linear quadratic Gaussian (LQG) controller is selected for this purpose. Nagarajaiah *et al.* [43] considered is an eight-storey base-isolated building similar to existing buildings in Los Angeles, California as the benchmark structure with an objective to provide a well-defined base-isolated building with a broad set of carefully chosen parameter sets, performance measures and guidelines to the participants, so that they can evaluate their control algorithms. Agrawal *et al.* [44] proposed to develop experimental benchmark models to truly demonstrate the capability of various structural control systems in protecting the integrity of buildings during earthquakes. These two experimental benchmark models will be made available to the international structural control community for testing various control algorithms and devices.

2.2.5 Effect of Superstructure Stiffening on Base Isolation:

Jain and Thakkar [45] explored the idea of superstructure stiffening is to enhance the effectiveness of base isolation for 10 to 20 storeys range of buildings. The superstructure stiffening may result in reduced fixed base period and such buildings, if base isolated may develop smaller seismic response. Jangid and Kulkarni [46] made a comparison of the seismic response of a multi-storey base-isolated building by idealizing the superstructure as rigid and flexible with the corresponding response under rigid superstructure conditions to study the influence of superstructure flexibility under various isolation system parameters (i.e. isolation period, damping, yield strength of the elastomeric bearings and friction coefficient of sliding systems).

2.2.6 Base Isolation concept to Soft First Story Buildings

Chang and Mo [47] described a practical system that combines a flexible first story with sliding frictional interfaces. Energy dissipation is provided by the first story ductile columns and by the Teflon sliders.

2.2.7 Seismic response of torsionally coupled Base Isolated Structures:

Datta and Jangid [48] presented the stochastic response of a one-storey model of an asymmetric building, isolated by different base isolation systems, to random excitation by directly solving the stochastic differential equation of the combined system. Jangid [49] obtained the response of a one-storey model of a torsionally coupled (asymmetric) building with sliding support to two component random ground motions. The base of the system consists of sliding blocks resting on sliding foundation raft. Colunga and Rojas [50] described the torsional response of base-isolated structures when eccentricities are set in the isolation system. The amplification factors for the maximum isolator displacement of the asymmetric system with respect to the symmetric system increase as the eccentricity increases. Nagarajaiah *et al.* [51] investigated in this paper the torsional coupling in elastomeric base-isolated structures. Stiffness eccentricity in the superstructure, eccentricity in the isolation system is studied in this paper. Asymmetry and dynamic characteristics of the superstructure are as important as that of the isolation system. Colunga and Sobero´n [52] presented the torsional response of base-isolated structures when eccentricities (e_s) between the centre of mass and the centre of rigidity are set in the superstructure is. Maximum isolator displacements and peak displacement ductility demands were studied and compared to the ones obtained for symmetric systems of reference for the different ground motions.

Hong *et al.* investigated [53] in this study the UBC-91 and UBC-97 static lateral load procedures for isolated structures and a new formula is proposed for the vertical distribution of seismic load. The proposed method provides conservative results compared with those from dynamic analysis and UBC-91 approach, and produces a more economic solution compared with the UBC-97 static lateral response procedure. Colunga and Cruz [54] studied the peak responses for different ratios of the static eccentricities (e_s) between the centres of mass and the centres of rigidity in the superstructure due to asymmetries. Nonlinear dynamic analyses were used to study peak responses for different ratios of the static eccentricities (e_s) between the centres of mass and the centres of rigidity in the superstructure due to asymmetries. Kilar and Koren [55] contributed to a better understanding of the behaviour of base isolated asymmetric structures. The symmetrical structural variant and appropriate LRB bearing properties were designed according to Eurocode 2 and 8. The asymmetric variants were produced by shifting the centre of mass CM toward one side of the building. Tena-colunga and Villegas-jiménez [60] presented a methodology for the design of base isolated structures located in the Mexican Pacific Coast using dynamic principles. The dynamic method is based on the one proposed by the UBC-97 code

2.2.8 Seismic Isolation to reduce the ductility demand:

Kelly *et al.* studied [56] the effectiveness of seismic base isolation in controlling the deformations in prefabricated concrete structures. When utilised in prefabricated concrete structures seismic base isolation has the potential to reduce the ductility demand from these structures under seismic loading.

2.2.9 Base-isolated structures during impact with adjacent structures:

Jangid and Matsagar [57] computed the variation of top floor absolute acceleration and bearing displacement for different isolation systems during impact upon the adjacent structures under different earthquakes to study the behaviour of the building during impact and comparative performance of various isolation systems. Variation of system parameters like size of gap, stiffness of impact element, superstructure flexibility and number of story of base-isolated building effecting impact response of isolated building was also studied.

2.2.10 Experimental Study on Elastomeric Isolation Bearings:

Nagarajaiah *et al.* [58] experimentally determined the effect of horizontal displacement or shear strain on critical load and studied the validity of the approximate correction factor. It is shown that the critical load decreases with increasing horizontal displacement or shear strain. It is also shown that substantial critical load capacity exists at a horizontal displacement equal to the width of the bearing. Buckle and Nagarajaiah [59] stated combination of rubber layers and reinforcing steel shims gives a device that is axially very stiff but soft laterally. But increasing the shear flexibility of these short columns can lead to relatively low buckling loads, which may be further reduced when high shear strains are simultaneously imposed. Abrams *et al.* [60] presented the Results of an experimental study that help to illustrate the effectiveness of using base isolators for reducing the lateral-force demand for engineered masonry building structures in areas of high seismicity. Kelly *et al.* [64] proposed a novel isolation system which can be used in certain low rise buildings to isolate the horizontal and vertical ground motions. Aiken *et al.* [65] overviewed the studies at the Earthquake Engineering Research Center of the University of California at Berkeley describing the different types of devices, the results of the shake table experiments, and associated analytical work. Aiken [67] proposed energy dissipation devices by which damage to the built environment can be mitigated. Aiken and Kikuchi [68] proposed an hysteretic model for elastomeric isolation bearing for accurately predicting the seismic response of base isolated structures. Extensive series of experimental tests were carried out to fully identify the mechanical characteristics. Aiken [69] discussed the important characteristics of isolation devices and the influence of these characteristics on testing, in terms of such factors as displacement, force, rate of loading, and test temperature. Aiken *et al.* [70] carried out a study to understand how and why this change in engineering and construction practice has taken place. It is clear that the Kobe earthquake helped to trigger the acceptance of new technologies in the Japanese seismic design community, but the occurrence of severe earthquake shaking is not by itself enough to change the direction of engineering and construction practice.

2.3 Objective and scope of present Investigation:

Following objectives of the present study are arrived at based on the literature review presented in Section 2.2:

a) **To measure the Vibration Parameters (Natural Frequency, Mode Shapes, and corresponding Modal participating mass ratios)** of a selected aluminium frame analytically by carrying out modal analysis method using computer program SAP 2000 v12.0.0, to validate the modal analysis results by experimental study with FFT Analyser and to observe the response of frame in shake table test.

b) **To find out the response (i.e. Displacement, Velocity, and Acceleration) of the Aluminium frame subjected to a selected earthquake ground motion by non-linear time history analysis. (**Northridge earthquake, January 17, 1994, Reseda, a neighbourhood in the city of Los Angeles, California, USA) using SAP 2000 v12.0.0.

c) **To compare the performance of a base isolated framed structure with a fixed base otherwise similar framed structure** to conclude the effectiveness of base isolation using rubber bearing.

CHAPTER

3

MATHEMATICAL FORMULATION

3.1 Modal Analysis:

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

1. Eigenvector analysis
2. Ritzvector analysis

3.2 Eigenvector analysis

Eigenvector analysis determines the undamped free-vibration mode shapes and frequencies of the system. These natural modes provide an excellent insight into the behaviour of the structure. Free vibration of linear MDF systems without damping with p (t) = 0 is given as,

$$m\ddot{u} + ku = 0$$

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

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

Substituting,

$$u(t) = \emptyset_n(A_n \cos w_n t + B_n \sin w_n t)$$

Where, A_n and B_n are constants.

$\emptyset_n$= The deflected shape.

w_n= natural frequency for nth number of mode.

Substituting the value of u (t) in the above equation, we get

$$[-w_n^2 m\emptyset_n + k\emptyset_n]q_n(t) = 0$$

Here either$q_n(t) = 0$, which indicates $u(t)$=0 and there is no motion of the system (called as trivial solution). Other solution is given as,

$$k\emptyset_n = w_n^2 m\emptyset_n$$

$$[k - w_n^2 m]\ \emptyset_n = 0$$

This equation is called as matrix eigenvalue problem.

It has non-trivial solutions if

$$\text{Det}\ [k - w_n^2 m] = 0$$

When the determinant is expanded a polynomial of order N in w_n^2 is obtained. Hence the above equation is called as frequency equation.

The N roots, w_n^2 of the equation determine the N natural frequencies w_n(n = 1, 2, ...N) of vibration. Corresponding to the N natural vibration frequencies w_n of an N-DOF system, there are N independent vectors $\emptyset_n$ which are known as natural modes of vibration, or natural mode shapes of vibration.

Let the natural mode $\emptyset_n$ corresponding to the natural frequency w_n have elements $\emptyset_{jn}$ where j indices the DOFs. The N eigenvectors can be displayed in a single square matrix, each column of which is a natural mode:

$$\emptyset = [\emptyset_{jn}] = \begin{bmatrix} \emptyset_{11} & \emptyset_{12} & \cdots & \emptyset_{1N} \\ \emptyset_{21} & \emptyset_{22} & \cdots & \emptyset_{2N} \\ \vdots & \vdots & \ddots & \vdots \\ \emptyset_{N1} & \emptyset_{N2} & \cdots & \emptyset_{NN} \end{bmatrix}$$

The N eigenvalues can be assembled into a diagonal matrix Ω^2, which is known as Spectral matrix of the eigenvalue problem,

$$\Omega^2 = \begin{bmatrix} w_1^2 & 0 & 0 & 0 \\ 0 & w_1^2 & 0 & 0 \\ 0 & 0 & \ddots & 0 \\ 0 & 0 & 0 & w_1^2 \end{bmatrix}$$

3.3 FFT Analyzer.

Apparatus Required for Vibration Test

The apparatus which are used in free vibration test are

- Modal hammer.
- Accelerometer.
- FFT Analyzer.
- PULSE software.
- Specimens to be tested

The apparatus which are used in the Vibration test are discussed briefly one by one.

3.3.1 Modal hammer

The modal hammer excites the structure with a constant force over a frequency range of interest. Three interchange tips are provided which determine the width of the input pulse and thus the band width of the hammer structure is acceleration compensated to avoid glitches in the spectrum due to hammer structure resonance. For present experiment modal hammer type 2302-5 was used, which is shown in Fig. 6.

Fig. 1. Modal Impact Hammer (type 2302-5**)**

3.3.2 Accelerometer:

Accelerometer combines high sensitivity, low and small physical dimensions making them ideally suited for modal analysis. It can be easily fitted to different test objects using a selection of mounting clips. For the present experiment accelerometer type 4507 was used and which was fixed on plates by using bee wax. The accelerometer which is used in the present free vibration test is presented in Fig. 7.

Fig. 2. Accelerometer attached to Aluminium frame (type 2302-5)

3.3.3 Portable FFT Analyzer - type (3560C)

Bruel and kajer pulse analyzer system type –3560 software analysis was used to measure the frequency for any structure. It can be used for both free vibration as well as forced vibration study. The system has some channels to connect the cables for analyzing both input and output signals. Bruel Kajer FFT analyzer is shown in Fig. 8

Fig. 3. Bruel & Kajer FFT (spectrum) Analyzer

3.3.4 Display unit

This is mainly in the form of PC (Laptop).When the excitation occurs to the structure, the signals transfer to the portable PULSE and after conversion this comes in graphical form through the software and display on the screen of laptop. Mainly the data includes graphs of force Vs time, frequency Vs time resonance frequency data etc. The display unit is shown below in Fig. 9.

Fig. 4. Display unit used in Free Vibration Test.

3.4 Time-History Analysis:

Time history analysis of the frame was carried out to determine the response of the frame under a given dynamic loading.

- Time history analysis is the most natural and intuitive approach. The response history is divided into time increments Δt and the structure is subjected to a sequence of individual time-independent force pulses Δf (*t*). The nonlinear response is thus approximated by series of piecewise linear systems.
- Here time history records of Northridge Earthquake, Century City (17/01/1994) data recorded at LACC NORTH available from PEER Strong Motion Database (http://peer.berkeley.edu/svbin/download/qid=131&sid=428) (Fig-1) is used for the time history analysis. From the available time history functions file in SAP2000 two records lacc nor-1.th and lacc nor-2.th are chosen for analysis which are shown in the form of function graph as shown below. Each record is divided into 3000 points of acceleration data equally spaced at .020 sec. (Units: cm/sec/sec).
    - **lacc nor-1.th**

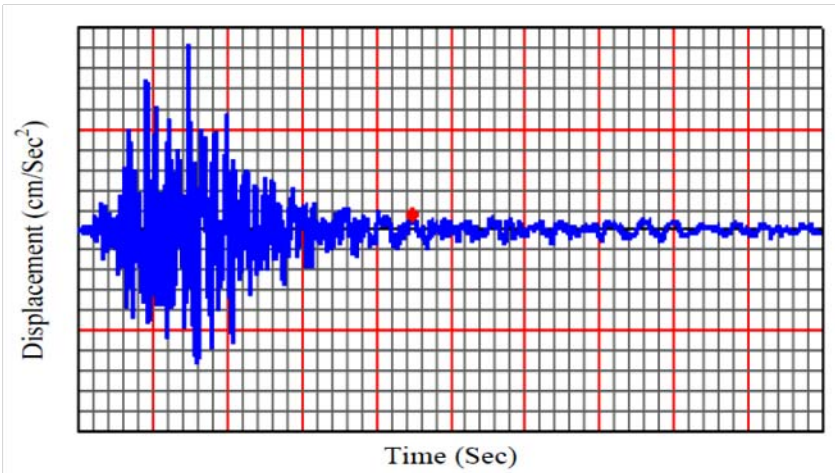

Fig. 5. Time history function record lacc nor-1.th (from SAP window).

