NUMERICAL STUDY OF BASE ISOLATED BUILDING FRAMES FOR EFFECTIVE SEISMIC RETROFIT

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This foregoing thesis is hereby approved as a credible study of an engineering subject carried out and presented in a manner satisfactory to warrant its acceptance as a prerequisite to the degree for which it has been submitted. It is understood that by this approval the undersigned do not endorse or approve any statement made, opinion expressed or conclusion drawn therein but approve the thesis only for the purpose for which it has been submitted.

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Declaration of Originality and Compliance of Academic Ethics

I hereby declare that this thesis contains literature survey and original research work by the undersigned candidate, as part of his MASTER OF CONSTRUCTION ENGINEERING studies.

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NUMERICAL STUDY OF BASE ISOLATED BUILDING FRAMES FOR EFFECTIVE SEISMIC RETROFIT

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ABSTRACT

Seismic hazards are the most devastating hazard in the world. It causes severe damage to the structure and a significant loss of life in the world. A flexible structure largely decoupling the structure from the ground motion and the structural response acceleration are usually less than the ground acceleration. As a result base isolation limits the effects of earthquake attacks. The application of base isolation techniques to protect structures from severe seismic activity is considered as the most effective approaches and it is widely accepted. So, base isolation system is gaining popularity as an effective method of seismic retrofit.

In this thesis paper, a series of non-linear dynamic analyses are carried out to study using SAP2000. Time history analyses are performed on 7-story multistoried building with different ground motion obtained from recorded earthquake data. At the same time an investigation of the seismic response of asymmetrical 4 storied building has also been carried out. For different ground acceleration responses, these two building frames have been analyzed with base isolation system.

In the present work an attempt has been made to study and compare the non-linear behavior of the multistoried building frame (Symmetric and Asymmetric) with conventional design (Fixed base) and with a base isolation system. The time history analysis, has been carried out with three earthquake data such as from previous earthquakes corresponding to Park Field (1966), Imperial (1938) and Sikkim (2011). Study focuses to reduction of the acceleration, displacement and also reduction of the inter-story drift of the base isolated structure with respect to fixed base structure.

Chapter I

INTRODUCTION

1.1 General

An Earthquake is one of the most devastating natural disasters that can cause great loss of life and property. Earthquake is the sudden release of accumulated energy in the tectonic plates of the earth crust and resulting in propagation of seismic waves; P waves, S waves and surface waves. Earthquake occurs at the faults at boundaries the tectonic plates, causing colliding, separation, sliding, or sub ducting between the adjacent plates. Epicenter is the ground surface point that intersects vertically in a line to the depth of the hypocenter, which considered being a significant factor of seismic hazard. Actual ground acceleration records during earthquake, obtained near and away from the sources, the ground motion consist of positive and negative peaks in three mutually perpendicular directions of varying amplitude and time intervals depending upon the earthquake magnitude and the type of source. The motion is intense over a short time duration which is the main cause of damage of structures and their contents, structural as well as non-structural. The consequences of earthquake events are well known to the public: thousands of persons are killed or injured each year, thousands are homeless, heavy damage to the building stock, complete disruption of the infrastructure, irreversible damage to the cultural heritage, very large indirect costs resulting from business interruption, loss of revenues, and interruption of industrial production. Recent Earthquakes have clearly demonstrated that the houses, bridges, public buildings constructed in many third world countries are not engineered to resist even moderate earthquakes. Recently in India, earthquakes caused huge economic losses and death toll, however not much attention is given in preventing such structural damages caused by earthquakes.

1.1.1. Traditional design approach

In traditional seismic design methods, the anti-seismic resistance of the structure is ensured by way of increasing the strength and stiffness of structural members. Our approach for seismic design at present is to ‘confront’ brute forces as generated by the earthquake shaking. That is however, these methods have not met the demands of modern structures.

1.1.2. Requirement of Base isolation for seismic vulnerability

Traditional design methods based only on strength have been gradually replaced by new theories and design methods in which seismic base isolation is recognized.

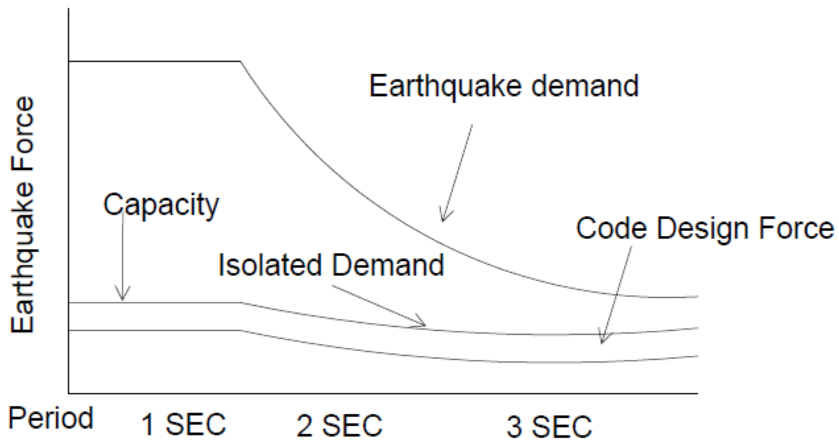

Fig. 1.1 : Design Principles of seismic base isolation

From the figure, we can easily understand that the base isolation system reduced the seismic force under the design maximum force. Base isolation is also an effective up gradation strategy by increasing the performance of the structure. Base isolation can upgrade the performance of an old existing structure to a required seismic performance level.Seismic isolation systems have been shown to not only reduce the response of the primary structure, but to also reduce damage to equipment and other non-structural secondary elements.

The concept of base isolation systems is quite simple by interposing structural elements with low horizontal stiffness between the structure and the foundation to decouple the structure from the horizontal components of the ground motion which gives the structure a very low frequency than both its fixed base and the ground motion. The deformation of first dynamic mode happens in base isolation while the structure above is rigid; the deformation of higher modes happens in the structure which is orthogonal to the first mode and to the ground motion, these higher modes do not participate in the motion; therefore, the high energy in the ground motion cannot be transmitted to the structure. The base isolation system does not absorb the energy from the earthquakes but it deflects it through the system dynamics which is not depending on the damping level, but dampers are important to suppress resonance at the isolation frequency.

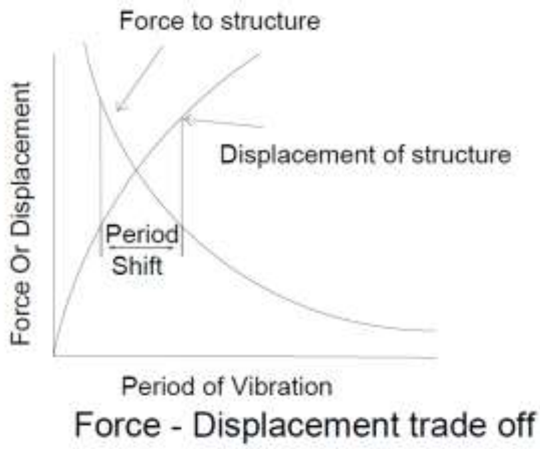

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Fig. 1.2: Seismic base isolation force-displacement trade off

The mechanism of the base isolator increases the natural period of the overall structure, and decreases its acceleration response to earthquake / seismic motion. The structural deformations going into the inelastic/plastic range and the consequent damage is likely to be completely eliminated. The structure will need designing for much smaller accelerations, hence should be more economical. The relative story displacements (drift) will be reduced hence the ‘non-structural’ damage to cladding, partition walls etc. will be minimised or eliminated altogether.

Inertial forces are reduced by lengthening the fundamental period of vibration and added damping through the introduction of elements
(Isolators)with horizontal and vertical stiffnessthat decouple the superstructure from the supporting substructure.Base isolation is incorporated to the structure to introduce flexibility at the supports of a structure in the horizontal plane so as to ensure that the time period of the structure is well above the predominant periods of the probable earthquake. Now in this process, the relative displacement amplitude increases, hence often damping or restraining elements have also to be introduced simultaneously to restrict the extent of relative movement caused by the earthquake. Although this principle is not new, its practical exploration has occurred only recently in the last about 15 years during which suitable hardware of

isolating devices has been developed and actually applied to some constructions of buildings, bridges and atomic power plants.

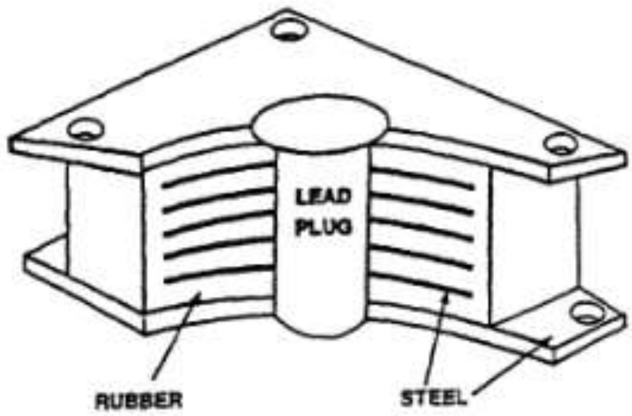


Fig. 1.3: Typical square lead-rubber bearing

This type of lead rubber bearing is used to dissipate the energy generated from earthquake. These rubber bearings are constructed in layers by sandwiching steel shims between each layer and bonding together by gluing. While the lateral stiffness and damping of the LDR bearing are provided by shearing of rubber, steel shims are inserted between the rubber layers to minimize lateral bulging of rubber due to high pressure from the vertical load. Other possible devices for introducing flexibility include spherical ends, cable suspensions, pinned or soft story columns, sleeved piles and sliding or rocking arrangement. Seismic isolation system also consists of active control devices and semi-active control devices. Active, semi-active control system is simulated in numerical model to study the various type of base isolation system.

1.2 Objective of the present study

The primary objective of the present study is to investigate the change in seismic response of symmetric and asymmetric building frame due to incorporation of base isolation system. The non-linear time history analysis is performed with three earthquake ground motion recorded from past earthquake data. The values obtained through non-linear dynamic time history analysis of different building frames with fixed and isolated base are compared to study the application of base isolation system for effective seismic retrofit as well as for new building structures.

1.3 Scope of Work

- Numerically investigate the effect of base isolation system over a building frame.
- Comparative study of symmetric building with fixed base and base isolated stimulate for different real-time earthquake ground acceleration.
- Comparative study of unsymmetrical building frame with fixed base and base isolated stimulate for different real-time earthquake ground acceleration.
- Comparison of the effect of base isolation for symmetric and asymmetric plan.

1.4 Outline of the thesis

The entire work is presented in seven chapters.Traditional seismic design approach, requirement of base isolation for seismic retrofit and the specific objectives of the present work and scope of the work have been discussed in chapter one followed by the review of related literatures in chapter two. Chapter three deals with the detail discussion on base isolation and different numerical approach. Chapter four describes the non-linear time history analysis and type of method adopted for this present study. Chapter five and six presents the details of numerical study on the fixed base and base isolated structure and the results obtained in different structure with different type of earthquake data. Finally chapter seven draws the significant conclusions based on the present study and identify important ways which may be directed towards the future development of the study.

Chapter II

Literature Review

2.1 General

During the last few decades the developments on the analysis of base isolation system are notable and the related important works are briefly discussed here.

C.E Ventura, W.D Liam Finn, J.-F Lord, N Fujita conducted ambient vibration tests on a base-isolated building in Takamatsu, Japan, to determine its dynamic response characteristics under very low levels of excitation. The natural frequencies, modal damping and mode shapes of the building were determined in the longitudinal, transverse and torsional directions in order to provide data for the calibration of a finite element model of the building in its initial state before the onset of strong shaking. The finite element model, calibrated by ambient vibration data and verified for a low level of earthquake shaking, provides the starting point for modelling the non-linear response of the building when subjected to strong shaking.

Gordon P. Warn1 and Andrew S. Whittaker, (2008) studied on the influence of vertical earthquake excitation on the response of a bridge isolated with low-damping rubber and lead–rubber bearings through earthquake simulation testing. Response data collected from the experimental program are used to determine the vertical load on the isolation system due to the vertical component of excitation. A comparison of the normalized vertical load data to the vertical base acceleration showed significant amplification of the vertical response for each simulation and configuration.Sample axial load histories from individual bearings showed the frequency of the vertical component to be significantly higher than that of the overturning component as was expected. However, the sum of the maximum absolute value of the two components tends to overestimate the maximum absolute value of the combined overturning plus vertical axial load history for allsimulations performed in this study. The spectral analysis procedure considering the full vertical stiffness of the isolator leads to reasonably accurate estimates of the vertical earthquake load on the isolation system for this bridge model and isolation systems.

However, for a hybrid isolation system such as those composed of flat sliding and elastomeric the reduction in vertical stiffness should be considered on a case-by-case basis using more advancedanalysis techniques.

N. Wongprasert, and M. D. Symans, 2005, studiedthe seismic response of a scale-model, base-isolated, multi-story structure which is numerically investigated. One approach to improving the performance of an isolation system that is subjected to disparate earthquake ground motions is to incorporate a device within the isolation system whose properties can be adjusted in real-time during an earthquake. Numerical simulations are performed to evaluate the dynamic response of the isolated test structure when different damping mechanisms (passive, semi-active, or active) are incorporated within the isolation system. In this study, the seismic response of a scale-model, baseisolated, three-story building frame is evaluated numerically. Two types of isolation systems are used in the analysis; one employing friction pendulum system (FPS) bearings and the other employing low-damping rubber (LDR) bearings. A semi-active variable orifice fluid damper is utilized between the foundation and the basement of the superstructure to provide additional damping to the isolation system. The amount of damping was selected in real-time based on an *H*` optimal control design using full-state feedback.

The dynamic properties of the fixed-base (non-isolated) 1:4-scale, three-story building frame were determined experimentally via system identification testing. The natural periods and damping ratios of the first, second, and third modes of the three-story structure are 0.345, 0.104, and 0.057 s and 1.2, 0.8, and 0.5%, respectively. Note that since the three-story structure is a welded steel moment resisting frame, the damping ratios are relatively small.