- **lacc nor-2.th**

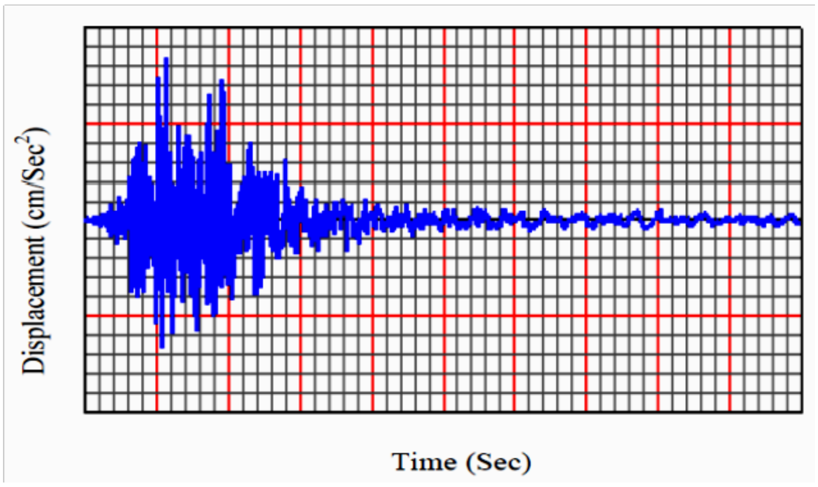

Fig. 6. Time history function record lacc nor-2.th (from SAP window).

3.5 Earthquake time histories

For input to the time-history analysis Northridge earthquake record was used. Non linear time history analysis was done by the use of Northridge earthquake record to get the result. In this study, the time history analyses of the selected building were carried out for bidirectional ground motions record of Northridge earthquake in two perpendicular directions.

Northridge earthquake

The Northridge earthquake was a massive earthquake that occurred on January 17, 1994 in Reseda, a neighbourhood in the city of Los Angeles, California, lasting for about 10–20 seconds. The earthquake had a "strong" moment magnitude of 6.7, but the ground acceleration was one of the highest ever instrumentally recorded.

3.6 Lead rubber bearing:

In the present paper, the isolators were initially designed to follow some available recommendations of the Uniform Building Code (UBC-97). The mechanical properties of the LRB isolation system were set to comply with a recommendation of the UBC-97 building code. The design parameters considered here are: the ratio Q/W of the characteristic strength Q over the total weight on the isolation system W, the yield force Fy , the isolator diameter

D, the lead core diameter d, the number of rubber layers n, and the layer thickness t. For design and analysis, the shape of the nonlinear force–deflection relationship, termed the hysteresis loop (represented as a bilinear curve as shown in Fig. 4), has an elastic (or unloading) stiffness k_e and a yielded (or post-elastic) stiffness k_p.

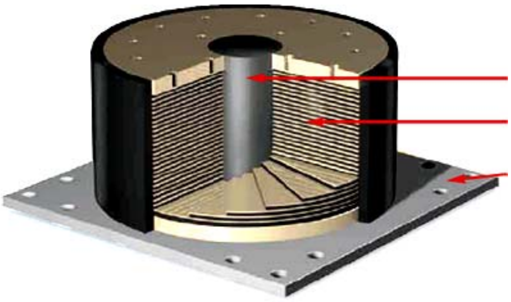

Fig. 7. The Lead rubber Bearing. (The top mounting plate is not shown)

Table 1. Parameters of basic hysteresis loop of isolator for bilinear modelling

Symbols	**Terms**
k_e	= Elastic stiffness
K_2 or K_p	= Yielded Stiffness
K_{eff}	= Effective Stiffness
Δ	= Designed displacement
D_y	= The yield displacement of the isolator.
ζ_{eff}	= Effective damping ratio
F_y	= Yield force
T^{iso}	= Fundamental isolation period

3.6.1 Elastic stiffness (k_e):

Elastic stiffness k_e is defined as the ratio of the yield strength to the yield displacement. This is the initial stiffness of the isolator, its value is dominated by lead core size and is important in controlling the response to service load such as wind.

Its value is expressed by the equation

$$k_e = \frac{f_y}{D_y}$$

f_y = The yield force of the isolator.

D_y= The yield displacement of the isolator.

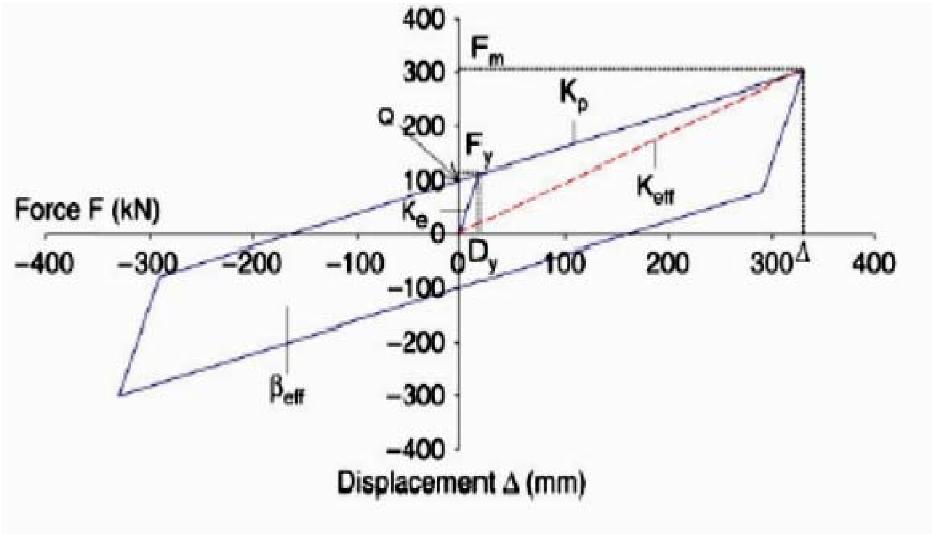

Fig. 8. Hysteresis loop of the LRB.

3.6.2 Yielded Stiffness (K_2 or K_p):

This is the secondary stiffness of the isolator and is a function of the shear modulus, total height and area of the rubber.

Its post-yield stiffness is given by the formula,

$$k_p = \frac{G \, . A_r}{t_r} f_l$$

Where,

G = Shear modulus of the rubber.

A_r = Cross-sectional area of the rubber layers

t_r = Total thickness of the rubber consisting of n-layers.

f_l = factor given by 1.15.

3.6.3 Effective Stiffness (K_{eff}):

This is the isolator force divided by the displacement. This is a displacement-dependent quantity. The average or effective stiffness k_{eff} is defined as the ratio between the force F_m (Force at designed displacement) (from fig. 4), occurring at a specified LRB isolator displacement Δ (Designed displacement), and the Δ (Designed displacement):

$$k_{eff} = \frac{F_m}{\Delta}$$

The effective stiffness k_{eff} can also be expressed as a function of the characteristic strength Q as in the following equation:

$$k_{eff} = k_p + \frac{Q}{\Delta} \quad (\text{When } \mathit{\Delta} > D_y)$$

Where,

D_y = Yield displacement as shown in Fig. 4.

On the other hand, when the designed displacement is $\mathit{\Delta} < D_y$, the effective stiffness (k_{eff}) is equal to elastic stiffness (k_e). The force F_m can be defined as

$$F_m = Q + k_p\,\Delta$$

While the yield force F_y can be obtained from

$$F_y = Q + k_p\,D_y$$

For lead-rubber bearings in which the elastic stiffness is approximately equal to 6.5, the yield displacement can be estimated as:

$$D_y = \frac{Q}{5.5\,k_p}$$

The area ED of the hysteresis loop can be obtained from the equation,

$$ED = 4Q(\Delta - \Delta_{y)}$$

This area represents the energy dissipation at each cyclic motion of LRB isolator. Then, the effective damping ratio ζ_{eff}, which produces the same amount of damping energy dissipation as the hysteretic energy dissipated at each cyclic motion of the LRB isolator, is expressed as;

$$\zeta_{eff} = \frac{ED}{2\pi k_{eff} \Delta^2}$$

Finally, the fundamental isolation period T^{iso} is given by the equation

$$T^{iso} = 2\pi \sqrt{\frac{M}{\sum k_{eff}}}$$

Where, M is the total mass on the isolation system, including the mass of the superstructure and the mass of the isolation system. The term $\sum k_{eff} = K_{eff}$ is the total effective stiffness of the isolation system.

$$C_{eff} = 2\zeta_{eff}\sqrt{M\,K_{eff}}$$

Where,

C_{eff}= effective damping coefficient

High-damping rubber bearings are made of specially compounded rubber that exhibits effective damping between 0.10 and 0.20 of critical. The increase in effective damping of high-damping rubber is achieved by the addition of chemical compounds that may also affect other mechanical properties of rubber.

3.6.4 Vertical Stiffness (K_v):

This is the vertical stiffness of the isolator.

3.6.5 Yield Force (F_y):

The yield force is the point in the model at which the initial stiffness changes to secondary stiffness. In reality there is a smooth transition from one stiffness to the other, rather than a well defined point. This value is mainly used in analytical modelling.

The characteristics strength is given by equation;

$$Q = A_{pb}\sigma_{ypb}$$

Where,

A_{pb} = Area of the lead core.

σ_{ypb} = The yield strength of the lead core (Ranging between 7.0 and 8.5 MPa).

3.6.6 Hysteretic Strength (Q_d):

This is the force-axis intercept of the isolator hysteresis loop. This parameter relates to damping and isolator response to service loads.

3.6.7 Hysteresis Loop:

This is the cyclic force-displacement plot generated by the shear testing of an isolator.

3.6.8 Energy Dissipated per Cycle (EDC):

This is the area of the hysteresis loop per cycle. This value is a measure of the damping of the isolator.

3.6.9 Design basis Earthquake (DBE):

Design basis Earthquake represents the ground motion that has a 10% chance of being exceeded in 50 years.

3.6.10 Maximum Credible Earthquake (MCE):

Maximum Credible Earthquake represents the ground motion that has a 2% chance of being exceeded in 50 years.

3.7 Isolation System

The isolation systems, which can be elastomeric systems, exhibit highly nonlinear behaviour. Nonlinear behaviour is restricted to the base and the superstructure is considered to be elastic at all times. All of the isolation bearings in this study are modelled by a bilinear model, based on three parameters: elastic stiffness (K_1), yielded stiffness (K_2), and characteristics Strength (Q). Refer Fig. 5 for details. The elastic stiffness (K_1) is either estimated from elastomeric bearing tests or as a multiple of K_2 for lead plug bearing. The characteristics strength (Q) is estimated from the hysteresis loops for the elastomeric bearings. For lead plug bearings Q is given by the yield stress in the lead and the area of the lead. The post-yield stiffness can be accurately estimated or predicted for the bearing. The effective stiffness, defined as the secant slope of the peak-to-peak values, in a hysteresis loop, is given by:

$$\mathrm{K_{eff}} = \mathrm{K_2} + \mathrm{Q/D} \quad \text{where,} \quad D > D_y$$

Here, $D_y = \frac{Q}{K_1 - K_2}$ is the yield displacement.