Table 1. Properties of Friction Pendulum System (FPS) and Low-Damping Rubber (LDR) Base Isolation Systems (Model Scale)

	$k_{b,\mathrm{eff}}$ (kN/m)	$c_{b,\mathrm{eff}}$ (kN s/m)	D_y (mm)	F_y (kN)	γ_b	β	θ	η	α
FPS	82.8	0	0.13	N/A	0.9	0.10	1	2	N/A
LDR	350.2	3.44	0.86	1.78	1.5	−0.54	1	4	0.17

Note: N/A=not applicable.

Table. 2.1 : Properties of FPS & LDR Base Isolation Systems (ref.-N. Wongprasert, and M. D. Symans, 2005)

It has been shown that the response of the three-story base isolated structure can be significantly reduced by using an active control system with the proposed feedback control design for both types of isolation bearings. For the FPS bearings, the active control system provides significant response reduction of the superstructure by sacrificing large bearing deformations (although the bearing deformation was always kept below the allowable limit). The maximum force produced by the active control system was about 15% of the total weight of the structure, which, although feasible for actual implementation, may be cost prohibitive due to excessive power requirements. Note that this maximum force is approximately equal to the bearing friction force. This is not surprising, given the relatively high maximum sliding coefficient of friction (15%) and thus the limited amount of restoring force within the isolation system.

Prayag J. Sayaniand Keri L. Ryan compared the relative performance of two systems of fixed base and isolated base building to achieve a given performance objective. When performance-based engineering matures, designers will be able to employ the latest design and analysis techniques to create efficient designs that meet specified performance objectives, and building owners will be able to comparatively evaluatebase isolation and fixed-base design with reference to a quantitative performance objective. When evaluated for a life safety performance objective, the superstructure design base shear of an isolated building is competitive with that of a fixed-base building with identical ductility, and the isolated building generally has improved response. Isolated buildings can meet a moderate ductility immediate-occupancy objective at low design strengths, whereas comparable ductility fixed-base buildings fail to meet the objective.When performance-based engineering matures, designers will be able to employ the latest design and analysis techniques to create efficient designs that meet specified performance objectives, and building owners will be able to comparatively evaluate base isolation and fixed-base design with reference to a quantitative performance objective. When evaluated for a life safety performance objective, the superstructure design base shear of an isolated building is competitive with that of a fixed-base building with identical ductility, and the isolated building generally has improved response. Isolated buildings can meet a moderate ductility immediate-occupancy objective at low design strengths, whereas comparable ductility fixed-base buildings fail to meet the objective. The reference fixed-base buildings were designed to code standards for fixed-base buildings, while the isolated buildings were designed to 100, 50, and 25% of code base shear for isolated buildings.

The seismic performance objectives implicit in United States building codes currently differ for fixed-base and base-isolated buildings. As an example, fixed-base buildings are permitted afforce reduction factor *R* of up to eight, which may allow significant inelastic action in the design basis earthquake and can be interpreted as a “life safety” performance objective. Likewise, isolated buildings are limited to *R* factors no larger than two, and remain essentially elastic due to over strength. The reduced *R* factor together with other requirements may be interpreted as seeking performance objective more comparable to “immediate occupancy” or “operational”. Isolated buildings can meet a moderate ductility immediate-occupancy objective at low design strengths, whereas comparable ductility fixed-base buildings fail to meet the objective.

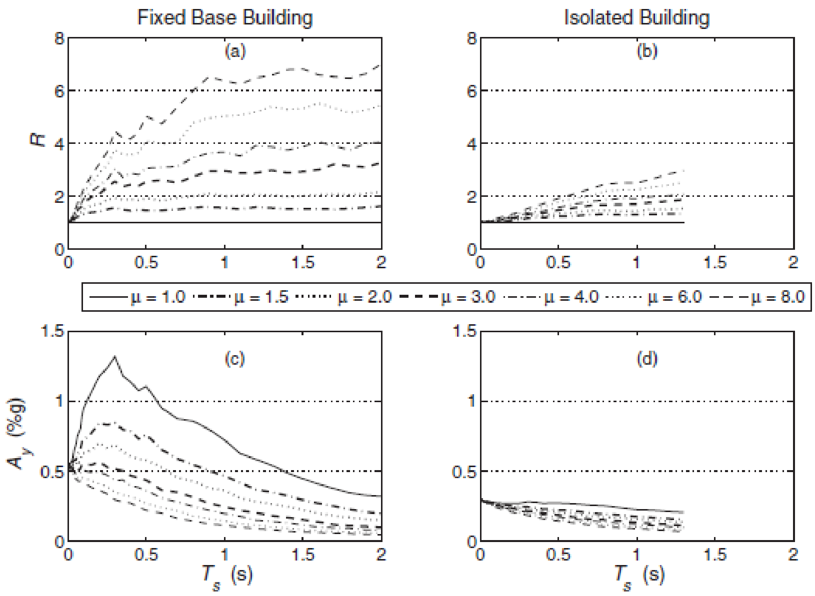


Fig. 2.1 :Constant ductility spectra for (a)–(b) force reduction factor *R* and (c)–(d) yield acceleration spectra A_y. Spectra are shown for fixed base buildings and base-isolated buildings with T_{shift}=2 and η=0.4. (Ref. Prayag J. Sayaniand Keri L. Ryan)

Radhikesh P. Nanda1; Manish Shrikhande; and Pankaj Agarwal, examined on low-cost friction base-isolation system in reducing seismic vulnerability of rural buildings. Four friction isolation interfaces, namely, marble–marble, marble–high-density polyethylene, marble–rubber sheet, and marble–geo-synthetic, were studied. The effectiveness of these isolation systems was investigated both analytically and experimentally for a spectrum compatible ground motion corresponding to the maximum credible earthquake for the most severe earthquake zone according to Indian standards for earthquake-resistant design.

Table- 2.2: Coefficient of Friction for Different Sliding Interfaces (Ref. Radhikesh P. Nanda1; Manish Shrikhande; and Pankaj Agarwal)

Sl. No.	Interface	Coefficient of static friction	Coefficient of dynamic friction
1	Marble–marble	0.09	0.08
2	Marble–HDPE	0.08	0.07
3	Marble–rubber	0.16	0.18
4	Marble–geo-synthetic	0.11	0.10

Shake Table Test - The performance of these sliding interfaces in reducing the seismic response of a half-scale single-story brick masonry building during earthquakes was investigated on a 3:5 × 3:5 m biaxial servo controlled shake table. An artificial accelerogram compatible with the design spectrum of Indian standard (IS) 1893 (Part 1) (2002)and corresponding to the level of maximum considered earthquake in the most severe seismic zone in India (with effective peakground acceleration of 0.36 g) was used as the base excitation inthe horizontal direction. The vertical motion was considered as two-thirds of the horizontal motion.

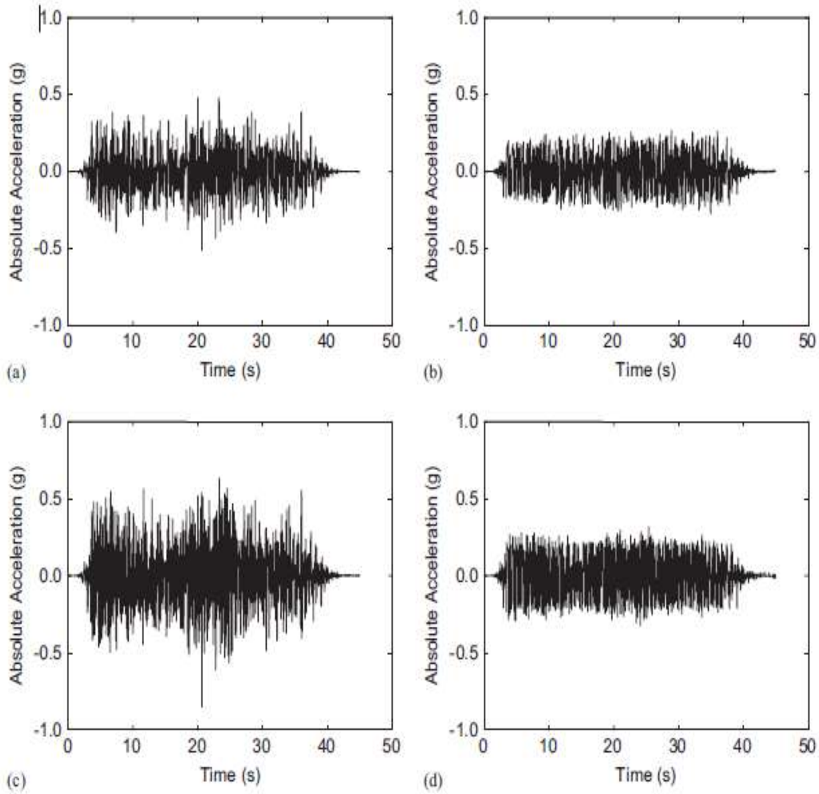


Fig. 2.2: Table motion and absolute acceleration response at roof level for marble–marble sliding and fixed base structure: (a) table motion, horizontal component; (b) experimental roof acceleration response of sliding model; (c) analytical roof acceleration response of fixed base model; (d) analytical roof acceleration response of sliding model(Ref. Radhikesh P. Nanda1; Manish Shrikhande; and Pankaj Agarwal)

Table 2.3 : Comparison of the Absolute Acceleration Amplification at Roof Level(Ref. Radhikesh P. Nanda1; Manish Shrikhande; and Pankaj Agarwal)

Interfaces	Maximum horizontal table acceleration (in g)	Fixed-base response (in g) (analytical)	Sliding base response	
			Analytical	Experimental
Marble–marble	0.51	0.86	0.32	0.27
Marble–HDPE	0.58	1.0	0.28	0.24
Marble–rubber	0.47	0.76	0.46	0.41
Marble–geo-synthetic	0.48	0.82	0.38	0.29

The friction test reveals that sliding interfaces made of marble–marble, marble–HDPE, and marble–geo-synthetic exhibit coefficient of friction values in the desirable range, i.e., 0.05–0.15. The shake table tests confirm the reduction of roof acceleration due to the friction base-isolation system, and the analytical predictions are in good agreement (within 19%) with the experimental observations.

The low-friction sliding material allows the superstructure to slide with equal effectiveness to previously recommended materials such as graphite; screened gravel; dune sand; Teflon–steel; clay; fine sand–terrazzo plates, the use of which has been restricted because of their high cost, construction complications, and poor durability. The proposed sliding couples marble–marble and marble–geo-synthetic can be easily bonded with building materials and can be easily used by the rural population for limiting the earthquake energy transmission to superstructures during strong earthquakes, which leads to a low-cost durable solution for earthquake protection of masonry buildings.

TakehikoAsai, Chia-Ming Chang, and B. F. Spencer is studied on experimentally investigating and verifying a smart base isolation system using real-time hybrid simulation (RTHS), which provides a cost-effective means to conduct such experiments because only the portion of the structure that is poorly understood needs to be represented experimentally, while the reminder of the structure can be modelled using a computer. Passive base isolation is one of the most widely accepted and deployed seismic protective systems. These systems are typically composed of low-stiffness devices (e.g., lead-rubber bearings, friction-pendulum bearings, or high-damping rubber bearings) that are inserted between the ground and superstructure to isolate the superstructure from potentially dangerous ground motions. However, large base displacements relative to the ground caused by these low-stiffness elements can potentially exceed the allowable limits of structural designs under severe Seismic excitations. Moreover, passive base isolation cannot adapt to varying loading patterns. While shaking table testing is considered to be one of the most powerful means to investigate dynamical behaviours of structures, it may not be the most cost-effective approach. Real-time hybrid simulation (RTHS) enables the testing of certain physical critical components of structure experimentally, while simulating the remainder of the structure numerically. However, RTHS is challenging because it requires execution of each testing cycle within a fixed, small increment of time (typically, less than 1 ms).The RTHS consisted of a computational model and physical specimen(s) in a loop, with an appropriate loading unit and testing equipment. A schematic configuration for the RTHS in this study is shown in Fig. 3(a). The testing hardware in the RTHS included digital signal processor running numerical integration for the structure and generating the command signals; a small-scale MR damper driven by a servo-hydraulic actuator, which was controlled by a servo-controller; and analogy-to-digital and digital-to-analogy converters for signal processing. The sensors included a LVDT for displacement measurements and a load cell for measuring the MR damper force.