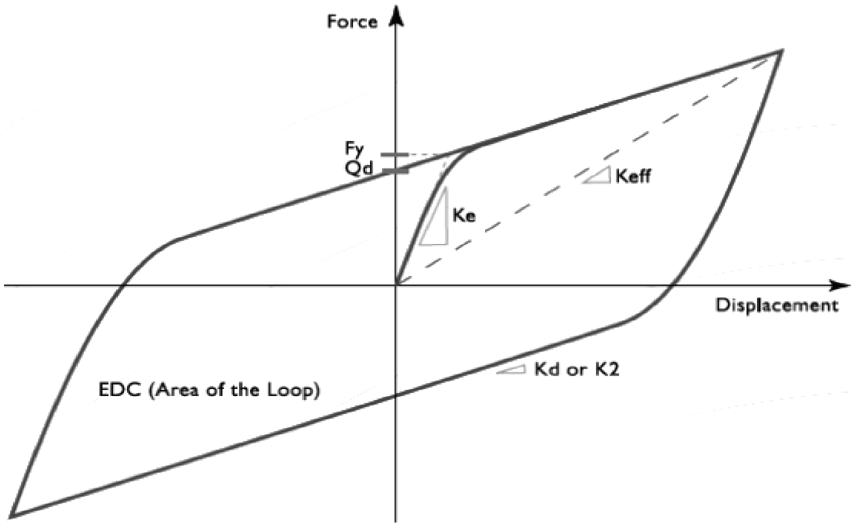

Fig. 9. Parameters of basic hysteresis loop of isolator for bilinear modelling

The natural frequency ω is given by:

$$\omega = \sqrt{\frac{K_{eff}\,g}{W}}$$

$$\omega = \sqrt{{\omega_0}^2 + \mu\,\frac{g}{D}}$$

Where

$$\mu = {}^{Q}/_{W}\,, {\omega_0}^2 = \sqrt{\frac{K_2 g}{W}}$$

The effective time period T is given by

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

$$\omega = \frac{2\pi}{\sqrt{\omega_0^2 + \mu \frac{g}{D}}}$$

And the area of the hysteresis loop is (the energy dissipated per cycle), W_D, is given as;

$$W_D = 4Q(D - D_y)$$

The effective damping β_{eff} is given by

$$\beta_{eff} = \frac{area\ of\ the\ hysterisis\ loop}{2\pi K_{eff}\ D^2}$$

This can be expressed in non dimensional quantities by defining a non-dimensional displacement.

$$y = {}^{D}/_{D_y}$$

And a non dimensional characteristics strength

$$a = {}^{Q}/_{(K_2\, D_y)}$$

Then the effective damping becomes

$$\beta_{eff} = \frac{2a}{\pi}\ \frac{y-1}{(y+a)*y} \qquad\qquad y \geq 1$$

3.8 Materials of LRB

Table 2. Materials of lead rubber bearing modelled as bilinear model

1	Rubber
2	steel
3	Lead core
4	Mounting plate. (Steel mounting plate and top mounting plate)

3.9 Moment Frame

Selected building is a two-story steel framed building with two bays along both X and Y direction. The building has uniform bay width of 30ft in both the horizontal direction. Also, 12ft storey height is uniform in the vertical direction. A computer model of the building is generated using commercial software SAP2000 v12. Beams and Columns are modelled by 3D frame elements. Both the beams and the columns of the frames of are assumed as steel having I/Wide Flange section. The rigid beam-column joints were modelled by using end offsets at the joints, to obtain the bending moments and forces at the beam and column faces. The concrete floor slabs were assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was distributed as triangular and trapezoidal load to the surrounding beams.

CHAPTER

4

RESULT AND DISCUSSION

First modal analysis of a benchmark problem is solved using SAP2000. The benchmark problem is taken from literature (Bezerra and Carneiro, 2003). The details of the problem are discussed in the following section.

4.1 Modal Analysis of a Benchmark Problem

Modal analysis of a typical building structure frame is done to determine the dynamic parametres like natural frequency, time period, modal participating mass ratios and their corresponding mode shapes. Typical building structure frame (Fig. 10) made of reinforced concrete has four floors and composed of columns 3.0m height and of cross section 30×50 cm^2 with $I = 3.1\times10^{-3}m^4$, and beams with span of 4.5m, cross-section 24x55 cm^2, and inertia $I = 3.5\times10^{-3}m^4$. The first natural frequency of the building is 2.3Hz.

From the modal analysis time period, frequencies are noted for modes with considerable mass participation. These are the important modes of consideration. The first natural frequency of the building found in SAP2000 is 2.3195 Hz. (Table-3)

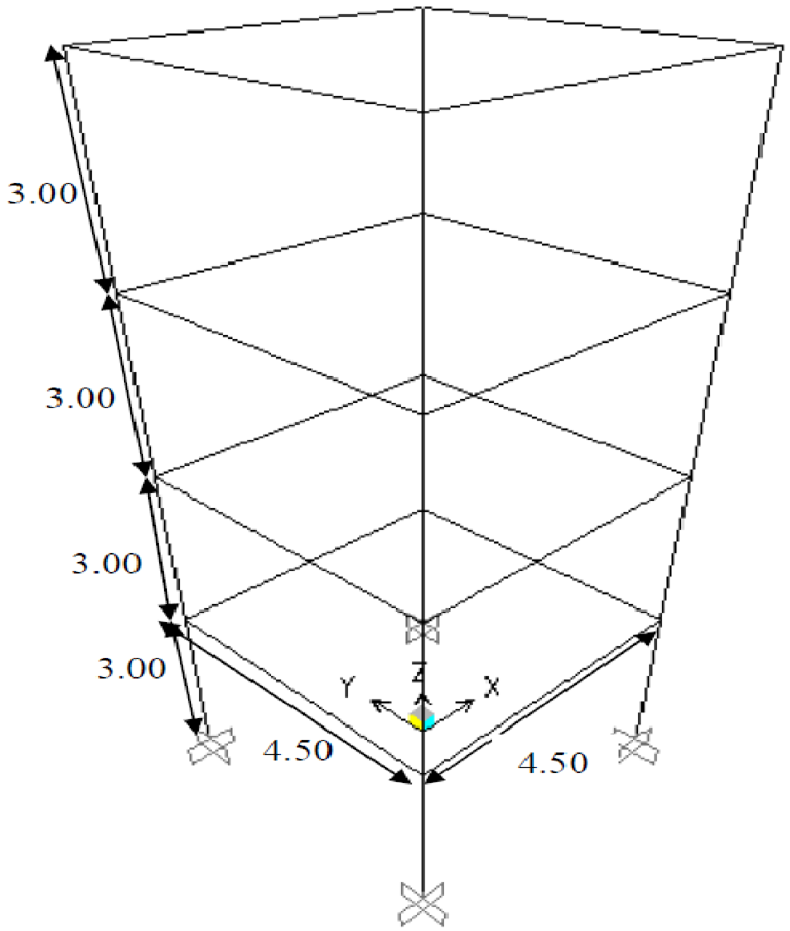

Fig. 10. Structural model of Building (from SAP window)

Table 3. Time period and frequency of the building for first three modes.

Mode	T (s)	f (cps)	UX (%)	UY (%)	RZ (%)
1	0.43	2.3195	0	86	0
2	0.35	2.8713	0	0	85
3	0.34	2.9387	83	0	0

	f (cps)	
	(Bezerra and Carneiro, 2003)	SAP output
First mode	2.3	2.3195

4.2 Vibration measurement:

4.2.1 Details of the Test Frame

The Aluminium frame present in the structural engineering laboratory, NIT Rourkela is taken into consideration for analysis. Natural frequency and mode shapes of the fixed base frame were found out by carrying out modal analysis method using SAP 2000 v12.0.0 software package. The result of the test was validated by experimental work with the help of FFT Analyser and shake table test. Fig. 11 presents the 3-D view of the aluminium frame modeled in SAP 2000.

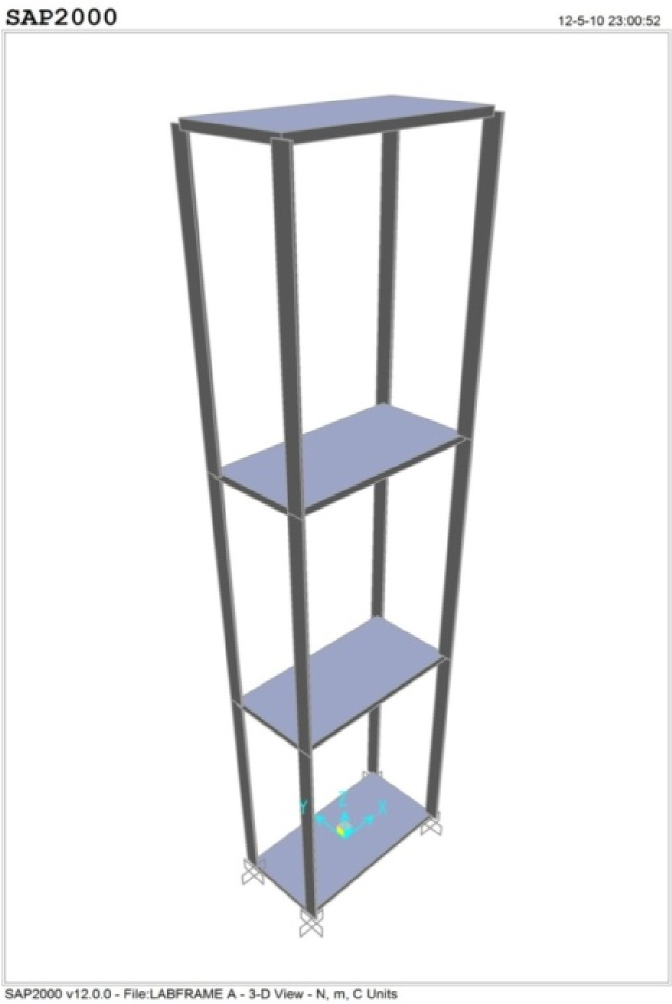

Fig. 11. Computational model of the Aluminium frame

4.2.2 Properties of the aluminum frame

- 3 storeys × 1 bay × 1 bay aluminum frame available in Structural Engineering laboratory, NIT Rourkela.
- Plan dimension of the building is 0.303m × 0.148m with a storey height of 0.4 m. Typical floor plan of the building is given in Fig. 12.
- Columns are of rectangular size (2.5cm × 0.3cm) directly supporting the slabs.
- Slabs having plan dimension 303mm ×148mm and thickness 11mm.

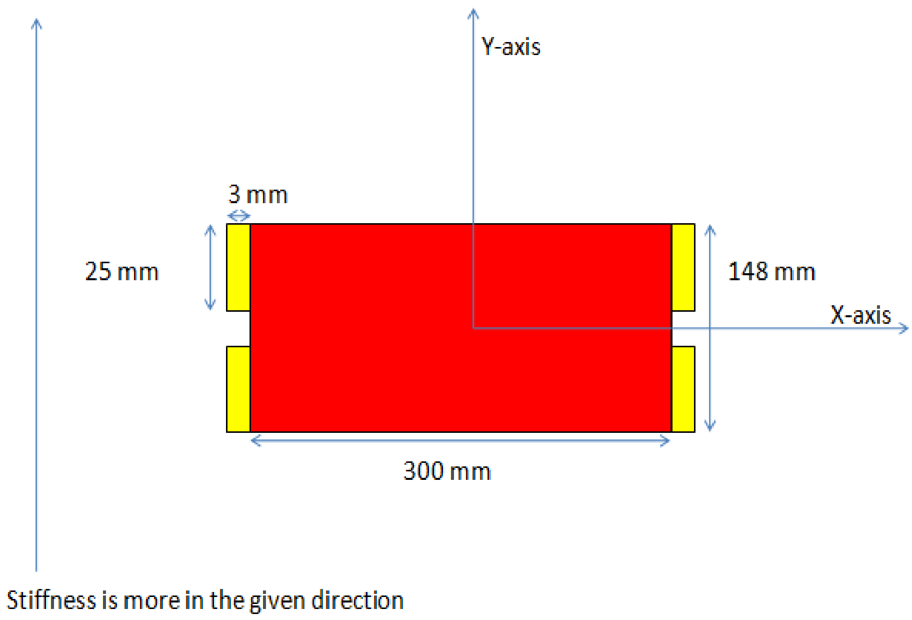

Fig. 12. Typical floor plan of the structural model of Aluminum frame

4.2.3 Modal Analysis of the frame:

A modal analysis is always linear. The modes are properties of the structure. By carrying out modal analysis method using SAP 2000 v12.0.0 software package following results were obtained.

Table 4. Modal period, frequencies and Modal participating mass ratios

Mode	T (s)	*f* (cps)	UX (%)	UY (%)
4	0.332	3.018	92	0
8	0.119	8.403	99	0
15	0.047	21.276	0	88
19	0.016	62.500	0	99