Tong Guo, Erjun Wu, Aiqun Li, Longwu Wei and Xingping Li, 2012, studied on the integral lifting and seismic isolation retrofit of the great hall of Nanjing Museum, a 78-year-old cultural and Historical building. The great hall of Nanjing Museum was built in 1933 and was known at the time as the Central Museum Preparatory Location. The hall represents an amalgamation of Eastern and Western culture with the style of a Liao Dynasty palace and RC frame structure. The main part of the hall has three stories and the rest has two stories. Laminated rubber bearings were installed underneath the columns of the hall after the lifting was completed. It is worth noting that base isolation might not be necessary for a typical structure without as much historic or cultural significance. Three types of rubber isolators were used in this project-

Table 2.4: Parameters of the Rubber Bearings (Ref.-Tong Guo, Erjun Wu, Aiqun Li, Longwu Wei and Xingping Li, 2012,)

Parameter	Type		
	LRB500	RB500	RB400
Effective area (cm2)	1885	1925	1237
Diameter of lead core (mm)	100	-	-
Number of rubber layers	20	20	20
Total thickness of rubber (mm)	96	100	100
Parameter	LRB500	RB500	RB400
Vertical stiffness (kN/mm)	2188	1579	741
Equivalent stiffness (kN/mm)	1.490	1.045	0.595
Original stiffness (kN/mm)	11.057	-	-
Post yielding stiffness (kN/mm)	1.084	-	-
Yielding force (kN)	47.87	-	-
Shear modulus of rubber (N/mm2)	0.392	0.392	0.392
Number of bearings used in the project	37	26	98

(a) Add a new floor after lifting (b) Rubber bearing in the new basement

Fig. 10. Extended column, added floor, and rubber isolators

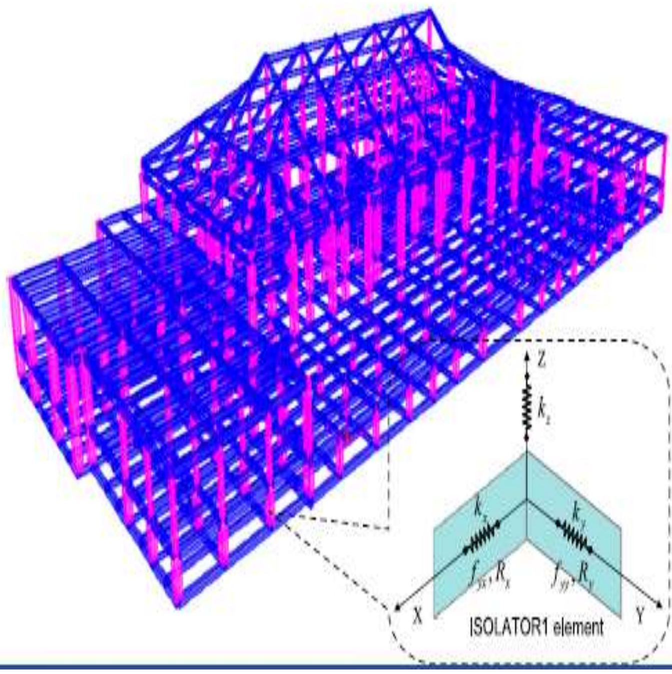


Fig. 2.3: Extended Column, Added Floor & Rubber Isoleters(Ref.-Tong Guo, Erjun Wu, Aiqun Li, Longwu Wei and Xingping Li, 2012,)

Table 2.5 : Story Shear Force before & after using Base Isolation (Ref.-Tong Guo, Erjun Wu, Aiqun Li, Longwu Wei and Xingping Li, 2012,)

Table 2. Story Shear Forces before and after Using Base Isolation (Loads in kN)

	x-direction		y-direction	
Story	Before	After	Before	After
3	9,408	3,294	9,203	3,093
2	15,088	5,005	14,903	5,424
1	21,983	7,224	22,432	8,132

Table 3. Interstory Displacement Angles before and after Base Isolation

	x-direction		y-direction	
Story	Before	After	Before	After
3	1/82	1/459	1/111	1/507
2	1/115	1/778	1/121	1/604
1	1/95	1/2,141	1/118	1/1,238

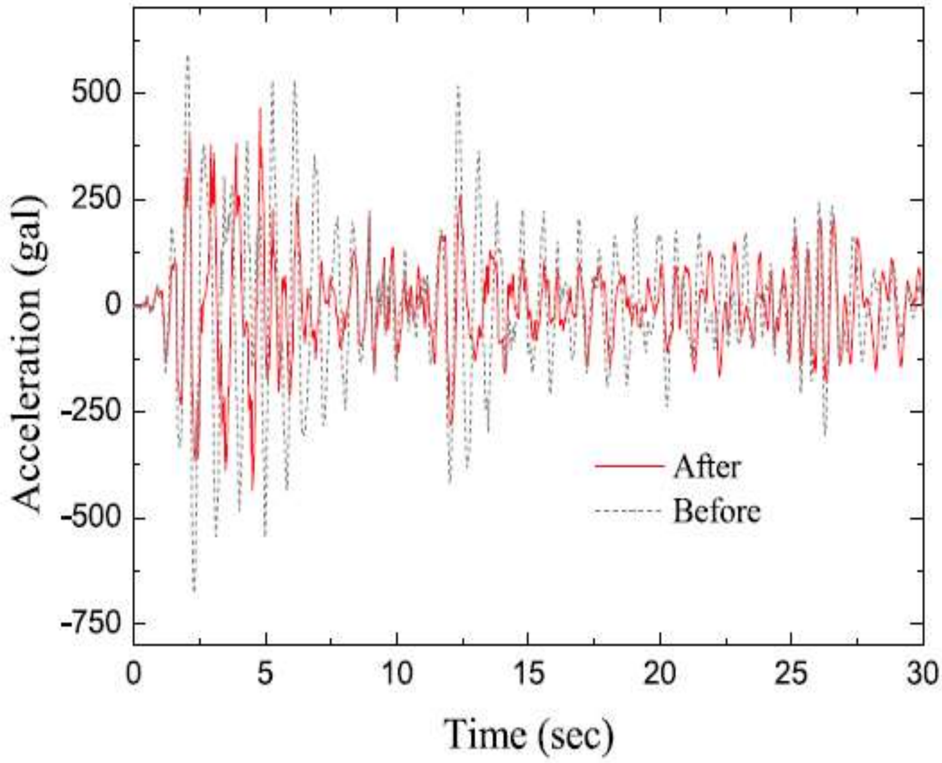


Fig. 2.4 : Acceleration (cm/sce^2) time-history of top floor before and after base isolation (Ref.-Tong Guo, Erjun Wu, Aiqun Li, Longwu Wei and Xingping Li, 2012,)

The seismic analyses indicated that the earthquake responses can be significantly reduced by using the laminated rubber isolators, thus improving the seismic performance of existing historical buildings.

2.2 Critical Remark on literature review

From the review of literature, it has been found that seismic isolation reduces the seismic hazard of a structure in a considerable amount. Various seismic base isolation systems are numerically modelled and investigated in recent years. From that we can able to find an efficient isolation system for a particular type of structure. Seismic isolation allows the engineer to control damage in moderate and large earthquakes for both a building and its contents using low-cost structural systems. Base isolation technique also provides alternative approach of upgrading existing structure. It also learned that very low cost base isolation system also available for the rural buildings.

However comparative study on the effect of base isolation of various isolator systems for different time history earthquake ground motion on symmetric and unsymmetrical building in Indian context is sparse. Thus the objective of the present study aims to study the effect of base isolation on RCC frame stimulates numerically by non-linear time history analysis.

Chapter III

Detail Discussion on Base Isolation

3.1 General

The structures are normally built firmly attached to the ground. Therefore naturally they receive all the ground movements at their base. The seismic response of any structural system for a given base motion depends on the mass, stiffness and damping distribution in the system. Equation of motion for a system is:- $\boldsymbol{m\ddot{v}} + \boldsymbol{c\dot{v}} + \boldsymbol{kv} = -\boldsymbol{m\ddot{v}_g}$

Base isolation technique provides an alternative approach for seismic design of many buildings as well as a convenient way of upgrading existing bridges.

It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent. Seismic Analysisis a subset of structuralanalysis and is the calculation of the response of a building (or non-building) structure to earthquakes. Abuilding has the potential to 'wave' back and forth during an earthquake (or even a severe wind storm). Thisis called the 'fundamental mode', and is the lowest frequency of building response. Most buildings,however, have higher modes of response, which are uniquely activated during earthquakes. The figure justshows the second mode, but there are higher 'shimmy' (abnormal vibration) modes. Nevertheless, the firstand second modes tend to cause the most damage in most cases. All real physical structures behavedynamically when subjected to loads or displacements. The additional inertia forces, from Newton's second law, are equal to the mass times the acceleration. If the loads or displacements are applied very slowly, theinertia forces can be neglected and a static load analysis can be justified. Hence, dynamic analysis is asimple extension of static analysis .In addition, all real structures potentially have an infinite number ofdisplacements. Therefore, the most critical phase of a structural analysis is to create a computer model witha finite number of mass less members and a finite number of node (joint) displacements that will simulatethe behaviour of the real structure. The mass of a structural system, which can be accurately estimated, islumped at the nodes. Also, for linear elastic structures, the stiffness properties of the members can beapproximated with a high degree of confidence with the aid of experimental data. However, the dynamicloading, energy dissipation properties and boundary (foundation) conditions for many structures are difficultanalyses using different computer models, loading and boundary conditions. It is not unrealistic to conduct20 or more computer runs to design a new structure or to investigate retrofit options for an existing structure.Because of the large number of computer runs required for a typical dynamic analysis, it is very importantthat accurate and numerically efficient methods be used within computer programs.

3.2 Linear Static Analysis or Response Spectrum Analysis

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building (either calculated or defined by the building code). The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels foisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces (e.g. force reduction factors).This response for a linear elastic systems to a given earthquake time history is most conveniently represented by acceleration response spectra.

3.3 Linear dynamic analysis

In the linear dynamic procedure, the building is modelled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modelled using either modal spectral analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. The advantage of these linear dynamic procedures with respect to linear static procedures is that higher modes can be considered. However, they are based on linear elastic response and hence the applicability decreases with increasing nonlinear behaviour, which is approximated by global force reduction factors.

In linear dynamic analysis, the response of the structure to ground motion is calculated in the time domain, and all phase information is therefore maintained.

Only linear properties are assumed. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis.

3.4 Non-linear static analysis or Push over analysis

In this process the magnitude of the lateral load is incrementally increased maintaining a predefined distribution pattern along the height of the building. With the increase in magnitude of the load, weak links and failure modes of the buildings can be observed. The structure is pushed until a collapse mechanism developed. Local non-linear effects are effects are modeled in push over analysis. The roof displacement against increased base shear generally be plotted to generate the push over curve.

3.5 Non-linear dynamic analysis - Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root sum of squares. In nonlinear dynamic analysis, the nonlinear properties of the structure are considered as part of a time domain analysis. This approach is the most rigorous, and is required by some building codes for buildings of unusual configuration or of special importance. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input; Therefore, several analyses are required using different ground motion records to achieve a reliable estimation of the probabilistic distribution of structural response. Since the properties of the seismic response depend on the intensity, or severity, of the seismic shaking, a comprehensive assessment calls for numerous nonlinear dynamic analyses at various levels of intensity to represent different possible earthquake scenarios. This has led to the emergence of methods like the Incremental Dynamic Analysis.

3.6 Base Isolation System

Base isolation is a mechanism that provides earthquake resistance to the structure. This system decouple the building from the horizontal ground motion induced by earthquake, and offer a very stiff vertical components to the base level of the superstructure in connection to substructure (foundation). It shifts the fundamental lateral period, Ta, dissipates the energy in damping, and reduces the amount of the lateral forces that transferred to the inter-story drift, and the floor acceleration. The Structural Engineers Association of Northern California (SEONC) published a simple regulation titled “Tentative Isolation Design Requirements” in 1986, which later was added as provisions in the Uniform Building Code 1997, FEMA 273 with exception of permit to pushover, and IBC2000. The structural bearing criteria include vertical and horizontal loads, lateral motion, and lateral rotation that transferred from the superstructure into the bearing and from the bearing to the substructure. Bearing allows for stress-free support of the structure in terms of (1) they can rotate in all directions, (2) they deform in all directions, (3) they take horizontal forces (wind, earthquake).

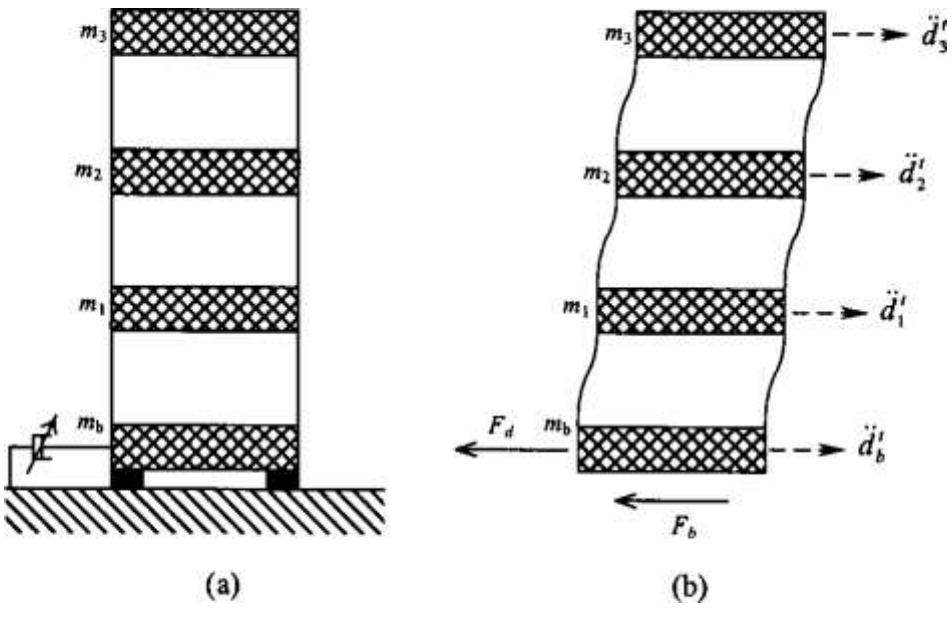


Fig.3.1 :Base isolated three story systems

The concept of base isolation systems is quite simple by interposing structural elements with low horizontal stiffness between the structure and the foundation to decouple the structure from the horizontal components of the ground motion which gives the structure a very low frequency than both its fixed base and the ground motion. Inertial forces are reduced by lengthening the fundamental period of vibration and added damping through the introduction of elements (Isolators)with horizontal and vertical stiffness that decouple the superstructure from the supporting substructure.Base isolation is incorporated to the structure to introduce flexibility at the supports of a structure in the horizontal plane so as to ensure that the time period of the structure is well above the predominant periods of the probable earthquake. Now in this process, the relative displacement amplitude increases, hence often damping or restraining elements have also to be introduced simultaneously to restrict the extent of relative movement caused by the earthquake. Although this principle is not new, its practical exploration has occurred only recently in the last about 15 years during which suitable hardware of isolating devices has been developed and actually applied to some constructions of buildings, bridges and atomic power plants.

3.7 Classification of base isolation system

A. Classification based on Basic principles of dynamics
    i) A method of control and adjust restoring forces characteristics.
    ii) A method to control and adjust damping.
    iii) A method to control and adjust mass.
    iv) A method to adjust input motion (a combination of above methods).
B. Classification based on Realization Procedure
    i) Passive way
    ii) Active way
C. Classification based on Installed Location
    i) External types (like base isolation)
    ii) Internal types (internal elements)

On the basis of above classification, the seismic base isolation is an external type, works in passive way and provides a method of seismic response control by adjusting stiffness and damping. Seismic isolation systems have been shown to not only reduce the response of the primary structure, but to also reduce damage to equipment and other non-structural secondary elements.

3.7.1 Types of structural bearings

The bearings require to (1) carry high permanent compression load with minimum compression deflection; (2) to accommodate horizontal movement by shear deflection with low shear stiffness to prevent excessive loads on the buildings footings due to thermal expansion and contraction; (3) to accommodate rotational deflections due to the transfer slab hogging and sagging; (4) to accommodate live loads with minimal additional compressive deflection; (5) to have a natural frequency particular to the application. The type of bearing as follows:

- Plain elastomeric
    1. Natural Rubber (poly-isoprene)
    2. Neoprene (poly- chloroprene)
- Steel reinforced elastomeric
- Roller bearing
- Rocker bearing
- Pot bearing
- Disc bearing
- Spherical and cylindrical bearing

Where factors affecting the selection criterion include: dead load, total load, lateral load, uplift, rotations, translations, cost and durability.

3.8. Essential elements for a base isolation system –

There are three types of practical base isolation system available

1. Decoupling between the superstructure and the base with or without flexible mounting so that the effective period of vibration of the total system is lengthened sufficiently to reduce the force response.
2. A damper or energy dissipater so that the relative displacements between the structure and its supports can be controlled.
3. A means of providing rigidity under low in-service load levels such as wind and minor earthquakes so that the structure behaves as if fixed at base during normal service loads.

3.9. Advantage of base isolation system

1. For a base isolation system, the structural deformation going into the inelastic/plastic range and the consequent damage is likely to be completely eliminated. The structure will need designing for much smaller accelerations, hence should be more economical.
2. The relative story displacements (drift) will be reduced. Hence the 'non-structural' damage to cladding, partition walls etc. will be minimized.
3. The response acceleration at higher floors will be much reduced, hence the damage to equipment, service lines will be minimized.
4. Most base isolation system can be easily replaced after a damaging earthquake by jacking up the structure.

Cost comparison studies of fixed-base and base-isolated buildings, which included initial design and construction costs, were performed for selected cases where comparative data were available. Incorporating seismic isolation into a new building was generally found to result in a cost premium in the range of 1–5%, because higher performance standards for isolated buildings did not allow sufficient reductions in the cost of the structural framing system to offset the cost of the isolation system. The cost premium for seismic isolation may have increased since 1990 due to additional requirements in recent codes. Seismic hazards on stiff or fixed building caused damage of ceiling & lights, building equipment's, elevators, and other content which leads to the heavy economic loss. In base isolated structure, structural frame, piping & duct work, façade & Windows, partitions etc. Base isolation also has some short term benefits such as it helps to reduce member size, equipment bracing and the cost of alter.

Chapter IV

Non Linear Time History Analysis

4.1. General

Non-linear static analysis is generally required to calculate the seismic demand of the structure and it utilizes the smoothened response spectra. Non-linear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root sum of squares. In nonlinear dynamic analysis, the nonlinear properties of the structure are considered as part of a time domain analysis. This approach is the most rigorous, and is required by some building codes for buildings of unusual configuration or of special importance. However, the nonlinear response can be very sensitive to the characteristics of the individual ground motion used as seismic input; Therefore, several analyses are required using different ground motion records to achieve a reliable estimation of the probabilistic distribution of structural response. Since the properties of the seismic response depend on the intensity, or severity, of the seismic shaking, a comprehensive assessment calls for numerous nonlinear dynamic analyses at various levels of intensity to represent different possible earthquake scenarios. This has led to the emergence of methods like the Incremental Dynamic Analysis.

Time history analysis provides nonlinear evolution of dynamic structural response under time dependent loading, which may vary according to the specified time function or arbitrary in nature. The dynamic equilibrium can be solved by using either modal or direct integration method.

4.2 Direct Integration Time History

Direct-integration time-history analysis is a nonlinear, dynamic analysis method in which the equilibrium equations of motion are fully integrated as a structure is subjected to dynamic loading. Analysis involves the integration of structural properties and behaviors at a series of time steps which are small relative to loading duration.

4.2.1. Newmark Method

The most general approach for the solution of the dynamic response of structural systems is the direct numerical integration of the dynamic equilibrium equations. This involves, after the solution is defined at time zero, the attempt to satisfy dynamic equilibrium at discrete points in time. Most methods use equal time intervals at Δt, $2\Delta t$ $3\Delta t$........$N\Delta t$. All approaches can fundamentally be classified as either explicit or implicit integration methods. Explicit methods do not involve the solution of a set of linear equations at each step. Basically, these methods use the differential equation at time "t" to predict a solution at time " $t + \Delta t$ ". For most real structures, which contain stiff elements, a very small time step is required in order to obtain a stable solution. Therefore, all explicit methods are conditionally stable with respect to the size of the time step. Implicit methods attempt to satisfy the differential equation at time "t" after the solution at time "$t - \Delta t$" is found. These methods require the solution of a set of linear equations at each time step; however, larger time steps may be used. Implicit methods can be conditionally or unconditionally stable. There exist a large number of accurate, higher-order, multi-step methods that have been developed for the numerical solution of differential equations. These multistep methods assume that the solution is a smooth function in which the higher derivatives are continuous. The exact solution of nonlinear structures requires that the accelerations, the second derivative of the displacements, are not smooth functions. This discontinuity of the acceleration is caused by the nonlinear hysteresis of most structural materials, contact between parts of the structure, and buckling of elements.

In 1959 Newmark presented a family of single-step integration methods for the solution of structural dynamic problems for both blast and seismic loading. During the past 40 years Newmark's method has been applied to the dynamic analysis of many practical engineering structures. In addition, it has been modified and improved by many other researchers. In order to illustrate the use of this family of numerical integration methods consider the solution of the linear dynamic equilibrium equations written in the following form:

$$\mathbf{M}\ddot{\mathbf{u}}_t + \mathbf{C}\dot{\mathbf{u}}_t + \mathbf{K}\mathbf{u}_t = \mathbf{F}_t$$

The direct use of Taylor's series provides a rigorous approach to obtain the following two additional equations:

$$\mathbf{u}_t = \mathbf{u}_{t-\Delta t} + \Delta t\,\dot{\mathbf{u}}_{t-\Delta t} + \frac{\Delta t^2}{2}\ddot{\mathbf{u}}_{t-\Delta t} + \frac{\Delta t^3}{6}\dddot{\mathbf{u}}_{t-\Delta t} + \ldots\ldots$$

$$\dot{\mathbf{u}}_t = \dot{\mathbf{u}}_{t-\Delta t} + \Delta t\,\ddot{\mathbf{u}}_{t-\Delta t} + \frac{\Delta t^2}{2}\dddot{\mathbf{u}}_{t-\Delta t} + \ldots\ldots$$

Newmark truncated these equations and expressed them in the following form:

$$\mathbf{u}_t = \mathbf{u}_{t-\Delta t} + \Delta t\,\dot{\mathbf{u}}_{t-\Delta t} + \frac{\Delta t^2}{2}\ddot{\mathbf{u}}_{t-\Delta t} + \beta\,\Delta t^3\dddot{\mathbf{u}}$$

$$\dot{\mathbf{u}}_t = \dot{\mathbf{u}}_{t-\Delta t} + \Delta t\,\ddot{\mathbf{u}}_{t-\Delta t} + \gamma\,\Delta t^2\dddot{\mathbf{u}}$$

If the acceleration is assumed to be linear within the time step, the following equation can be written:

$$\dddot{\mathbf{u}} = \frac{(\ddot{\mathbf{u}}_t - \ddot{\mathbf{u}}_{t-\Delta t})}{\Delta t}$$

The substitution of above equation into previous equations produces Newmark's equations in standard form

$$\mathbf{u}_t = \mathbf{u}_{t-\Delta t} + \Delta t\,\dot{\mathbf{u}}_{t-\Delta t} + (\frac{1}{2} - \beta)\Delta t^2\ddot{\mathbf{u}}_{t-\Delta t} + \beta\,\Delta t^2\ddot{\mathbf{u}}_t$$

$$\dot{\mathbf{u}}_t = \dot{\mathbf{u}}_{t-\Delta t} + (1-\gamma)\Delta t\,\ddot{\mathbf{u}}_{t-\Delta t} + \gamma\,\Delta t\,\ddot{\mathbf{u}}_t$$

4.2.2. The Hilber-Hughes-Taylor method

The Hilber-Hughes-Taylor (HHT) method (also known as the alpha-method) is widely used in the structural dynamics community for the numerical integration of a linear set of second Ordinary Differential Equations (ODE). A precursor of the HHT method is the Newmark method, in which a family of integration formulas that depend on two parameters β and γ is defined:

$$\mathbf{q}_{n+1} = \mathbf{q}_n + h\dot{\mathbf{q}}_n + \frac{h^2}{2}\left[(1-2\beta)\,\ddot{\mathbf{q}}_n + 2\beta\ddot{\mathbf{q}}_{n+1}\right]$$
$$\dot{\mathbf{q}}_{n+1} = \dot{\mathbf{q}}_n + h\left[(1-\gamma)\,\ddot{\mathbf{q}}_n + \gamma\ddot{\mathbf{q}}_{n+1}\right]$$

These formulas are used to discretize at time t_{n+1} the equations of motion using an integration step size h:

$$\mathbf{M}\ddot{\mathbf{q}}_{n+1} + \mathbf{C}\dot{\mathbf{q}}_{n+1} + \mathbf{K}\mathbf{q}_{n+1} = \mathbf{F}_{n+1}$$

The only combination of β and γ that leads to a second-order integration formula is γ = 12 and β = 14. This choice of parameters produces the trapezoidal method, which is both a stable and second order. The drawback of the trapezoidal formula is that it does not induce any numerical damping in the solution, which makes it impractical for problems that have high-frequency oscillations that are of no interest or parasitic high-frequency oscillations that are a byproduct of the finite element discretization process. Thus, the major drawback of the Newmark family of integrators was that it could not provide a formula that was a-stable and second order and displayed a desirable level of numerical damping. The HHT method came as an improvement because it preserved the A stability and numerical damping properties, while achieving second order accuracy when used in conjunction with the second order linear ODE problem of Eq. of motion. The idea proposed in actually does not pertain the expression of the Newmark integration formulas, but rather the form of the discretized equations of motion. The new equation in which the integration formulas of Equations are substituted is

$$\mathbf{M}\ddot{\mathbf{q}}_{n+1} + (1+\alpha)\,\mathbf{C}\dot{\mathbf{q}}_{n+1} - \alpha\mathbf{C}\dot{\mathbf{q}}_{n} + (1+\alpha)\,\mathbf{K}\mathbf{q}_{n+1} - \alpha\mathbf{K}\mathbf{q}_{n} = \mathbf{F}\left(\tilde{t}_{n+1}\right)$$

Where

$$\tilde{t}_{n+1} = t_n + (1+\alpha)\,h$$

As indicated, the HHT method will possess the advertised stability and order properties provided and

$$\gamma = \frac{1-2\alpha}{2} \qquad \beta = \frac{(1-\alpha)^2}{4}$$

The smaller the value of α, the more damping is induced in the numerical solution. Note that in the limit, the choice α = 0 leads to the trapezoidal method with no numerical damping.

4.2.3. P-Delta effect

P-Delta effect also known as geometric nonlinearity, involves the equilibrium and compatibility relationships of a structural system loaded about its deflected configuration. Of particular concern is the application of gravity load on laterally displaced multi-story building structures. This condition magnifies story drift and certain mechanical behaviors while reducing deformation capacity.P-Delta effect typically involves large external forces upon relatively small displacements. If deformations become sufficiently large as to break from linear compatibility relationships, then Large-Displacement and Large-Deformation analyses become necessary.P-Δ effect, or P-"big-delta", is associated with displacements relative to member ends. Unlike P-δ, this type of P-Delta effect is critical to nonlinear modeling and analysis.

As indicated intuitively gravity loading will influence structural response under significant lateral displacement. P-Δ may contribute to loss of lateral resistance, ratcheting of residual deformations, and dynamic instability (Deierlein et al. 2010). Effective lateral stiffness decreases, reducing strength capacity in all phases of the force-deformation relationship (PEER/ATC 2010). To consider P-Δ effect directly, gravity load should be present during nonlinear analysis. Application will cause minimal increase to computational time, and will remain accurate for drift levels up to 10%

Chapter V

Numerical Study

5.1 General

The study of base isolation system has been carried out on building frames. In this study we have provided a rubber isolator at the base of each column. Those buildings frames are analyses for seismic loads in time history ground acceleration with the help of SAP2000.