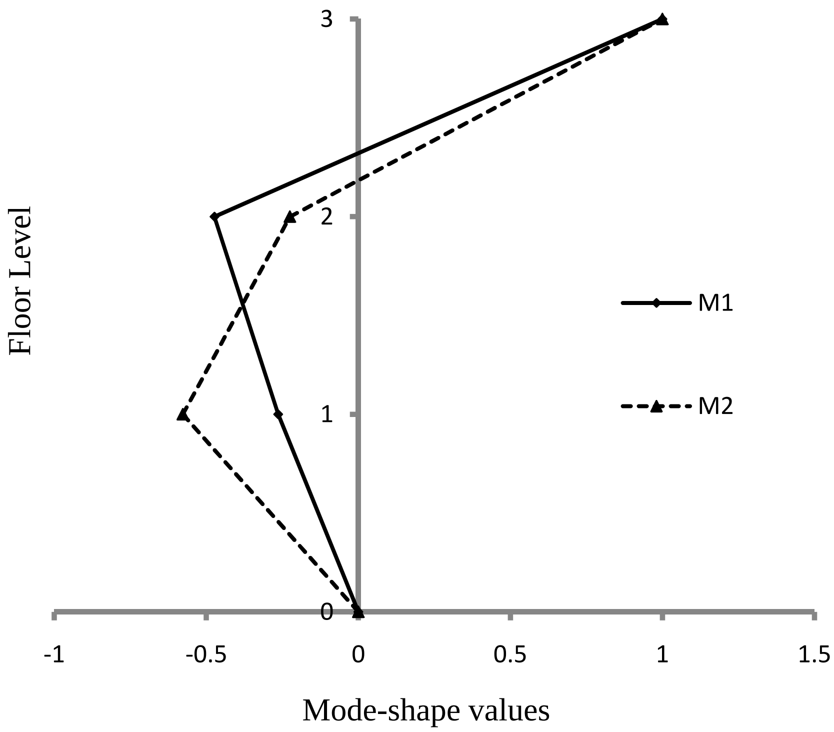

Fig. 13. Mode shapes along X-direction

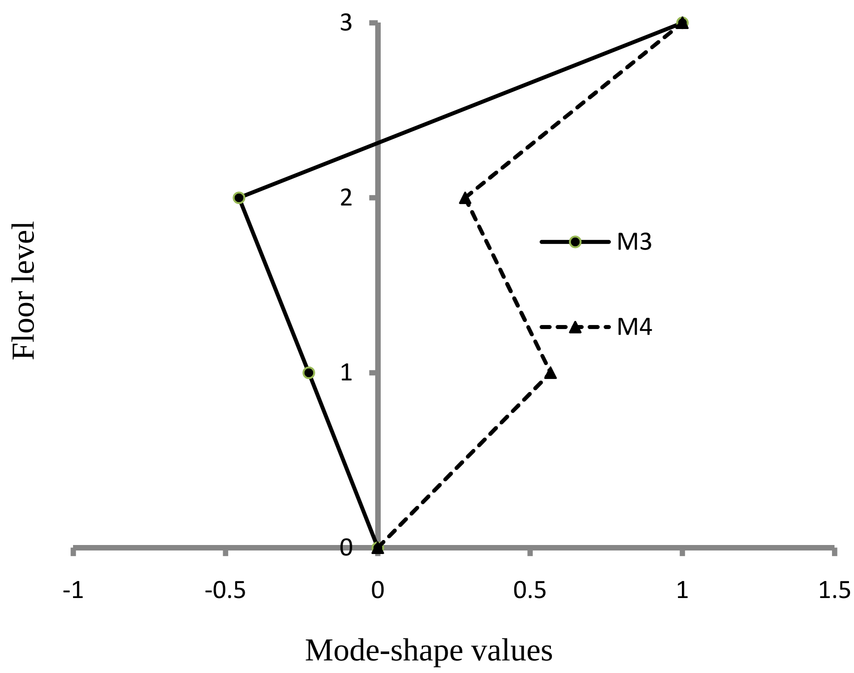

Fig. 14. Mode shapes along Y-direction

From table 4 it is concluded that in 4th and 8th mode, modal participating mass ratios are 92 and 99 % respectively along X-direction, hence the mode shapes corresponding to that frequencies are vital for the structure. Fig. 13 shows that mode shapes M_1 and M_2 corresponding to the above frequencies. From Table 4 it is also concluded that in 15th and 19th mode modal participating mass ratios are 88 and 99 % respectively along Y-direction, hence the mode shapes corresponding to that frequencies are vital for the structure. Fig. 14 presents the mode shapes M_3 and M_4 corresponding to the above frequencies.

Comparison with experimental results and results obtained using SAP 2000 v12.0.0 software package are presented to verify the accuracy.

4.3 FFT Analyzer

From the Time (Excitation) input graph, Auto-spectrum (Excitation) input graph it is found that the object has been striked properly. (Fig. 1 & Fig. 2). From the Time (Resonance) input graph, it is found that the object has been striked properly and any overloaded response is avoided. From Autospectrum (Excitation) window the approximate smoothness of the curve indicates the single hitting by hammer. From other graphs like frequency response, frequency resonance input graph the free vibration properties are obtained whose value approximately coincides with the value obtained from software package SAP v12.0.0.

4.4 Shake table test:

Fig. 15. Experimental set up for the Shake table test.

Fig. 16. Response of the Experimental set up for the Shake table test.

Peak response of the frame (Fig. 16) for an exciting frequency close to the natural frequency obtained from SAP 2000.

4.5 Time-History Analysis of the frame:

The response (*i.e.* Displacement, Velocity, and Acceleration) of the Aluminium frame subjected to a selected earthquake ground motion was found out by non-linear time history analysis using SAP 2000 v12.0.0. The selected earthquake ground motion is Northridge Earthquake record. (Northridge earthquake, January 17, 1994, Reseda, a neighbourhood in the city of Los Angeles, California, USA).

4.5.1 Response of the frame:

4.5.1.1 Displacement:

Displacement of the frame subjected to time history analysis is recorded in each node in both X-direction and Y-direction. No displacement (Fig. 17) is recorded at the base since the base is in the fixed condition (Fig. 11). It is to be noted that that maximum displacement is achieved at all the nodes at the same time i.e. at 6.44 sec or at time step 322 along X-direction. Storey displacement and inter-storey drift are calculated and plotted graphically as shown Fig. 20.

From the graph it is clear that inter-storey drift (Fig. 20) is more in the first storey which goes on decreasing in successive upper storeys. Displacement of the frame in each node in Y-direction is found to be very less as compared to the displacement of the frame in the X-direction when it is subjected to time history force.

4.5.1.2 Velocity:

Velocity of the frame subjected to time history analysis is recorded in each node in both X-direction and Y-direction. No velocity is recorded at the base since the base is in the fixed condition. Velocity envelope/ Profile graph (Fig. 18.) is plotted for the frame along X-direction.

From Fig. 18, it is clear that slope of the storey velocity graph goes on steeper for the successive upper storeys as compared to lower storey, which indicates storey velocity is more in the lower storey and it goes on decreasing in the successive upper storeys.

4.5.1.3 Acceleration:

Acceleration of the frame subjected to time history analysis is recorded in each node of the frame. No acceleration is recorded at the base since the base is in the fixed condition. Acceleration envelope graph is plotted for the frame along X-direction.

From the graph (Fig. 19) it is clear that slope of the storey acceleration graph goes on steeper for the successive upper storeys as compared to lower storey, which indicates storey acceleration is more in the lower storey and it goes on decreasing in the successive upper storeys.

4.5.1.4 Result

- Modal analysis of the fixed base aluminum frame is done to determine its natural frequency and mode shape followed by its time-history analysis using time history record of Northridge earthquake (January 17, 1994 in Reseda, a neighbourhood in the city of Los Angeles, California, USA) at an interval of .02 sec for 60 sec. duration to determine the response of the frame ender dynamic loading.

- It was concluded that the responses (displacement, inter-storey drift, velocity, acceleration) of the structure is more in lower storey as compared to the upper storeys.

- Maximum displacement is achieved at all the nodes at the same time i.e. at 6.44 sec or after 322 time history steps. Envelope of velocity and acceleration response is plotted.

- Inter-story drift is more in lower storeys and it goes on decreasing in the successive upper storeys.

- For dynamic loading design of building structures, we have to consider the dynamic loading response demand and go for the methods like strengthening the stiffness, strength, and ductility of the structures which has been in common use for a long time. Therefore, the dimensions of structural members and the consumption of material are expected to be increased, which leads to higher cost of the buildings as well as larger seismic responses due to larger stiffness of the structures.

- Base isolation decreases the dynamic loading response demand of the structure to a certain extent as compared to its bare frame by absorbing and dissipating the energy imparted on the structure due to dynamic loading.

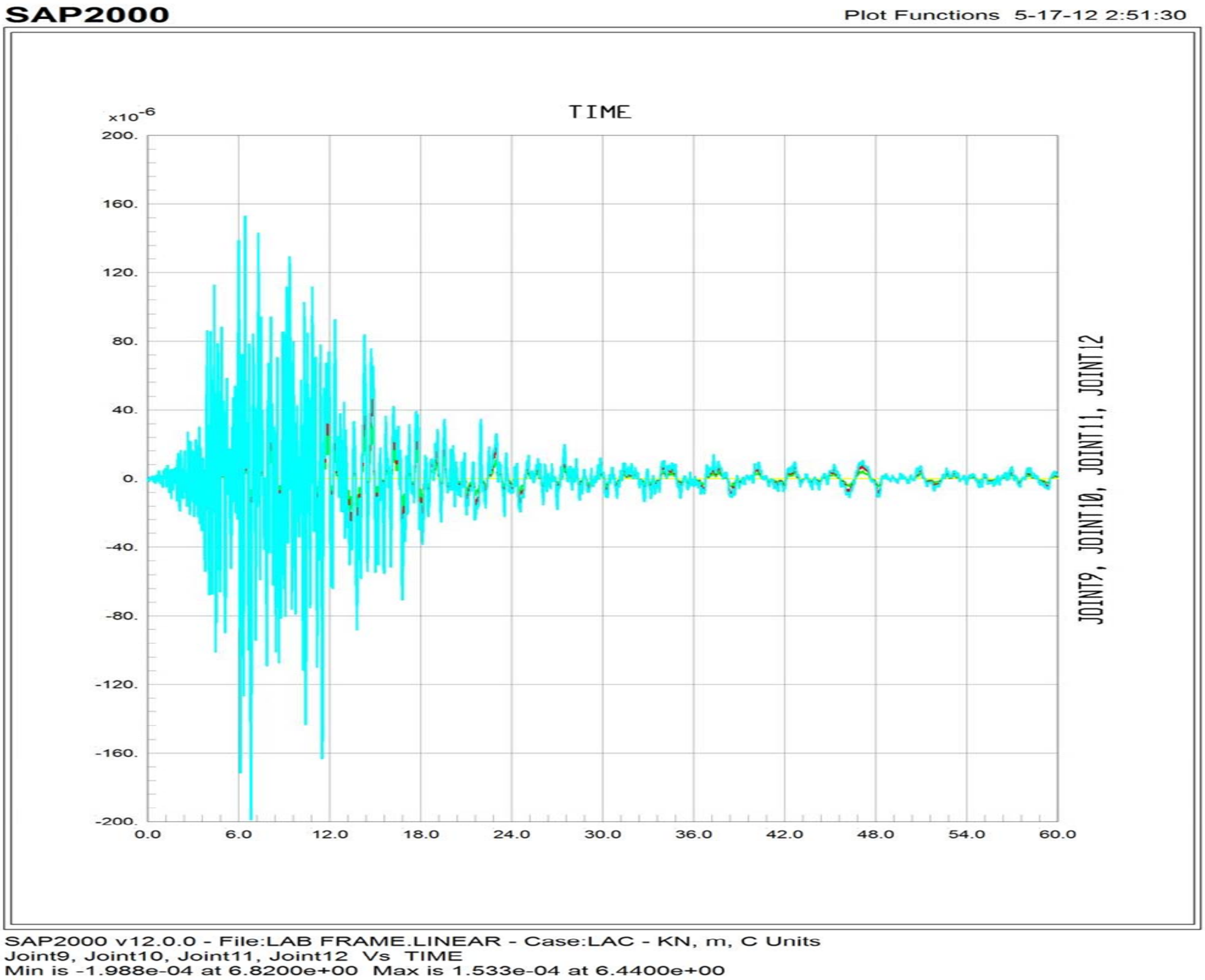

Fig. 17. Time history of displacement response of the Aluminium frame along Y-direction

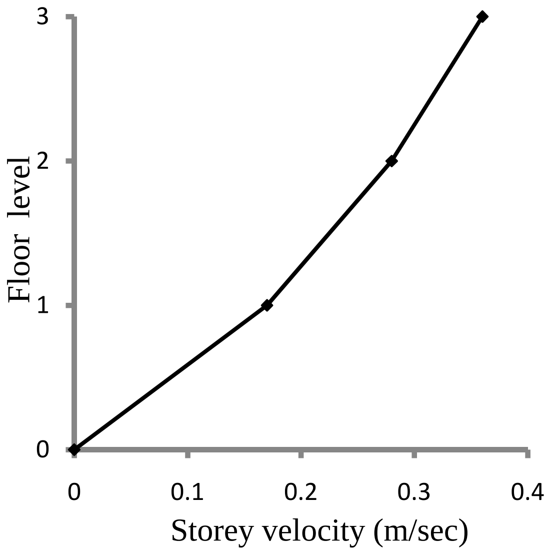

Fig. 18. Velocity Profile

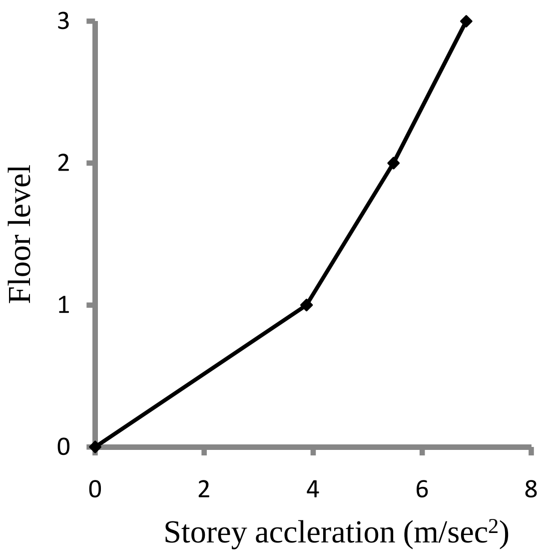

Fig. 19. Acceleration Profile

Table 5. Inter-storey drift of Aluminium Framed Structure

Storey No	Floor No.	Storey Height (m)	Displacement (mm)	Inter-storey Displacement (mm)	Inter-storey Drift (%)
1	0	0.4	0	9.23	2.30
	1		9.23		
2	1	0.4	9.23	7.20	1.80
	2		1.64		
3	2	0.4	1.64	3.82	0.95
	3		2.02		