5.2. Case Study

7 storied symmetric and asymmetric and 4 storied asymmetric building frames are considered for Non-linear time history analysis. The considered frames are as following

i) **7 storied symmetric building frame**

Plan, section and 3D model of the building frame are shown in the figure 5.1, 5.2 & 5.3

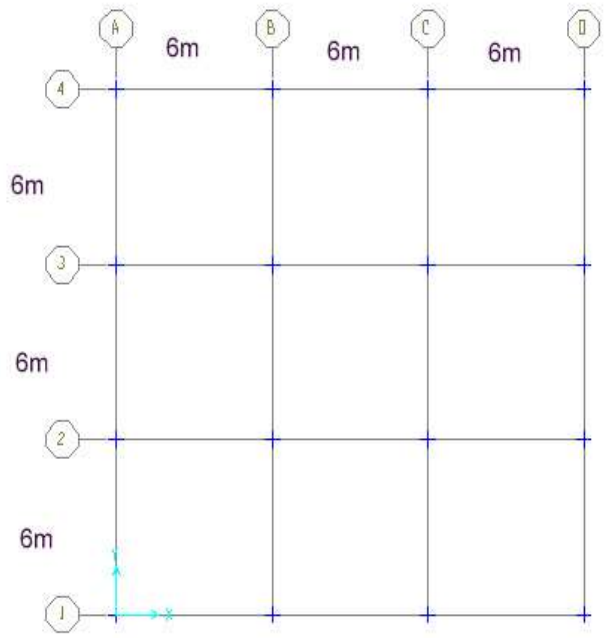


Fig 5.1 : Plan of 7 storied symmetric building frame

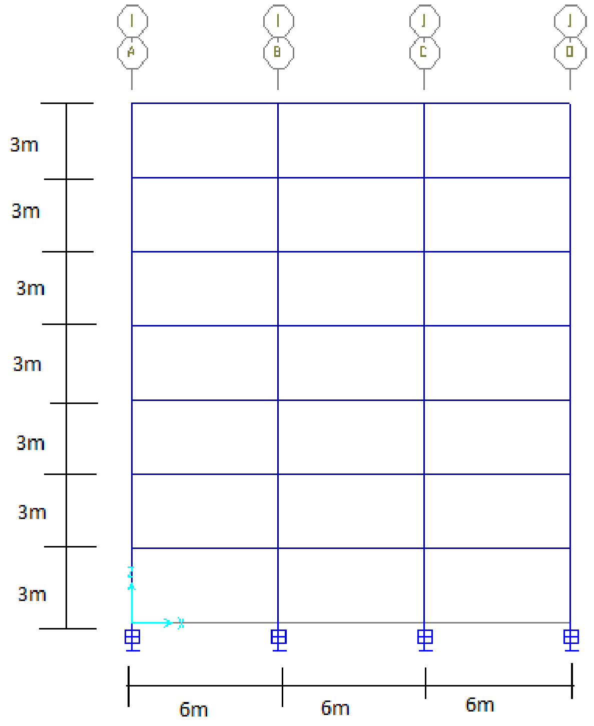


Fig 5.2 : Cross Section of 7 storied symmetric Building frame

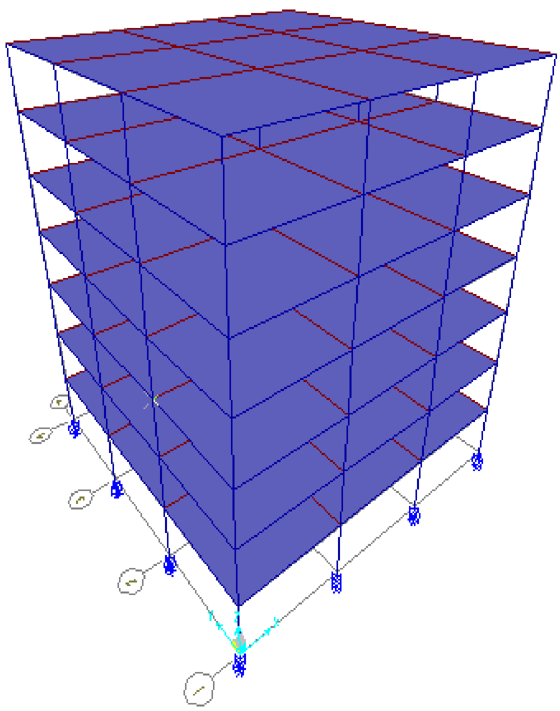


Fig 5.3 : 3D modeling of 7 storied symmetric building

ii) **4 storied asymmetric building frame**

Plan, section and 3D model of the building frame are shown in the figure 5.4, 5.5, 5.6 & 5.7

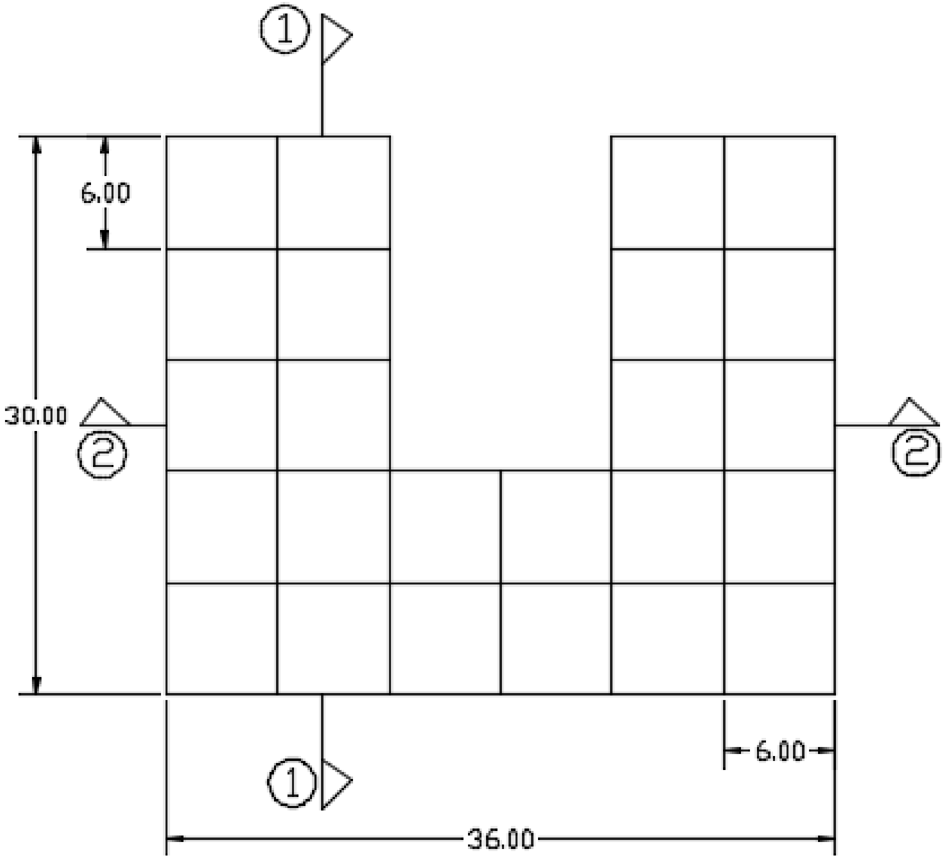


Fig 5.4 : Plan of 4 storied Building

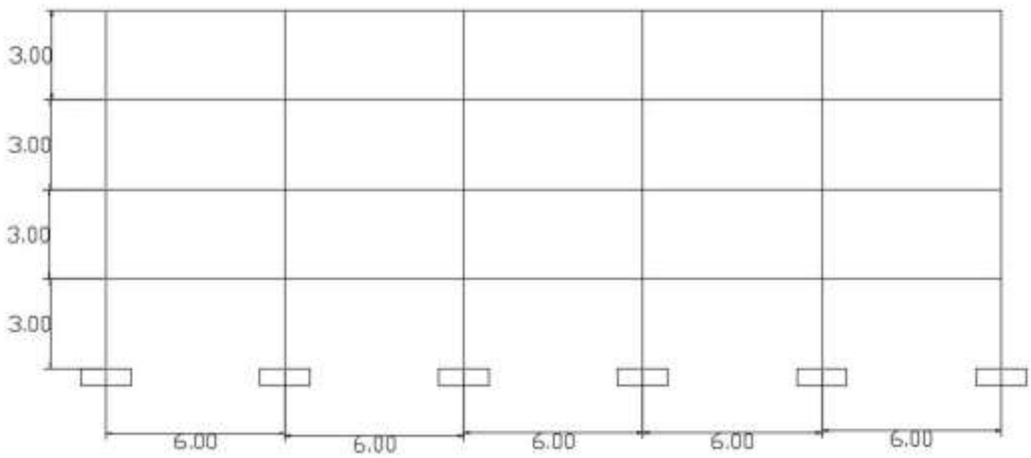


Fig 5.5 :Section -1-1

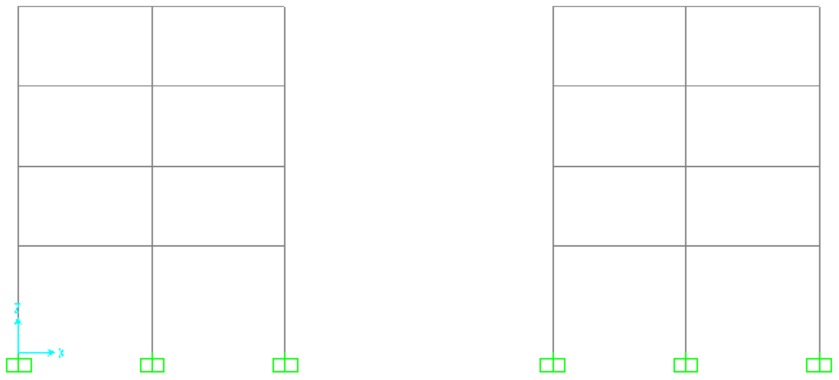


Fig 5.6 : Section 2-2

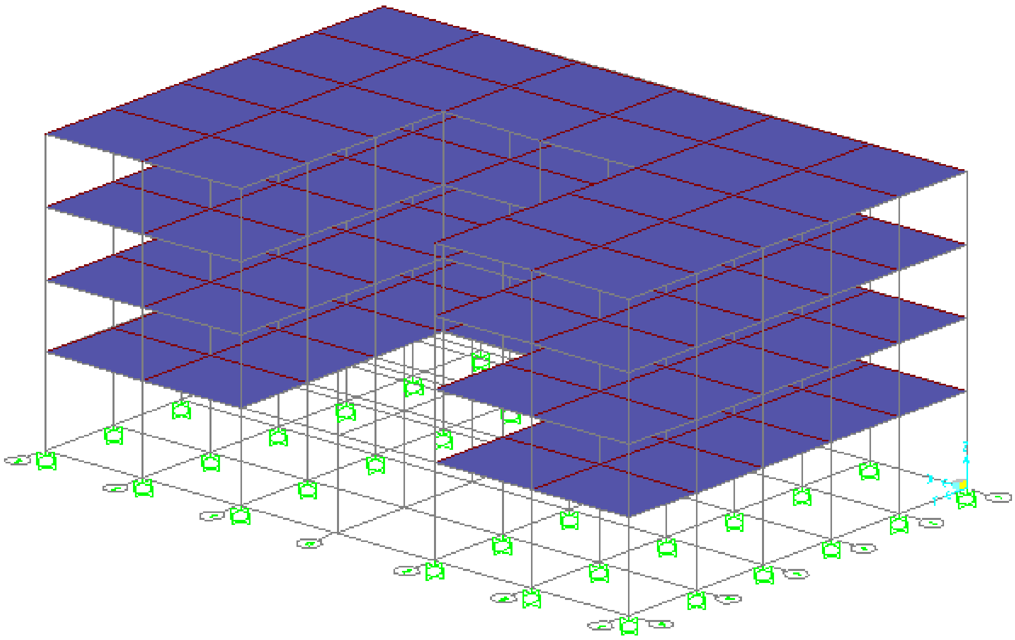


Fig 5.7 : 3D modelling of 4 storied building

iii) 7 storied Asymmetric building frame

Plan, section and 3D model of the building frame are shown in the figure 5.8, 5.9 & 5.10

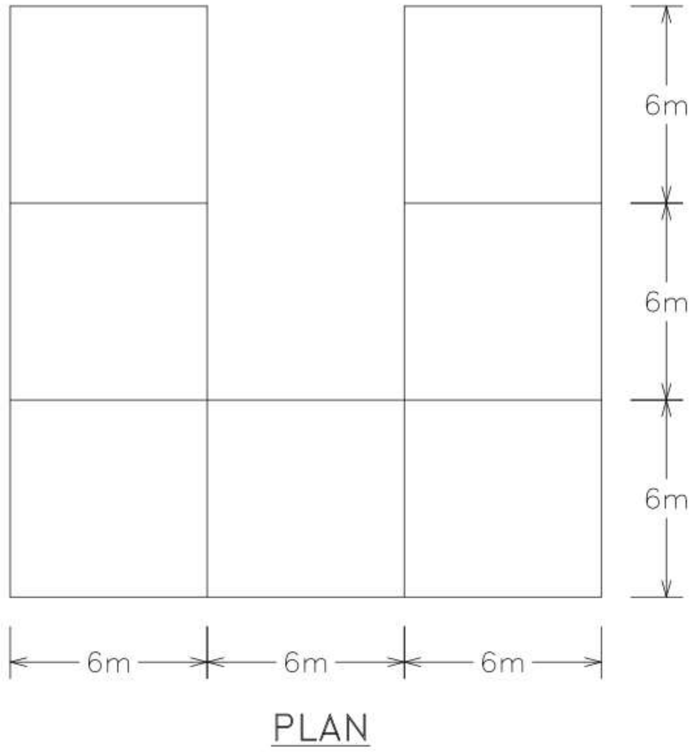


Fig 5.8 : Plan of 7 storied Asymmetric Building

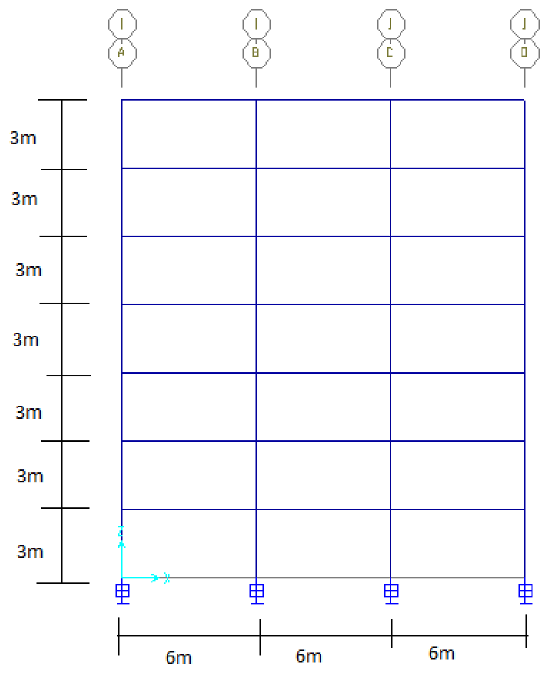


Fig 5.9 : Elevation of 7 storied Asymmetric Building

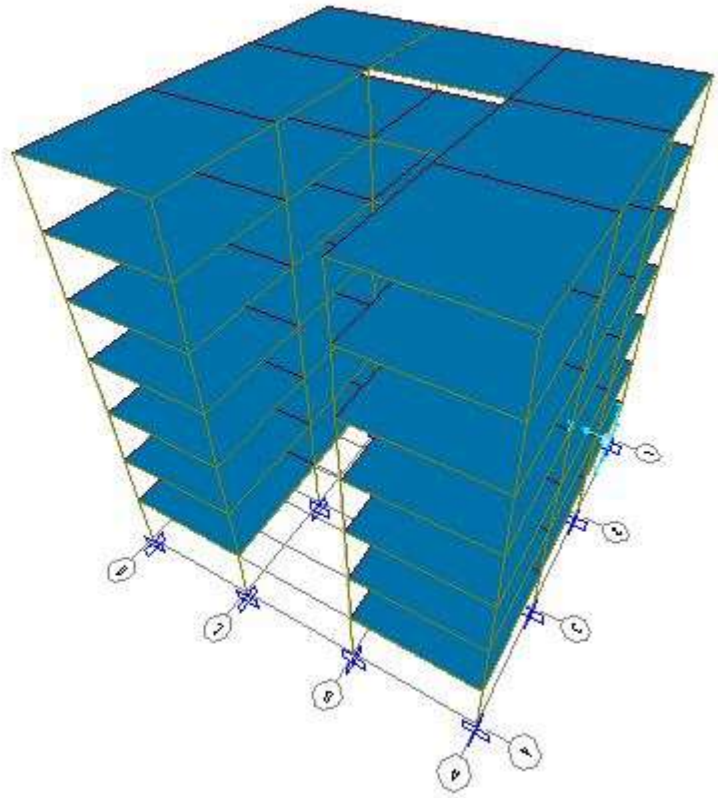


Fig 5.10: 3D modeling of 7 storied Asymmetric Building

5.3. Material property

For both the building frame, M30 grade of concrete and Fe415 steel for reinforcement has been used for all members. Adopted material properties and nonlinear stress strain model of concrete for nonlinear dynamic analysis are shown in figure 5.11,5.12,5.13 and 5.14 , as a screen shot from SAP model.