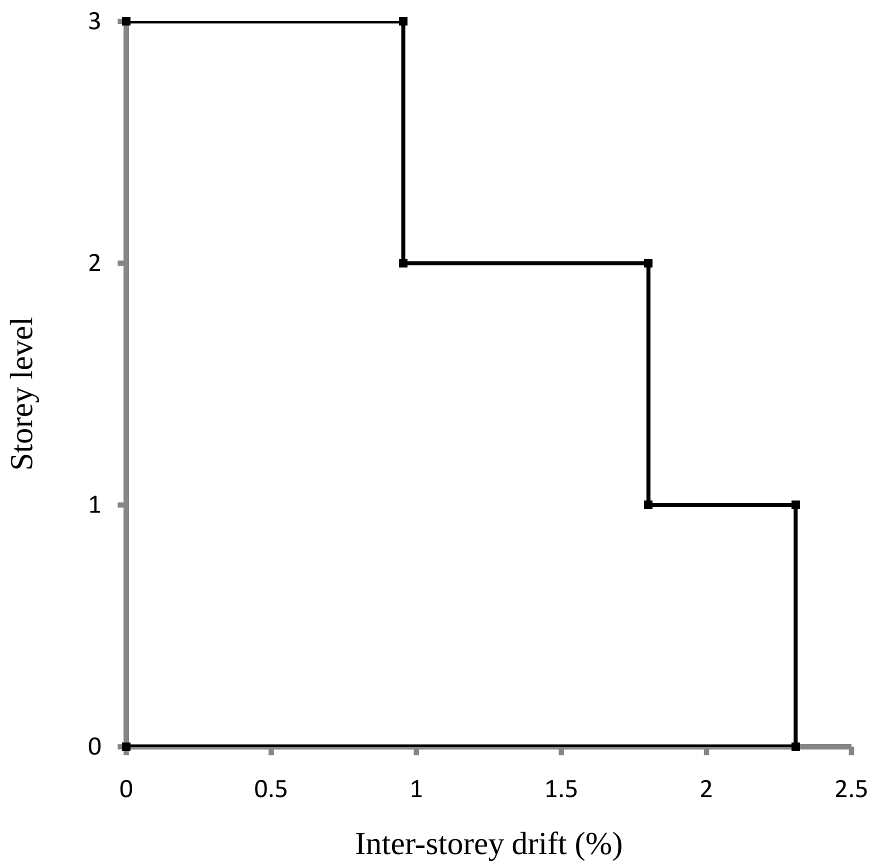

Fig. 20. Inter-storey drift of the Aluminium frame

4.6 Performance of base isolated framed structure as compared to its fixed base framed structure:

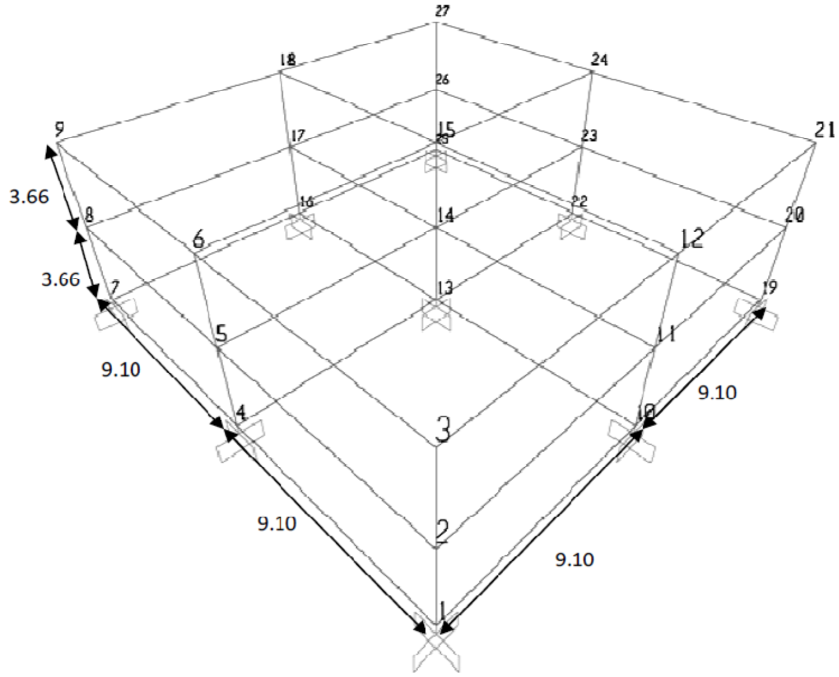

Fig. 21. Structural model of fixed base Framed Structure (From SAP window)

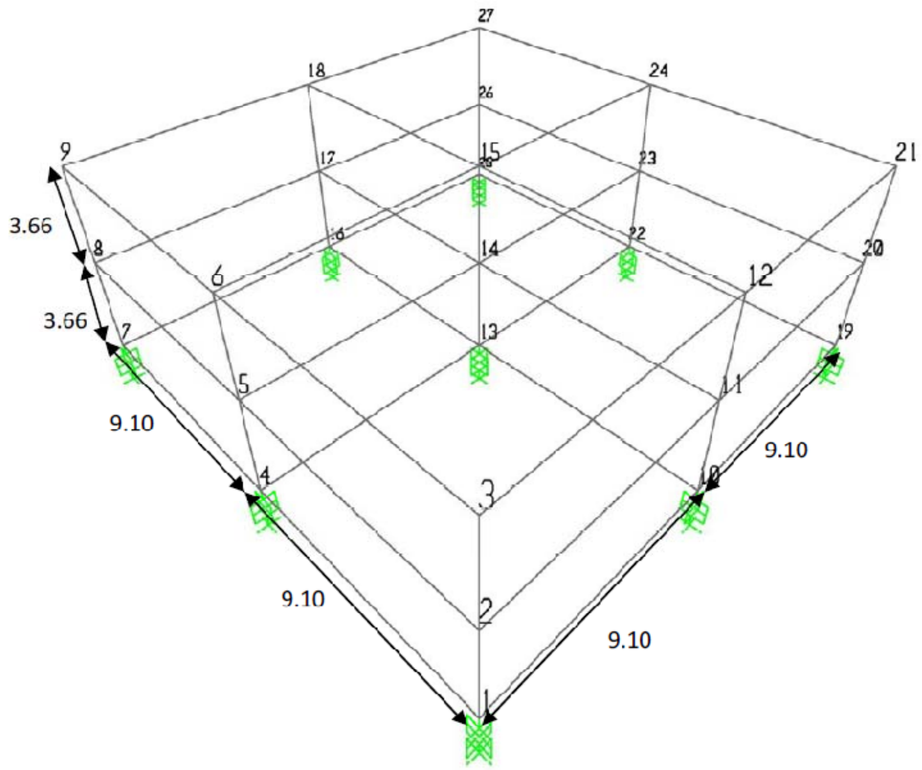

Fig. 22. Structural model of Base Isolated Framed Structure (From SAP window)

4.6.1 Moment Frame

Selected building is a two-story steel framed building (Fig. 21 and Fig. 22) with two bays along both X and Y direction. The building has uniform bay width of 30ft in both the horizontal direction. Also, 12ft storey height is uniform in the vertical direction. A computer model of the building is generated using commercial software SAP2000 v12 (Fig. 21 and Fig. 22). Beams and Columns are modelled by 3D frame elements. Both the beams and the columns of the frames of are assumed as steel having I/Wide Flange section. The rigid beam-column joints were modelled by using end offsets at the joints, to obtain the bending moments and forces at the beam and column faces. The concrete floor slabs were assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was distributed as triangular and trapezoidal load to the surrounding beams.

The isolation elements (Fig. 7) used are modelled by biaxial behaviour of elastomeric bearings (Fig. 9). The lead-rubber bearings are modelled using the biaxial model for elastomeric bearings. In the present study, bilinear isolators such as the commonly used lead rubber bearing (LRB) isolation systems were investigated. The introduction of LRB isolators in the nonlinear time-history analysis was achieved by activating the nonlinear link element of SAP. Post yield stiffness ratio is 0.20, having yield strength 34.47 x 10^3 kN/m^2 and having effective stiffness 44.48 x 10^3 kN in the U1 direction. The performance of a base isolated framed structure with a fixed base otherwise similar framed structure was compared one by one as described below using computer program SAP 2000 v12.0.0 to conclude the effectiveness of base isolation using bilinear behaviour of elastomeric bearing.

4.6.2 Shift in time period (S):

Base isolation shifts the fundamental period of the structure from the dominant period of the earthquake. It generally shifts the fundamental time period of the structure more than 2 seconds. The dominant periods of the earthquake are in the 0.2 to 0.6 second range (http://www.dis-inc.com/pdf_files/DIS_BASE_ISO.pdf). The severe accelerations of an earthquake are avoided due to period shift provided by isolation. From the table 6, the shift in time period is shown for successive 3 modes.

Table 6. Time period response of Base-Isolated Moment Framed Structure

Mode	Fixed base	Isolated base
1	0.56	3.11
2	0.42	3.09
3	0.40	2.70

4.6.3 Horizontal flexibility:

Base isolation increases the horizontal flexibility of the structure at the base in both X and Y direction as compared to the conventional fixed base one which is indicated from the curve (Fig. 23 and Fig. 24) below. Introduction of horizontal flexibility causes dissipation of horizontal component of earthquake ground motion effectively, so that transmission of ground motion to the superstructure is less which results in less damage and prevention of collapse in structure.

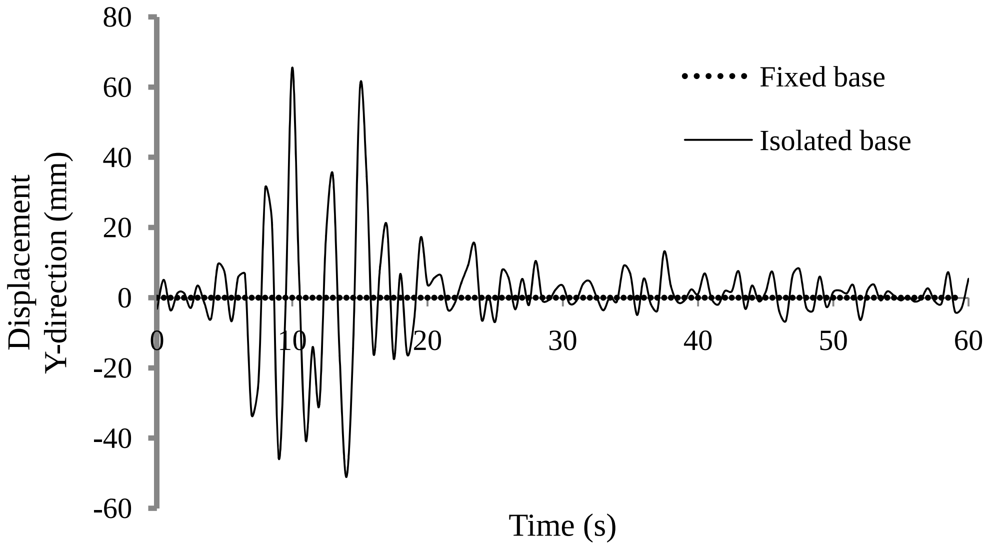

Fig. 23. Time history of displacement response at the base along Y-direction.

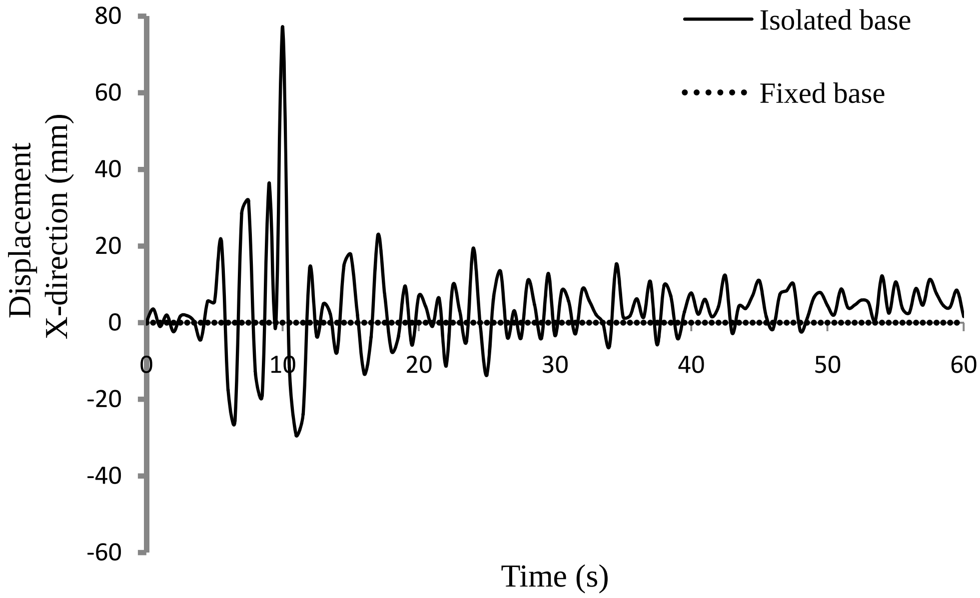

Fig. 24. Time history of displacement response at the base along X-direction.

4.6.4 Force–displacement loop:

The loop shows that the load-deflection behaviour of the isolation bearings used in the study is mildly non-linear (Fig. 25). The area of the hysteresis loop per cycle gives us the idea about Energy Dissipated per Cycle (EDC). This value is a measure of the damping of the isolator.

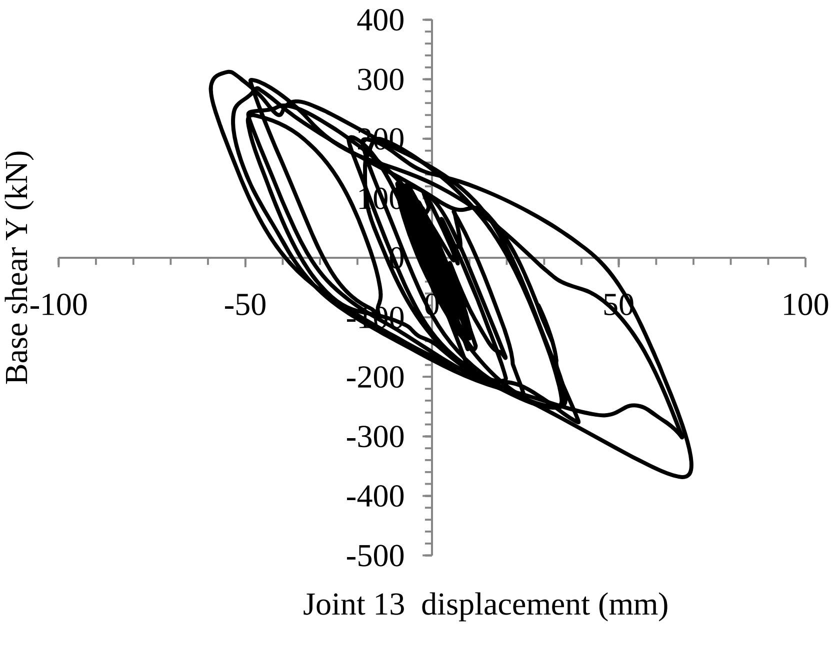

Fig. 25. Typical force–displacement relationship of the isolator.

4.6.5 Rigidity of the superstructure above isolator:

Entire super structure behaves almost as a rigid one due to the presence of isolator. Here floor displacement and roof displacement response curves of the isolated structure are plotted which equivalents (Fig. 26). The superstructure of a base-isolated building is relatively rigid compared with the isolation system. This can lead to idealization of the superstructure as a rigid body, modelling the base-isolated structure as a single-degree-of-freedom system. Introduction of flexible superstructure decreases the effectiveness and goal of using base isolation. Entire superstructure is considered to be rigid and linear. The nonlinearity are supposed to be concentrated in isolator level only.

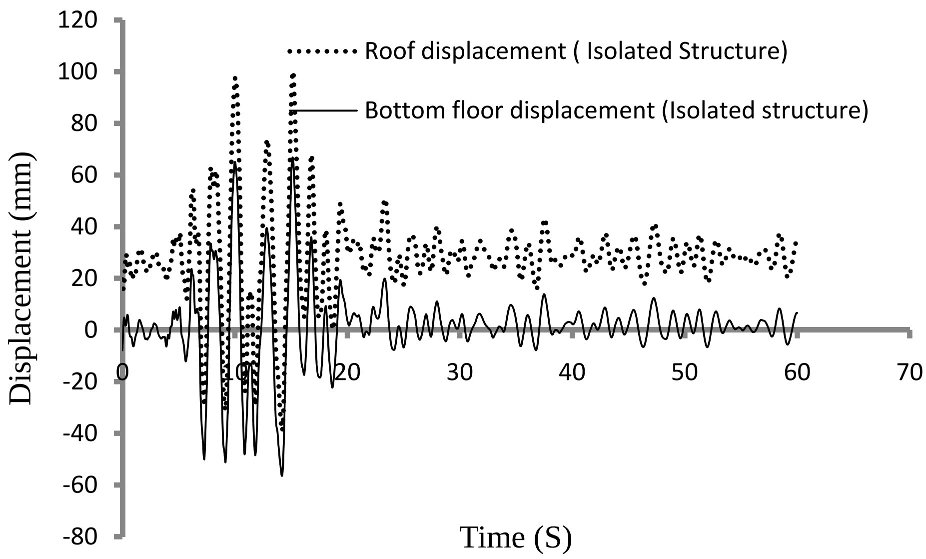

Fig. 26. Time history of displacement response at bottom floor and at the roof.

4.6.6 Time histories of Base shear response:

Base shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. It has been observed that base isolation process is very effective in reducing the base shear as compared to conventional fixed base structure. As a result the potential damage to the bottom level of two-storey frame is reduced. The seismic demand of the structure to be considered during design is drastically decreased. (Fig. 27.)

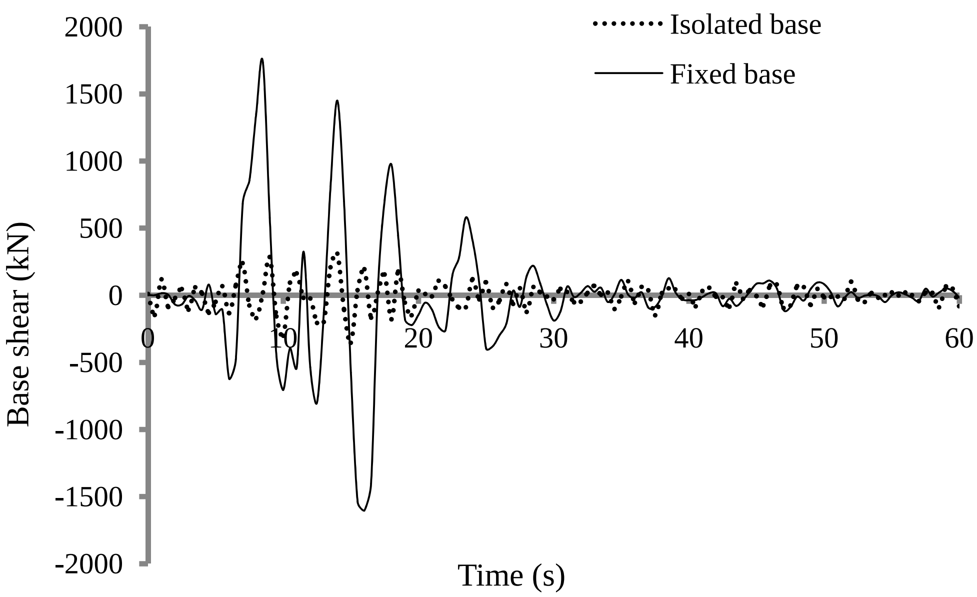

Fig. 27. Time history of base shear response.

The summary of the base shear response presented in Fig. 27 is presented in Table 7 as follows:

Table 7. Base shear response of base-isolated framed structure as compared to its fixed base framed structure.

		Isolated	Fixed
Maximum	Base Shear (kN)	307.47	1736.00
	Time (s)	14.00	16.00
	Reduction in base shear = 82.30 %		
Minimum	Base Shear (kN)	-351.33	-1604.00
	Time (S)	14.00	8.50
	Reduction in base shear = 78.10 %		

4.6.7 Time history of Displacement response:

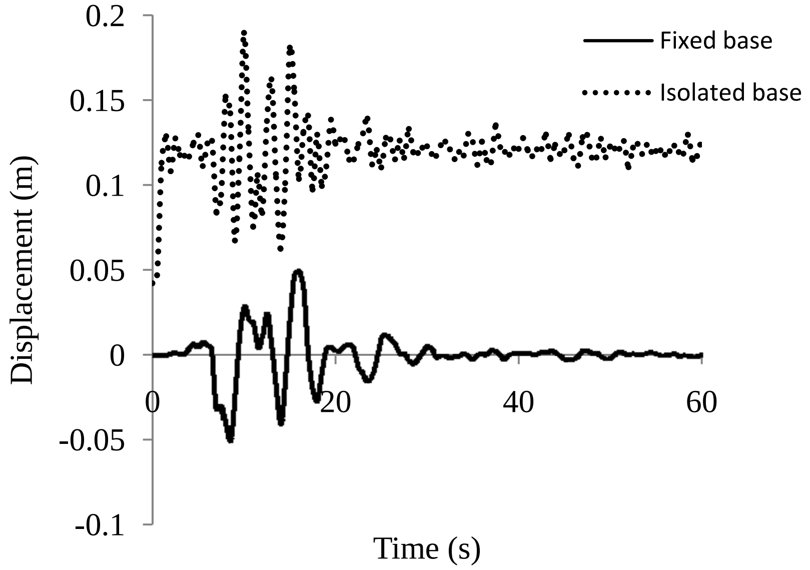

Fig. 28. Time history of displacement response at roof.

4.6.8 Time history of Velocity response:

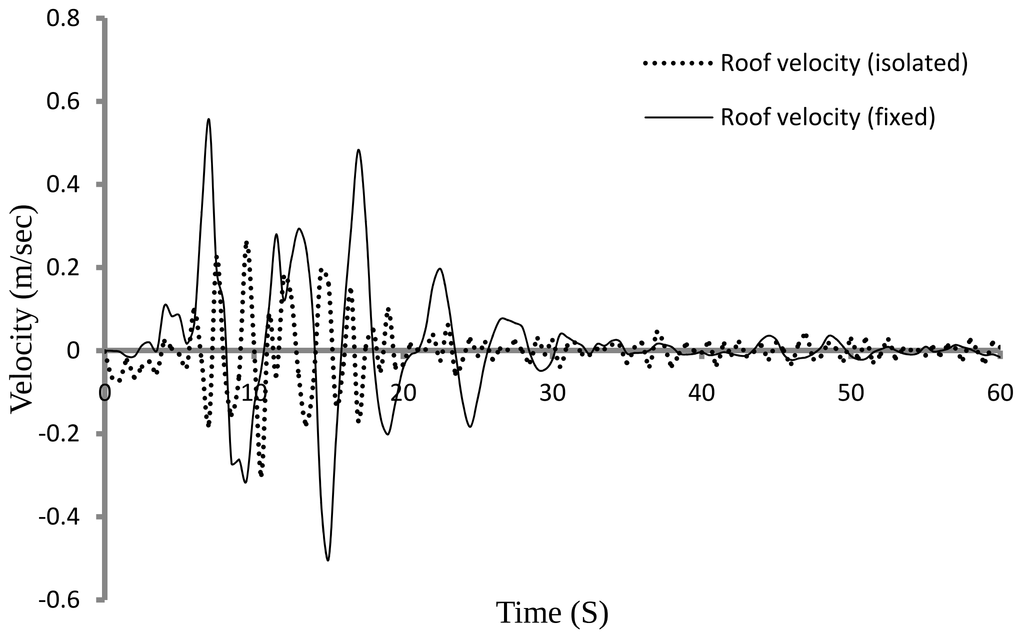

Fig. 29. Time history of velocity response at roof.

Table 8. Velocity response of base-isolated framed structure as compared to its fixed base framed structure.

		Isolated	Fixed
Maximum	Velocity (m/sec)	0.26	0.55
	Time (S)	9.50	7.00
			Reduction = 53.30 %
Minimum	Velocity (m/sec)	-0.29	-0.50
	Time (S)	10.50	15.00
			Reduction = 40.60 %

Fig. 28 presents the displacement response of the building roof subjected to Northridge Earthquake at base. This figure shows that the roof displacement of isolated base and fixed base building are quite similar in nature except there is a translational shift of the mean displacement by 0.12m (approx.) in case of isolated base building. However, the displacement range (maximum – minimum) is little more for isolated base.

Similar results are shown in Fig. 29 for the velocity response. This figure shows that unlike displacement response the velocity of both the buildings is oscillating with a zero mean velocity. It is found that the velocity range (maximum – minimum) is approximately 60% more for fixed base building. Table 8 presents the summary of the roof velocity response of the two building models.

Similar results are shown in Fig. 30 for the acceleration response. This figure shows that unlike displacement response the acceleration response of both the buildings is oscillating with a zero mean velocity. It is found that the acceleration range (maximum – minimum) is approximately 30% more for fixed base building. Table 9 presents the summary of the roof acceleration response of the two building models.

4.6.9 Time history of Acceleration response:

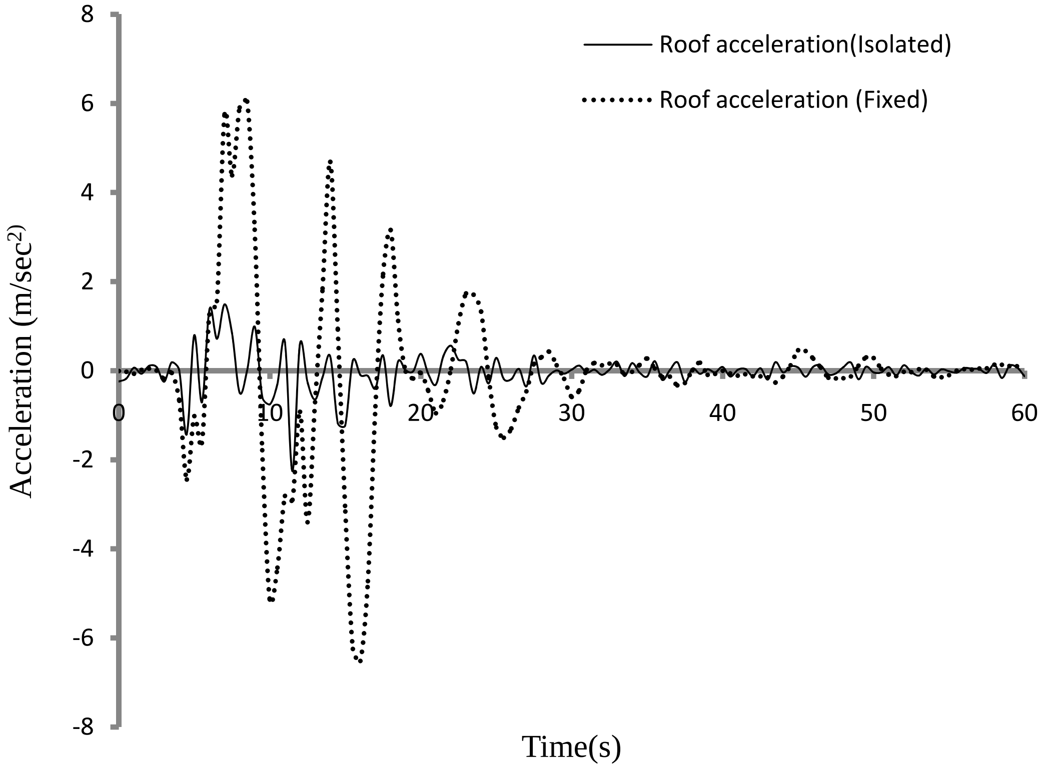

Fig. 30. Time history of acceleration response.