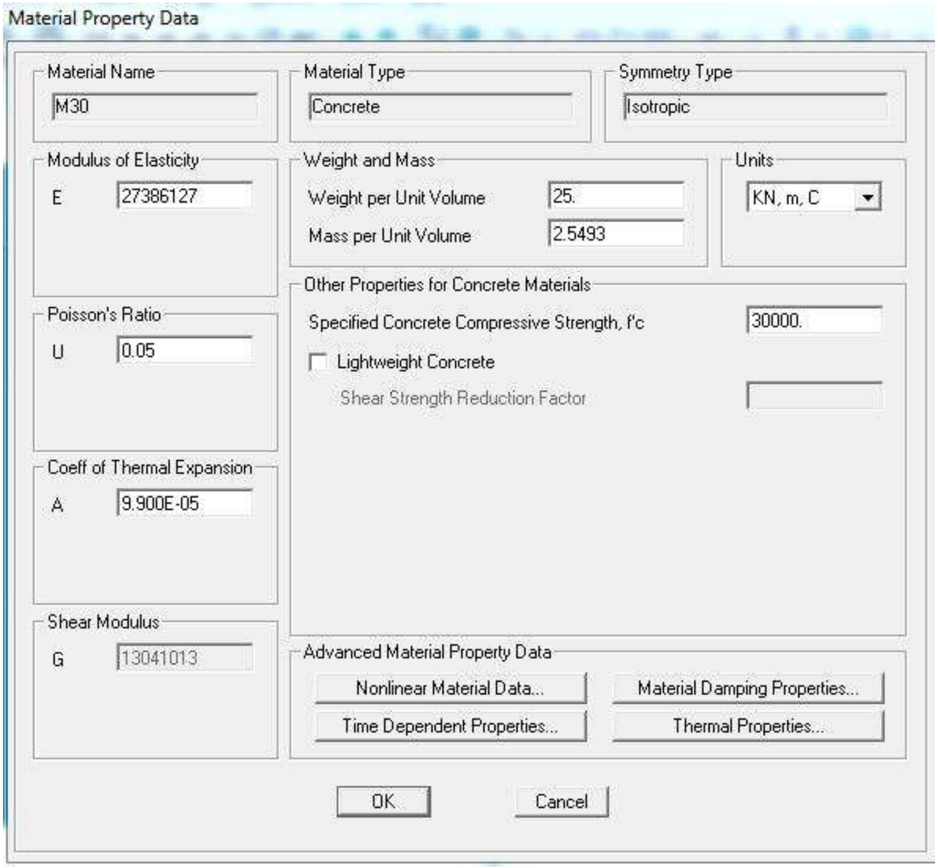


Fig 5.11 : Concrete material property

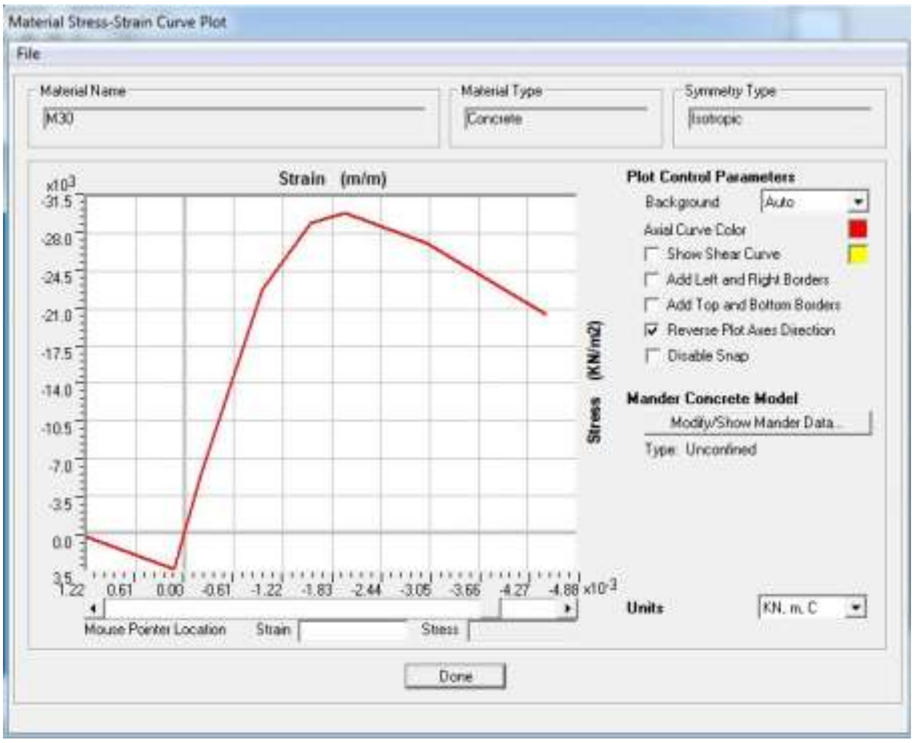


Fig 5.12 :Non linear stress strain relationship of concrete

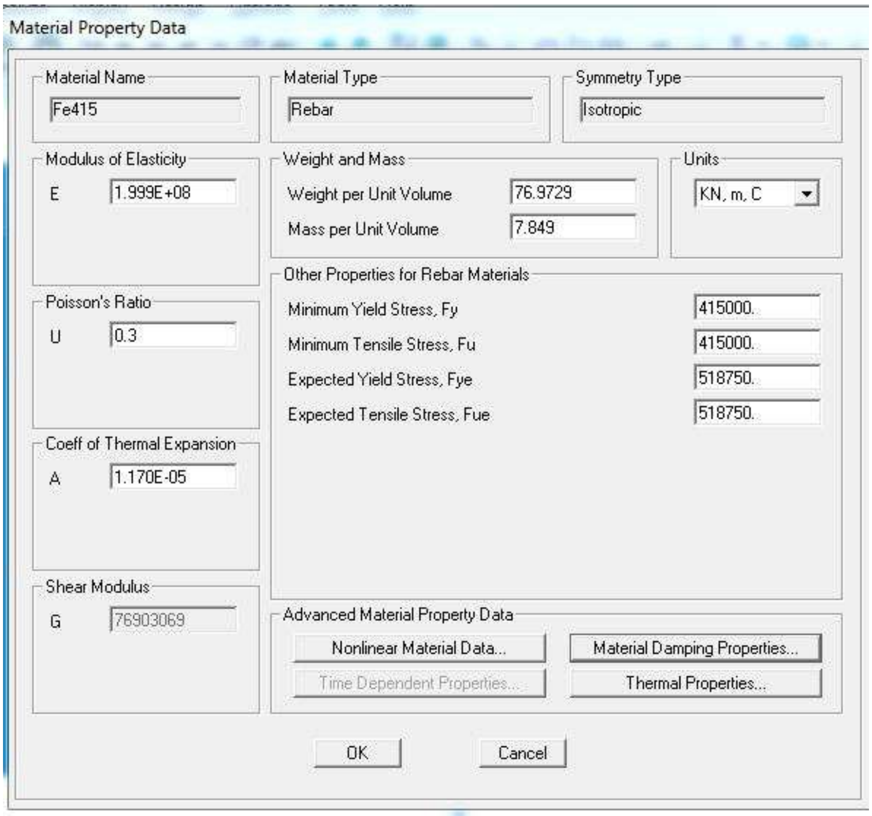


Fig 5.13 : Reinforcement material property

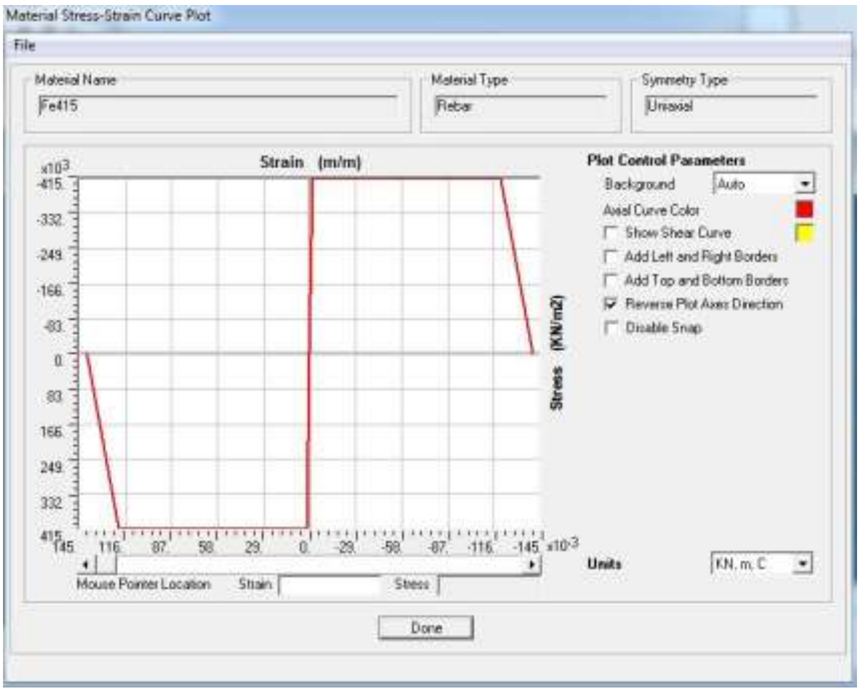


Fig 5.14 :Non linear stress strain relationship of Reinforcement

5.4. Frame Section properties

Stiff building has been considered for time history analysis. Following section properties for building frame has been used for modeling.

i) 7 storied symmetric and asymmetric building frame

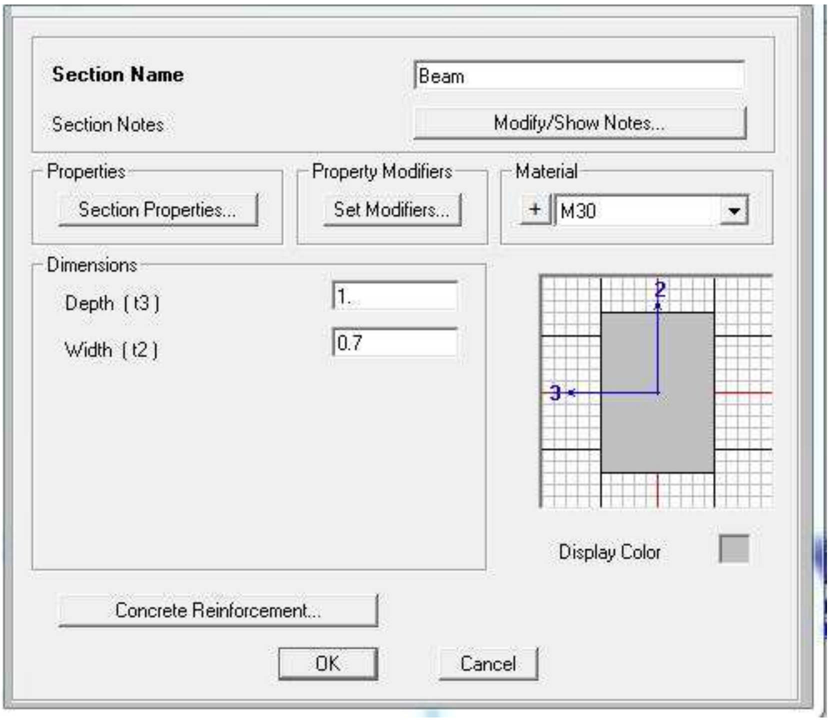


Fig 5.15 : Beam section details (in meter)

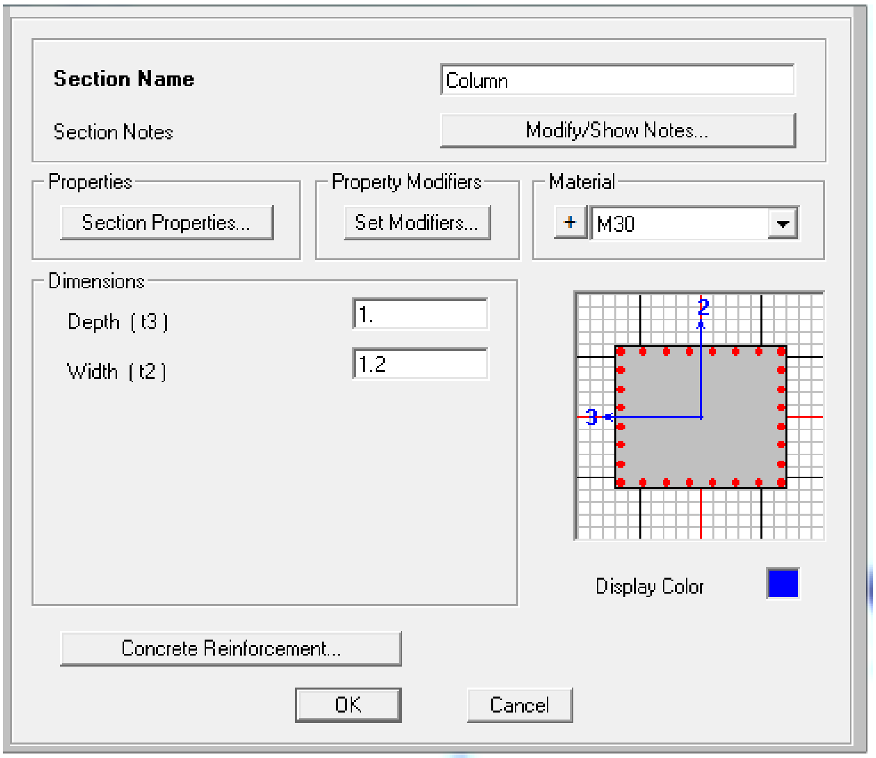


Fig 5.16: Column Section Details (in meter)

ii) 4 storied asymmetric building frame

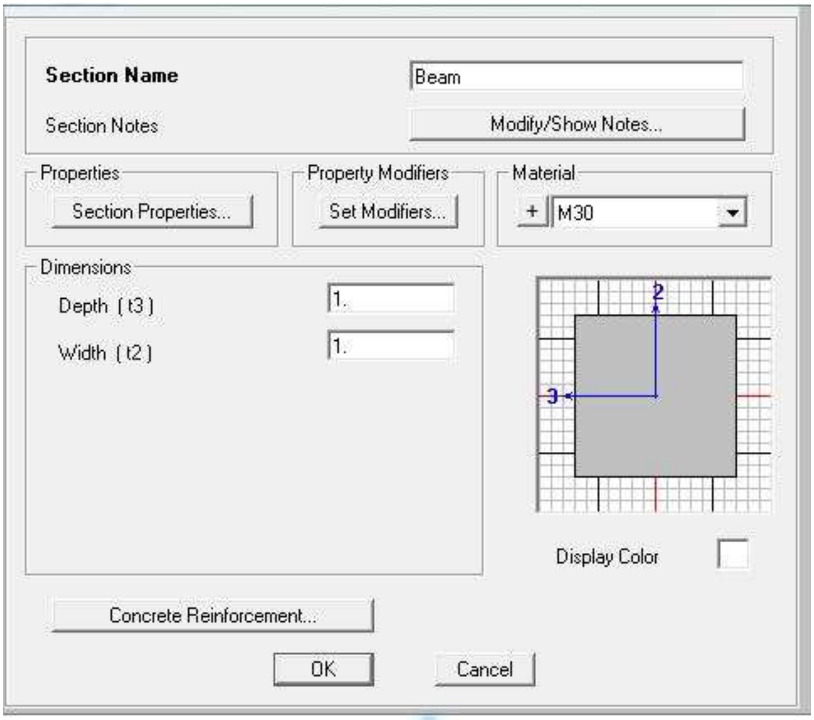


Fig 5.17 : Beam section details (in meter)

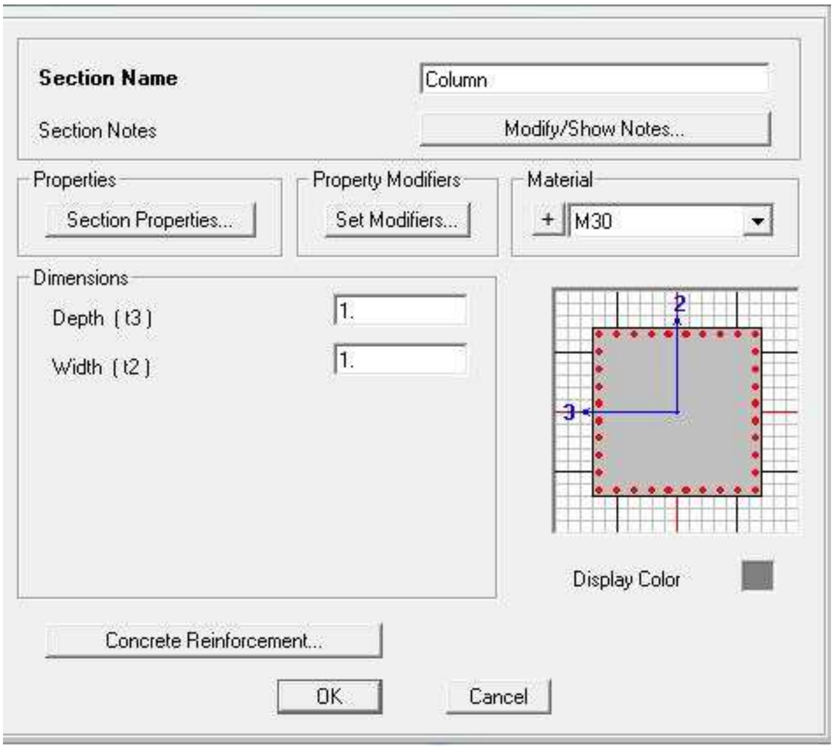


Fig 5.18 : Column Section Details (in meter)

5.5 Rubber isolator properties

These building frames are typically composed of rubber bearings that are inserted between the ground and superstructure to isolate the superstructure from potentially dangerous ground motions. (Ref. SAP 2000 manual)

Table 5.1 Properties of rubber isolator

Parameter	Type
	RUBBER
Vertical stiffness (kN/mm)	1751.68
Equivalent stiffness (kN/mm)	1.75
Post yielding stiffness ratio (kN/mm)	0.2
Yielding Strength (kN)	22.24

5.6. Structural Modeling

SAP2000 is structural analysis programming software. The 3D model of building has been analyzed using SAP2000 after defining the material properties and the frame section properties. Two different geometrical building frames have been analyzed with using different size of beam and column. Material properties of concrete and reinforcement steel have been defined. Fixed supports have been assigned to the base of the column.

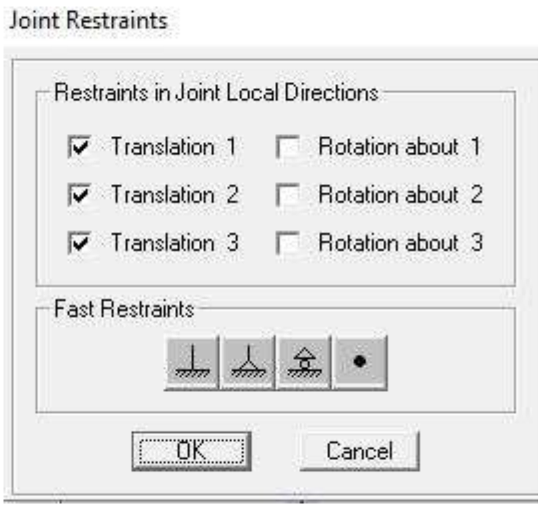


Fig 5.19 : Fixed Support system

Load pattern of dead load and live load have been defined. The data of acceleration in terms of gravity of a particular earthquake in time history format was recorded in a text file at the time of earthquake. This earthquake acceleration file is defined in the time history function.

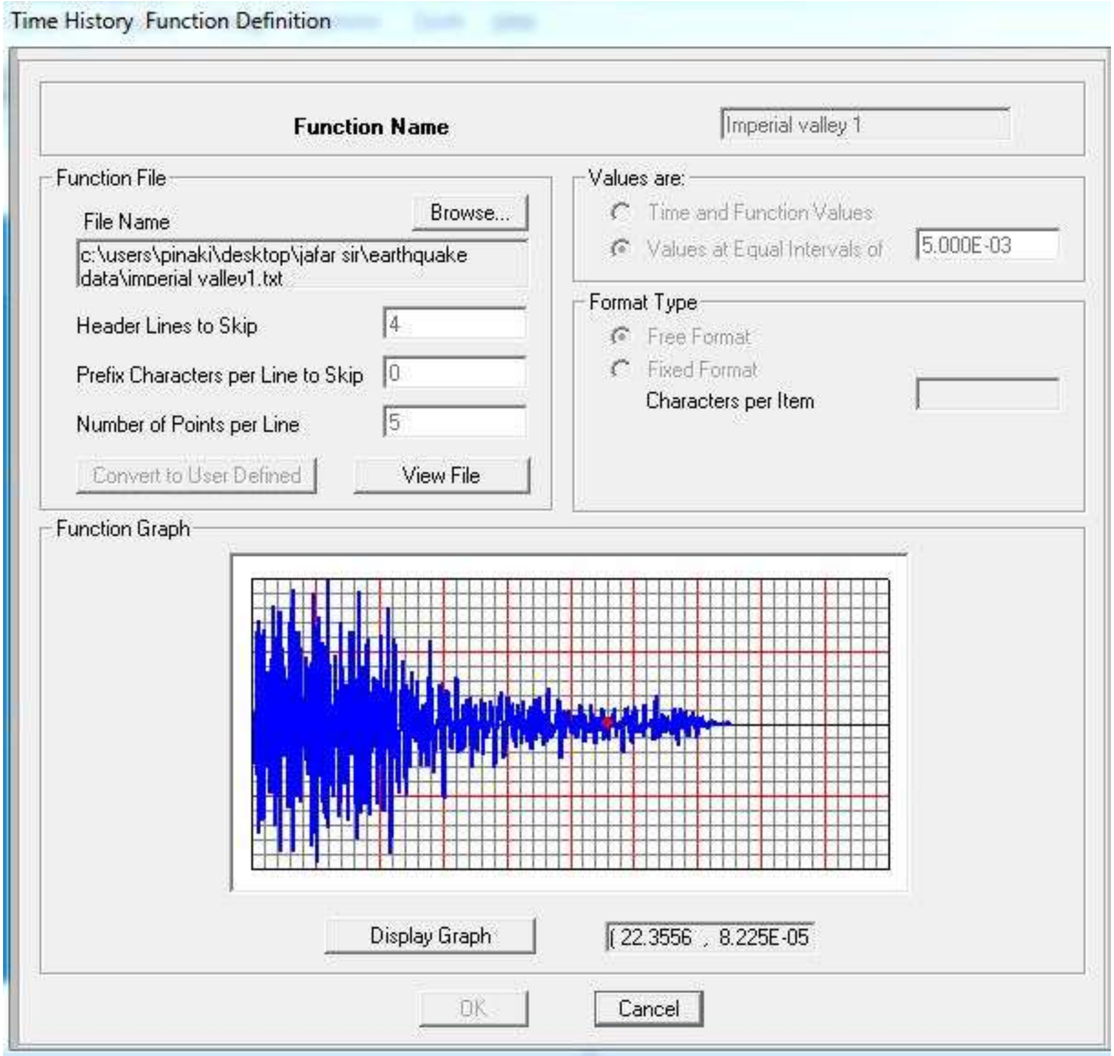


Fig 5.20 : Time History function input

Acceleration with respect to time has been displayed by the software. After that the time history function of the particular earthquake has been defined in the load cases area. After define the time history load case, the building frames are analyses without base isolation by running the load cases. Record the absolute ground acceleration and absolute displacement at roof level and various floor levels. The acceleration of the roof level has also been plotted. After that rubber isolator has been provided at the base of the structure.