Table 9. Acceleration response of base-isolated framed structure as compared to its fixed base framed structure.

		Isolated	Fixed
Maximum	Acceleration (m/s^2)	1.48	6.04
	Time (s)	7.00	8.50
Reduction = 75.40 %			
Minimum	Acceleration (m/s^2)	-2.25	-6.52
	Time (s)	11.50	16.00
Reduction = 65.50 %			

4.6.11 Relative measure of Floor displacements with respect the base of the frame.

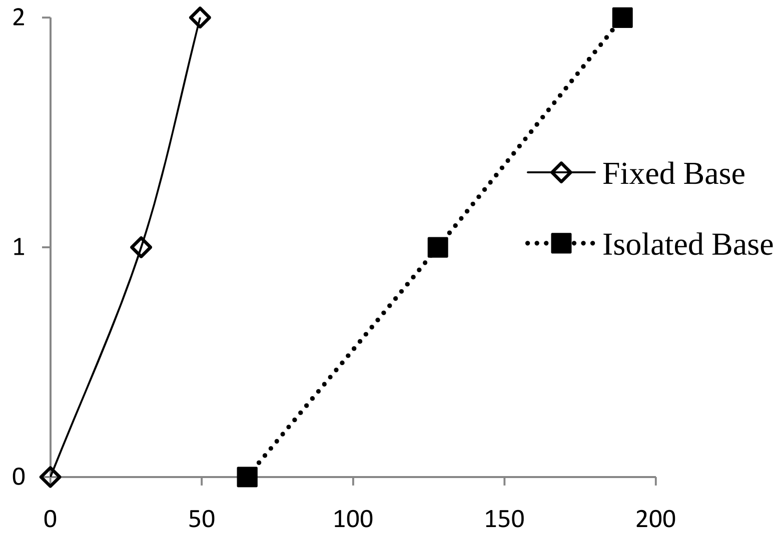

Fig. 32. Floor displacements with respect the base of the frame.

In case of isolated framed structure the displacement of the framed structure at each floor is more as compared to its corresponding fixed base one (Fig. 32.) with respect to the base of the frame. The increase in displacement is more uniform in isolated base as compared to the fixed base framed structure.

4.6.12 Relative measure of Floor displacements with respect the base of the frame.

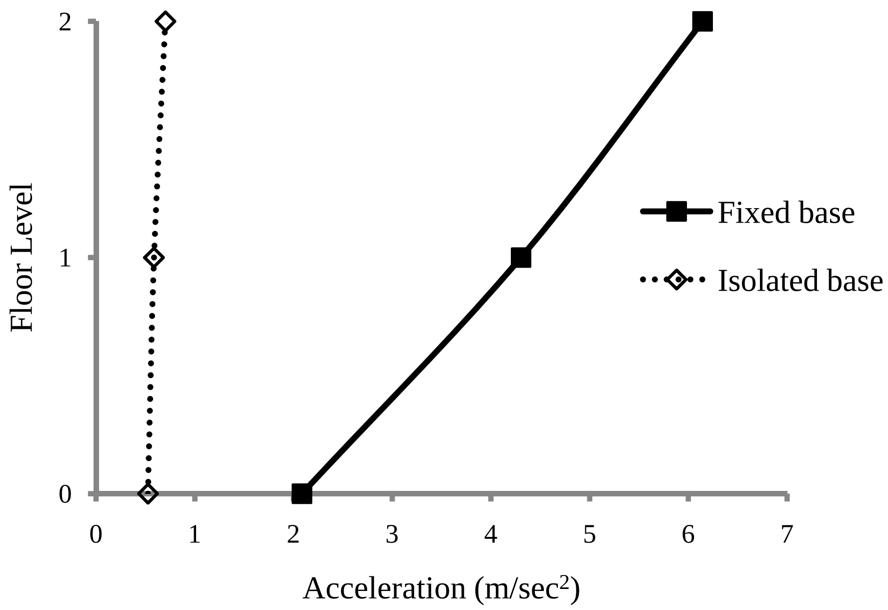

Fig. 33. Floor acceleration with respect to the ground of the frame (mm/sec^2).

In case of isolated framed structure the acceleration of the framed structure at each floor is less as compared to its corresponding fixed base one (Fig. 33.) with respect to the ground. The magnitude of acceleration imparted at each floor is approximately equal which signifies the rigidity of the superstructure above the isolator and the entire superstructure can be idealised as an S-DOF system. The increase in displacement is uniform in fixed base framed structure.

CHAPTER

5

SUMMARY AND CONCLUSION

5.1 SUMMARY

The investigation of dynamic properties of the framed structure, nonlinear response of framed structure under dynamic loading and effectiveness of base isolation of structure under dynamic loading are done and following conclusions achieved. This chapter first presents the modal analysis results of the benchmark problem. Then it discusses the free vibration analysis and time history analysis results of moment frame with fixed and isolated base subjected to Northridge Earthquake ground motion. The results show that the base isolation reduces the responses (displacement, velocity, acceleration, and inter-storey drift) drastically. Also, base isolation reduces the stiffness and thereby increases the fundamental period of the building to bring it out of the maximum spectral response region. Therefore it can be concluded from the results presented here that base isolation is very effective seismic control measures.

5.2 CONCLUSION

Modal analysis study: From the modal analysis study natural frequency and the mode shape of the framed structure is obtained. The determination of mode shape is essential to analyse the behaviour of the structure under applied dynamic loading. From the modal analysis of the Aluminium frame natural frequency, mode shapes and corresponding modal participating mass ratios are obtained. The mode shapes for which modal partiapting mass ratios are maximum taken into consideration. SAP 2000 is very effective tool to validate the results obtained experimentally. From the modal analysis first mode time period of fixed base building is found to be 0.56 sec whereas the first mode period of isolated building is found to be 3.11s (approximately 6 times the fixed-base period!). This value is away from the dominant spectral period range of design earthquake. Similar Shift was also observed in the higher modes, which shows the effectiveness of base isolation.

Time history analysis study: By conducting the nonlinear time history analysis it was shown that base isolation increases the flexibility at the base of the structure (Figs. 19 and 20), which helps in energy dissipation due to the horizontal component of the earthquake and hence superstructure's seismic demand drastically reduced as compared to the conventional fixed base structure. The lead core present at the centre increases the energy absorption capacity of the isolator (Fig. 3). The area of each cyclic loop represents the energy dissipated per cycle (Fig. 21). Here floor displacement and roof displacement response curves of the isolated structure are plotted which are equivalent and it indicates the rigidity of the superstructure above the isolator (Fig. 22). Base isolation reduces the base shear by 75-85% (Fig. 23) and reduces the velocity, acceleration response by 55-75% (Figs. 25 and 26). It also reduces interstory drift as compared to the conventional fixed base structure. It reduces the force imparted on the structure at each floor (Fig. 29) and the force imparted is equivalent at each floor as compared to the fixed base structure.

5.3 FUTURE SCOPE OF STUDY:

The vibration control technology is developing and its application is spreading in various fields of engineering structures. Factories, hospitals and residential houses will be protected from environmental vibration. It is evident that this technology will be progressed and become more important in the coming century.

In the present study natural frequency, mode shape, modal mass participating ratios of the structural model and nonlinear time history analysis was carried out to determine the behaviour of the structure under dynamic loading. Effectiveness of base isolation was studied by considering bilinear model of the LRB and modelling the same and superstructure by SAP 2000. The future scope of the present study can be extending as follows:

- Introduction of analysis software such as ETABS, SAP 2000 and LARSA help in explicit modelling of isolators which exhibit mildly nonlinear behaviour during dynamic loading.
- More research in earthquake time history records will help to study the behaviour of the structural model under a given loading.
- With recent advancement in material technology, more study can be focussed on material qualities used in isolators like their strength, durability, high vertical stiffness, low horizontal stiffness and high energy dissipating capacity.
- Development in testing methodology of the isolator to predict more accurate behaviour of the isolator under a given loading is of prime importance which needs a considerable attention in future.

A boom of base isolation study in Japan will be over soon, but more steady study and research will be continued for aiming earthquake free structures.

CHAPTER

6

REFERENCES

1. Izumi Masanory. Base Isolation and passive Seismic response control, Proceedings of Ninth World Conference on Earthquake Engineering, VIII, (1988): pp. 385-396
2. Garevski A Mihail. Analysis of 3-D vibrations of the Base Isolated School Building "Pestalozzi" by analytical and experimental approach, Proceedings of Ninth World Conference on Earthquake Engineering, 12 (2000): pp. 1-8.
4. Murty C.V.R.. Earthquake Tips-Learning Earthquake Design and Construction, Kanpur, National Information Centre for Earthquake Engineering, 2009
5. Constantinou M. C. *et al.* Non-linear dynamic analysis of multiple building base isolated structures, Computers and Structures, 50, (1994): pp. 47-57
6. Jain S K and Thakkar S K. Effect of Superstructure Stiffening in Base Isolated Tall Buildings, IE (I) Journal.CV, 85, (2004): pp. 142-148

7. Jangid R. S. and Kulkarni Jeevan A.. Rigid body response of base-isolated structures, Journal of Structural Control, 9, (2002): pp. 171–188
8. Naharajaiah Satish and Sun Xiahong. Seismic Performance of base Isolated Buildings in the 1994 Northridge Earthquake, Eleventh World Conference on Earthquake Engineering, 598, (1996): pp. 1-8
9. Hang *et al.* Multifunctional vibration-absorption RC mega frame structures and their seismic responses, Earthquake Engineering and Structural Dynamics , 29, (2000): pp. 1239-1248
10. Mazza Fabio and Vulcano Alfonso. Base-Isolation techniques for the seismic protection of RC Framed Structures subjected to near-fault ground motions, 13th World Conference on Earthquake Engineering, 2935, (2004): pp. 1-14
11. Palazzo B. and Petti L. Reduction factors for Base Bsolated structures, Computers & structures, 60, (1996): pp. 94S-956.
12. Dutta T.K and Jangid R.S. Seismic Reliability of Base Isolated Building Frames, Eleventh World Conference on Earthquake Engineering, 491, (1996): pp. 1-8
13. Mei C. In-plane Vibrations of Classical Planar Frame Structures – an Exact Wave-based Analytical Solution, Journal of Vibration and Control, 16(9), (2010): pp. 1265–1285
14. Nagarajaiah Et Al. Nonlinear Dynamic Analysis Of 3-D Base- Isolated Structures, Journal Of Structural Engineering, 117, (1991): pp. 2035-2054
15. Deb Sajal Kanti. Seismic base isolation – An overview, Current Science, 87, (2004): pp. 1426-1430
16. Kitagawa Yoshikazu et al. Experimental Study on Base Isolation Building using Lead Rubber Bearing through vibration tests, Ninth World Conference on Earthquake Engineering, V, (1988): pp. 711-716

17. Shenton H. W. and Lin A. N. Relative performance of Fixed-base and Base-isolated concrete frames, Journal of Structural Engineering, 119, (1993),: pp. 2952-2968
18. Luciano M. Bezerra and Rodolfo C. Carneiro. A Numerical Evaluation of Anti-Vibration Mechanisms Applied to Frame Structures Under Earthquake, 17th International Conference on Structural Mechanics in Reactor Technology (SMIRT 17), 13-2, (2003): pp. 1-8
19. Kang Beom-Soo *et al*. Dynamic response characteristics of seismic isolation systems for building structures, Journal of Mechanical Science and Technology, 23 (2009): pp. 2179-2192
20. Ibrahim R.A. Recent advances in nonlinear passive vibration isolators, Journal of Sound and Vibration, 314 (2008): pp. 371–452
21. Providakis C.P. Effect of LRB isolators and supplemental viscous dampers on seismic isolated buildings under near-fault excitations, 30 (2008) : pp. 1187–1198
22. Aiken Ian D. Observed behaviour of Seismically Isolated buildings, Journal of Structural Engineering, 125 (1999): pp. 955-964.
23. Hasebe Akiyoshi. Design experience of a Base-isolation system applied to a Computer Centre Building, Nuclear Engineering and Design, 127 (1991): pp. 339-347
24. Hang Dai. Multifunctional vibration-absorption RC mega frame structures and their seismic responses, Earthquake Engineering and Structural Dynamics, 29 (2000): pp. 1239-1248.
25. Chang Y. F. and Mo Y. L. Application of Base Isolation Concept to Soft First Story Buildings, Computers & Structures, 55 (1995): pp. 883-896.
26. Jangid R. S. Stochastic response of building frames isolated by lead–rubber bearings, Structural control and Health Monitoring, 17 (2010):pp. 1–22

27. Dolce M. and Telesca F.R. The design of R/C framed building with nonlinear isolation systems, 7th international seminar on Seismic isolation, Passive Energy Dissipation and Active Control of Vibrations of Structures, Assisi, Italy, (2001): pp. 1-9

28. Jangid R. S. Seismic Response of an Asymmetric Base Isolated Structure, Computers & Structures, 60 (1996): pp. 261-267

29. Nagarajaiah Satish and Sun Xiahong. Seismic Performance of Base Isolated buildings in 1994 Northridge earthquake, Eleventh World Conference on Earthquake Engineering, 598, (1996): pp. 1-8

30. Hur M.W. *et al.* Evaluation of dynamic characteristics of Base Isolated Residential building, The 14th World Conference on Earthquake Engineering, (2008): pp. 1-9

31. Rao Bhaskar P. and Jangid R.S. Experimental Study on Base isolated structures, ISET journal of earthquake Technology, 38 (2001): pp. 1-15

32. Hasebe Akiyoshi *et al*. Design experience of a Base-isolation system applied to a Computer Centre Building, Nuclear Engineering and Design, 127 (1991): pp. 339-347

33. Tena-Colunga Arturo and Villegas-Jiménez Omar. Dynamic design procedure for the design of Base Isolated Structures located on The Mexican Pacific Coast, The 12th World Conference on Earthquake Engineering, 0929 (2000): pp. 1-8

34. Constantinou M. C. Nonlinear dynamic analysis of multiple building Base isolated Structures, Computers & Structures, 50 (1994): pp. 47-57.

35. Ferrell Keith and Nagarajaiah Satish. Stability of elastomeric seismic Isolation Bearings, Journal of Structural Engineering, 125 (1999): pp. 946-954

36. Nagarajaiah Satish and Xiaohong Sun. Base-Isolated FCC Building: Impact response in Northridge Earthquake, Journal of Structural Engineering, 127 (2001): pp. 1063-1074

37. Nagarajaiah Satish and Xiaohong Sun. Response of base-isolated USC hospital building in Northridge Earthquake, Journal of Structural Engineering, 126 (2000): pp. 1177-1186

38. Aiken Ian D. *et al.* Observed behaviour of Seismically Isolated Buildings, Journal of Structural Engineering, 125 (1999): pp. 955-964

39. Mazza Fabio and Vulcano Alfonso. Nonlinear Response of RC Framed Buildings with Isolation and Supplemental Damping at the Base Subjected to Near-Fault Earthquakes, Journal of Earthquake Engineering,, 13(2009): pp. 690–715

40. Nagarajaiah Satish *et al.* Structural control benchmark problem: Phase II—Nonlinear smart base-isolated building subjected to near-fault earthquakes, Structural Control and Health Monitoring, 15 (2008): pp. 653–656

41. Gavin *et al.* Smart Base Isolated Building benchmark problem, 16th ASCE Engineering Mechanics Conference, (2003): pp. 1–12

42. Erkus Baris and Johnson Erik A. Smart base-isolated benchmark building Part III: A sample controller for bilinear isolation, Structural Control and Health Monitoring, 13 (2006): pp. 605–625

43. Nagarajaiah S. *et al.* Smart base-isolated benchmark building. Part I:problem definition, Structural Control and Health Monitoring, 13 (2006): pp. 573–588

44. Agrawal Anil K. Development of experimental benchmark problems for international collaboration in structural response control, Bridge Maintenance, Safety, Management, Health Monitoring and Informatics – Koh & Frangopol (eds) , 13 (2008): pp. 3298–3305

45. Jain S K and Thakkar S K. Effect of Superstructure Stiffening in Base Isolated Tall Buildings, IE (I) Journal.CV, 85 (2004): pp. 142–148

46. Jangid R. S. and Kulkarni Jeevan A. Rigid body response of base-isolated structures, Journal of Structural Control, 9 (2002): pp. 171–188

47. Chang Y. F. and Mo Y. L. Application of Base Isolation Concept to Soft First Story Buildings, Computers & Structures, 55 (1995): pp. 883-896.

48. Datta T. K. and Jangid R. S. Performance of Base Isolation systems for Asymmetric building subject to random excitation, Engineering Structures, 17 (1995) pp. 443–454

49. Jangid R. S. Seismic Response of an Asymmetric Base Isolated Structure, Computers & Structures, 60 (1996): pp. 261-267.

50. Tena-colunga Arturo and Zambrana-Rojas Christian. Torsional response of Base-Isolated Structures due to Stiffness Asymmetries of the Isolation System, 13th World Conference on Earthquake Engineering, 2022 (2004) : pp. 1-15

51. Nagarajaiah Satish et al. Torsion in Base-Isolated Structures with Elastomeric Isolation Systems, Journal of Structural Engineering, 119 (1993) : pp. 2932-2951

52. Tena-Colunga A. and Go´mez-Sobero´n L. Torsional response of base-isolated structures due to asymmetries in the superstructure, Engineering Structures, 24 (2002) : pp. 1587–1599

53. Hong Jang-Mi. Vertical distribution of equivalent static loads for base isolated building structures, Engineering Structures 23 (2001): pp. 1293–1306

54. Tena-Colunga Arturo, Escamilla-Cruz Jos´e Luis, Torsional amplifications in asymmetric base-isolated structures, Engineering Structures, 29 (2007): pp. 237–247

55. Kilar Vojko and Koren David. Seismic behaviour of asymmetric base isolated structures with various distributions of isolators, Engineering Structures, 31 (2009) : pp.910-921.

56. Kelly *et al.* Uasge of seismic base isolation to reduce the ductility demand from prefabricated concrete structure, Holmes Consulting group limited.

57. Jangid R.S. and Matsagar Vasant A. Seismic response of base-isolated structures during impact with adjacent structures, Department of Civil Engineering, Indian Institute of Technology Bombay, 1311-1323

58. Nagarajaiah Satish. Stability of Elastomeric Isolation Bearings: Experimental Study, Journal of Structural Engineering, 128 (2002): pp. 3-11

59. Nagarajaiah Satish and Buckle Ian. Stability of Elastomeric Isolators: Critical Load Tests and Computations, MCEER Bulletin, 16 (2002) : pp. 1-4

60. Abrams D.P, Shaking table study of Base isolation for masonry buildings, Journal of Structural Engineering, 117 (1991) : pp. 3315-3336

61. Kareem Ashan et al. Migration of motion of Tall buildings with specific examples of recent applications, Wind and structures, 2 (1999): pp. 201-25

62. Agrawal M.K. et al. Studies on response of structures, equipment and piping systems to earthquake and its mitigation, BARC news letter, reactor safety division, 290 (2008): pp. 15-30

63. Sorace Stefano and Terenzi Gloria., Analysis and Demonstrative Application of a Base Isolation/Supplemental Damping Technology, Earthquake Spectra, 24 (2008): pp. 775–793

64. Aiken I.D et al. Elastomeric bearings for 3-dimensional seismic isolation, proceedings 1990 ASME PVP Conference, 200 (1990): pp. 1-7

65. Aiken Ian D., Testing of Passive Energy Dissipation Systems, Earthquake Spectra, Earthquake Engineering research institute California, 9 (1993): pp. 1-36

66. Aiken I.D et al. Performance considerations for isolation systems in the region of high seismicity, Proceedings, ATC-17-1, Seminar on seismic isolation, passive energy

dissipation and active control, applied technology Council, San Francisco, California, 17-1 (1991): pp. 1-12

67. Aiken Ian., Passive energy dissipation- Hardware and applications, Proceeding Las angles county and SEAOSC symposium on passive energy dissipation systems for new and existing buildings, (1996): pp. 1-24
68. Aiken I.D et al. An analytical hysteresis model for elastomeric seismic isolation bearings, Earthquake engineering and Structural engineering, 26 (1997): pp. 1-17
69. Aiken Ian. D., Testing of seismic isolators and dampers-considerations and limitations, proceedings, structural engineering world congress, San francisco, California, (1998): pp. 1-8
70. Aiken Ian. D., The 1995 Kobe (Hyogo-ken Nanbu) Earthquake as a Trigger for Implementing New Seismic Design Technologies in Japan, Earthquake Engineering Research Institute, III (1999): pp. 1-38
71. Chopra Anil K. Dynamics of Structures, Theory and Applications to Earthquake Engineering. PHI publications, New Delhi, 2004.
72. Naeim F. and Kelly, J. M., Design of Seismic Isolated Structures, John Wiley, New York, 1999.

CHAPTER

7

APPENDIX-I: NOTATION

Δ = designed displacement of the isolator.

$\Delta f(t)$ = individual time-independent force pulses.

Δt = Time increments.

$\emptyset_n$ = The deflected shape.

Δ_y = Yielded displacement of the isolator.

A_n and B_n = Constants of integration.

A_{pb} = Area of the lead core.

A_r = Cross-sectional area of the rubber layers.

C_{eff} = effective damping coefficient.

$D_y = \frac{Q}{K_1 - K_2}$ = Yielded displacement of the material.

f_l = Factor given by 1.15.

F_m = Force at designed displacement.

f_y = The yield force of the isolator.

K_1 = Elastic stiffness of the isolation bearing material.

K_2 = Yielded stiffness of the isolation bearing material.

K_{eff} = Effective stiffness of the isolation bearing material.

t_r = Total thickness of the rubber consisting of n-layers.

W_D = Area of the hysteresis loop i. e. (the energy dissipated per cycle).

w_n = Natural frequency for n^{th} number of mode.

β_{eff} = Effective damping of the isolation bearing material .

ζ_{eff} = effective damping ratio.

σ_{ypb}= The yield strength of the lead core (Ranging between 7.0 and 8.5 MPa).

Ω^2 = Spectral matrix of the eigenvalue problem.

$\mu = {}^{Q}/_{W}$ = Characteristic strength Q over the total weight on the isolation system W.

D = The isolator diameter D.

g = Acceleration due to gravity.

G = Shear modulus of the rubber.

k = Stiffness matrix of the structure.

K_v = Elastic stiffness of the isolation bearing material.

m = Mass matrix of the structure.

Q = Characteristic strength. (Force intercept at zero displacement).

Q = Characteristics Strength of the isolation bearing material.

T^{iso} = The fundamental isolation period.

u = Displacement of the structure.

W = The total weight on the isolation system.

y $= D/D_y$ = Non-dimensional displacement.

ω = Natural frequency of the isolation bearing material.

APPENDIX-II: ABBERATION

2-D	Two Dimensional
3-D	Three Dimensional
ASCE	American Society of Civil Engineers
CPS	Cyclic per second.
DBE	Design basis Earthquake
EC	Euro Code
EDC	Energy Dissipated per Cycle
EERC	Earthquake Engineering Research Centre.
FCC	Fire Command and Control
FF	Far-field
FREI	Fiber-reinforced elastomeric isolator
HDRB	High-damping rubber bearings
IIT	Indian Institute of Technology.
IS	Indian Standard
ISET	Indian Society of Earthquake Technology
kN	Kilo Newton
LQG	Linear quadratic Gaussian
LRB	lead–rubber bearings
MCE	Maximum Credible Earthquake
MCEER	Multidisciplinary Centre for Earthquake Engineering Research
MDOF	Multi Degree of Freedom
N-DOF	N-Degree of Freedom system.
NF	Near-field

NICEE	National Information Centre for Earthquake Engineering.
NISEE	National information service for earthquake engineering
NIT	National Institute of Technology.
PEER	Pacific Earthquake Engineering Research
SAP	Structural Analysis Program
SDOF	Single Degree of Freedom
SREI	Steel-reinforced elastomeric isolator
UBC	Uniform Building Code
USA	United States of America.
USC	University of Southern California
WCEE	World Conference on Earthquake Engineering

APPENDIX-III: LAMINATED RUBBER BEARING

- The vertical stiffness of the laminated rubber bearing is given by,

$$K_v = \frac{E_c A}{t_r}$$

Where,

E_c is the instantenious compression modulus of the steel rubber composites.

For a building square in plan instantaneous compression modulus (Kelly, 1997) is given as

$$Ec = 6.73\, S^2 G$$

Where S is the Shape factor (Ratio of the loaded area to the force free area of the rubber layer).

Where,

And G is the shear modulus of the bearing which is typically dependent upon the rubber hardness.

- The horizontal stiffness of the laminated rubber bearing is given by,

$$K_h = \frac{GA}{t_r}$$

- Several Experimental tests were performed to obtain their static and dynamic properties. These tests include
  i. The hardness test. (For evaluation of shear modulus)
  ii. The force-deformation behaviour in vertical and horizontal directions.
  iii. The free vibration test for measuring the damping

APPENDIX-IV: SOME IMPORTANT BASE ISOLATED BUILDINGS

INDIA

- The four-storey Bhuj Hospital building was built with base isolation technique.
- Two single storey buildings (one school building and another shopping complex building) in newly relocated Killari town.

USA

- Utah State Capitol: Salt Lake City
- Foothill Communities Law & Justice Centre: County of San Bernardino, California.
- Los Angeles County Fire Command & Control Facility, California
- Los Angeles County Emergency Operations Center, California
- Caltrans Traffic Management Center, Kearney Mesa, California
- Oakland City Hall, California (retrofit)
- San Francisco City Hall, California (retrofit)
- Los Angeles City Hall, California (retrofit)

JAPAN

- The Funabasi fire ststion.
- High city kyosumi
- MM21 building.
- Takasu and Yazawa Hospitals.
- Asian Art Museum.
- F-Museum.

New Zealand

- New Zealand Parliament Building

APPENDIX-V: TYPES OF VIBRATION CONTROL

- Vibration control is the mechanism to mitigate vibrations by reducing the mechanical interaction between the vibration source and the structure, equipment etc. to be protected.
- vibration isolation approach

❑ Low frequency tuning.

❑ High frequency tuning.

These are of following types;

1. Passive vibration control.
2. Active vibration control.
3. Semi-active vibration control.
4. Hybrid vibration control.