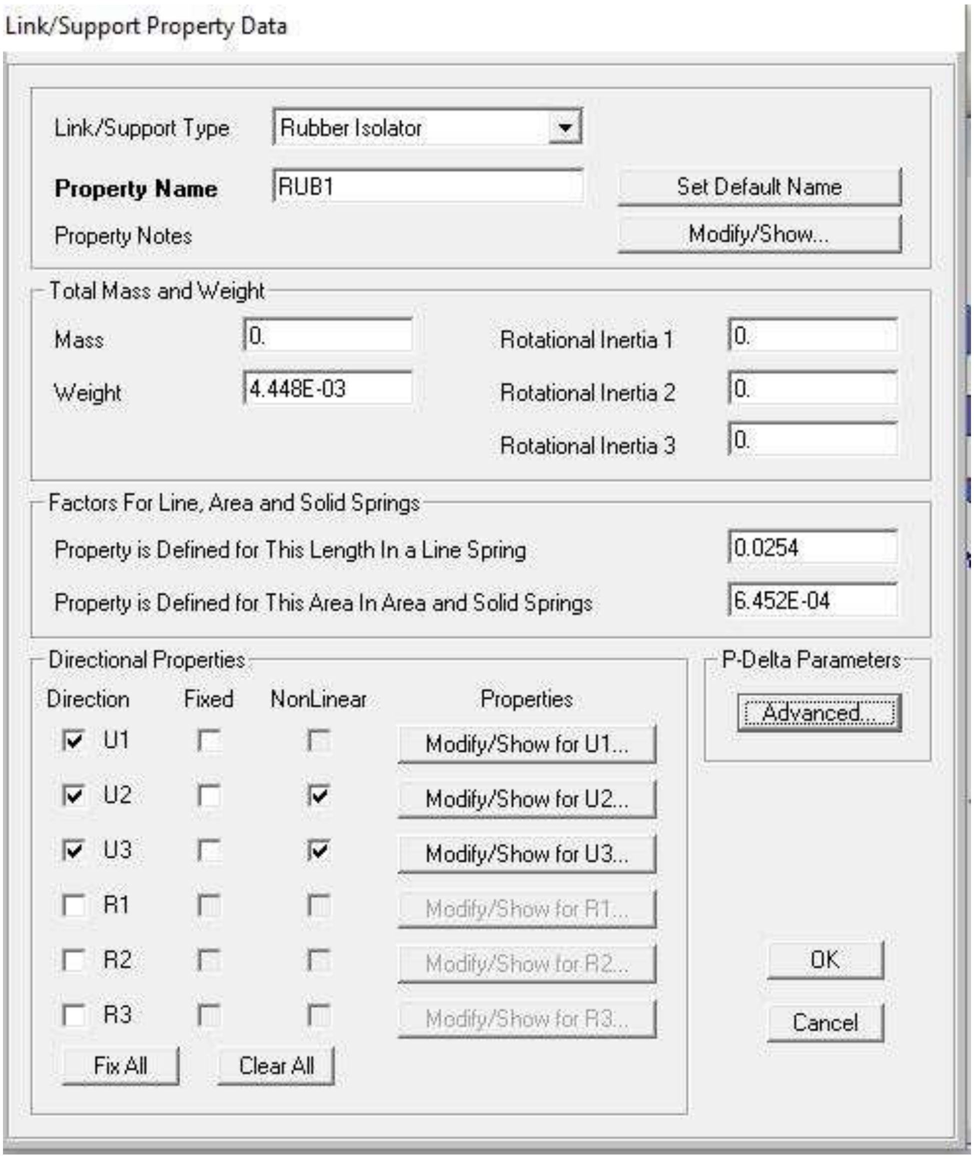


Fig 5.21 : Isolation Support system

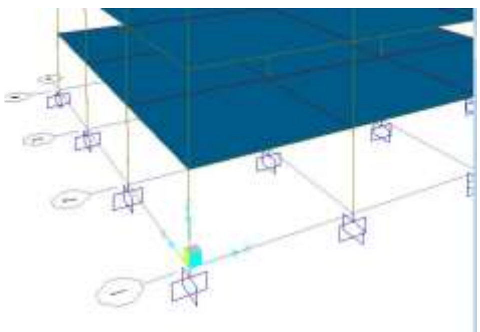


Fig 5.22 : Fixed base

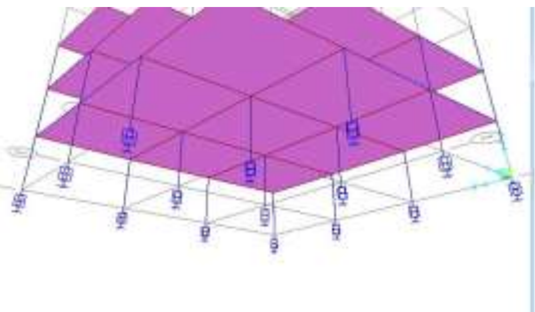


Fig 5.23 : Isolated Base

Each building frames are analyzed with rubber isolator and absolute acceleration and displacements and acceleration are recorded. The acceleration of roof and ground has been plotted as shown in fig.5.21 & 5.22. snapped from SAP 2000.

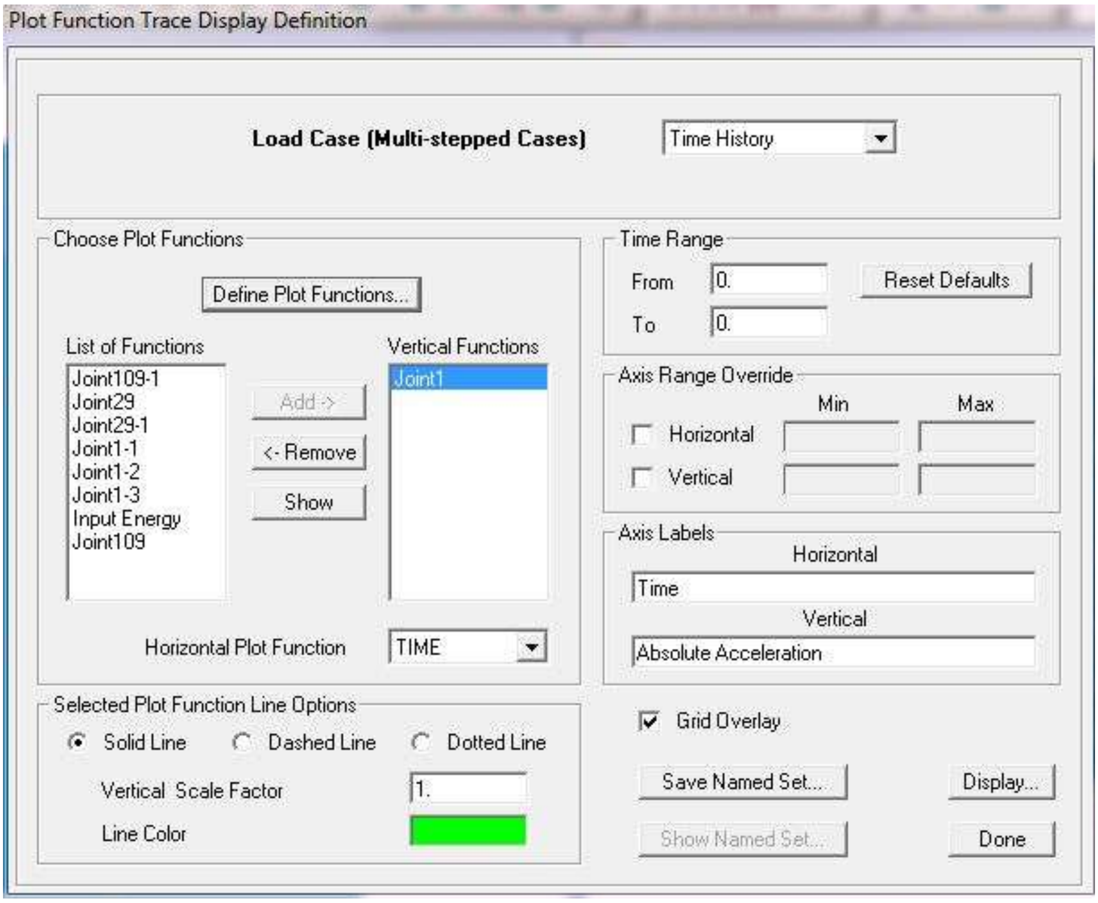


Fig.5.24 Plot functions trace display definition

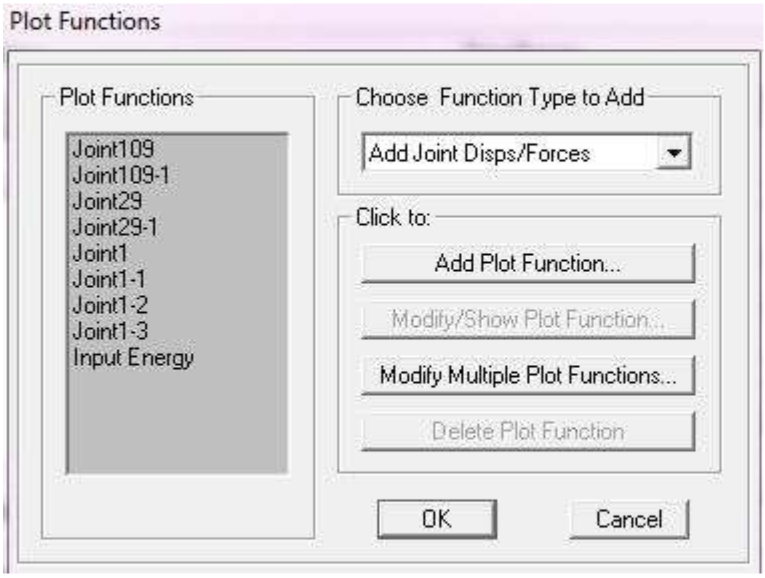


Fig. 5.25 Plot Function

5.7. Ground Acceleration

Some recorded earthquake ground acceleration data are inputted as a user defined function for non linear dynamic analysis.

Following earthquake ground acceleration are considered.

i) Park field earthquake (1966)
ii) Imperial valley earthquake (1938)
iii) Sikkim earthquake (2011)

I) Park field earthquake (1966)

Park field earthquake was occurred on 28^{th} June 1966 at 04:26. Data was recorded at an interval of 0.01 sec and total 4369 data are available for 43.69 sec. Magnitude of the earthquake was 6.0.Peak ground acceleration was .0475g

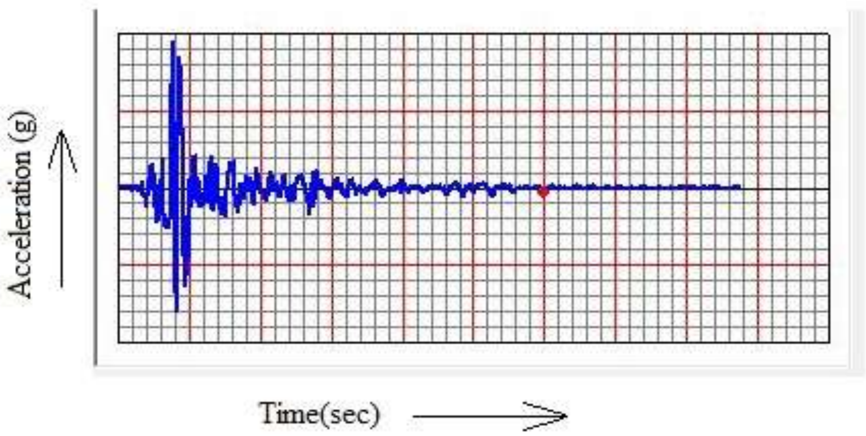


Fig 5.26 : Time vs. acceleration graph for park field earthquake

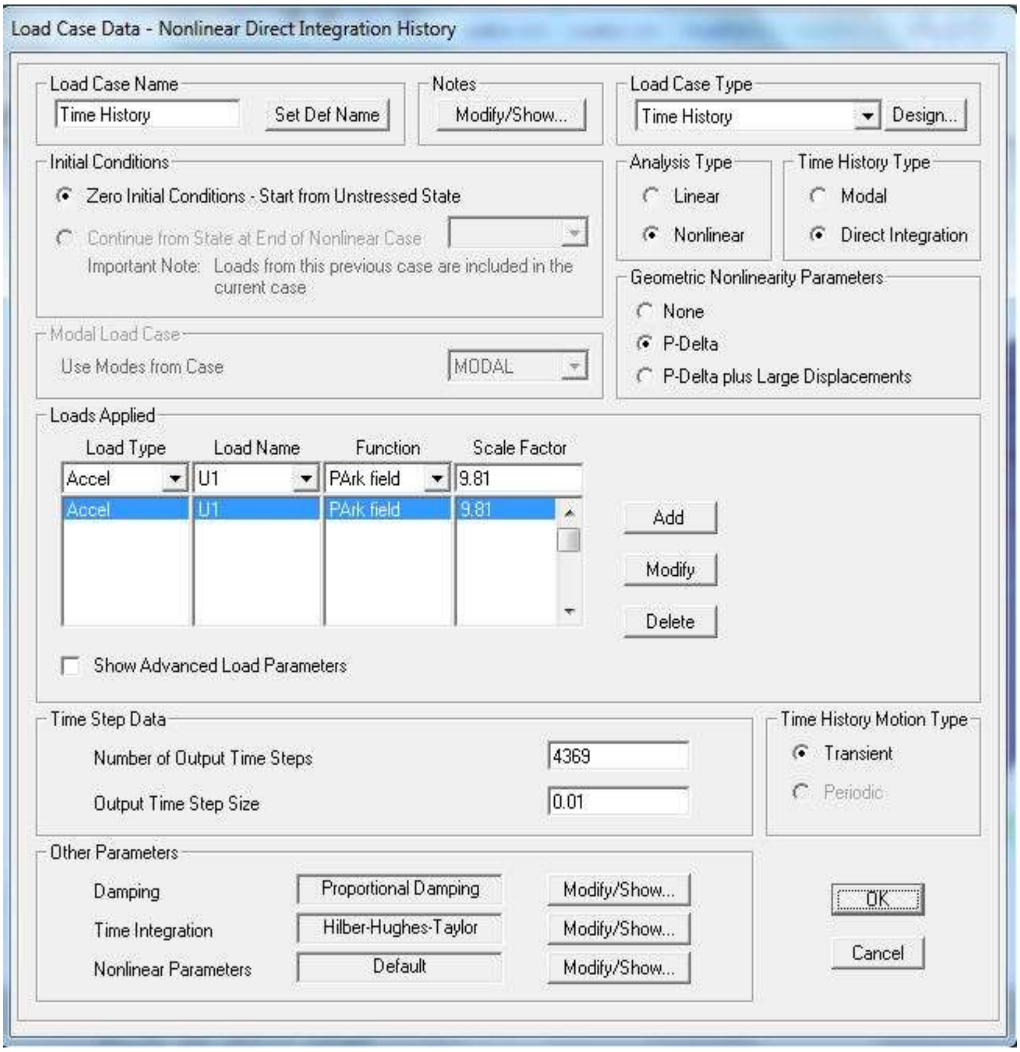


Fig 5.27 : Non Linear Direct Integration Time History data for park field earthquake

II) Imperial valley 1 Earthquake (1938)

Imperial Valley earthquake was occurred on 6th June 1938 at 02:42. Data was recorded at an interval of 0.005 sec and total 6001 data are available for 30.005 sec. Peak ground acceleration was 0.0118g

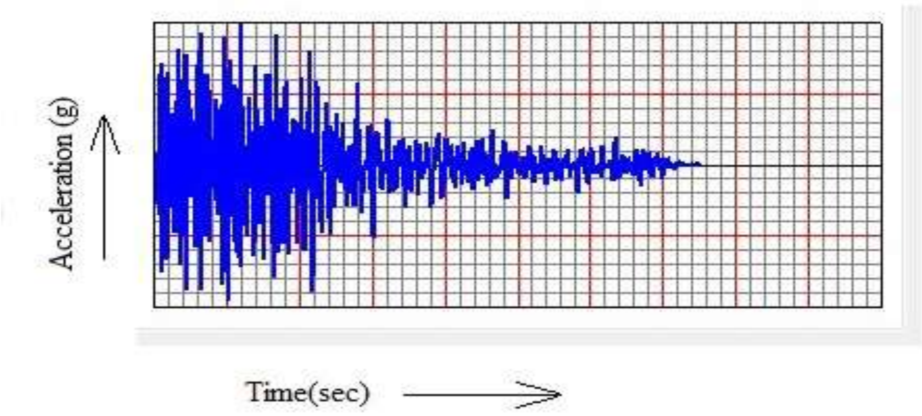


Fig 5.28 : Time vs Acceleration graph for Imperial Valley 1 earthquake

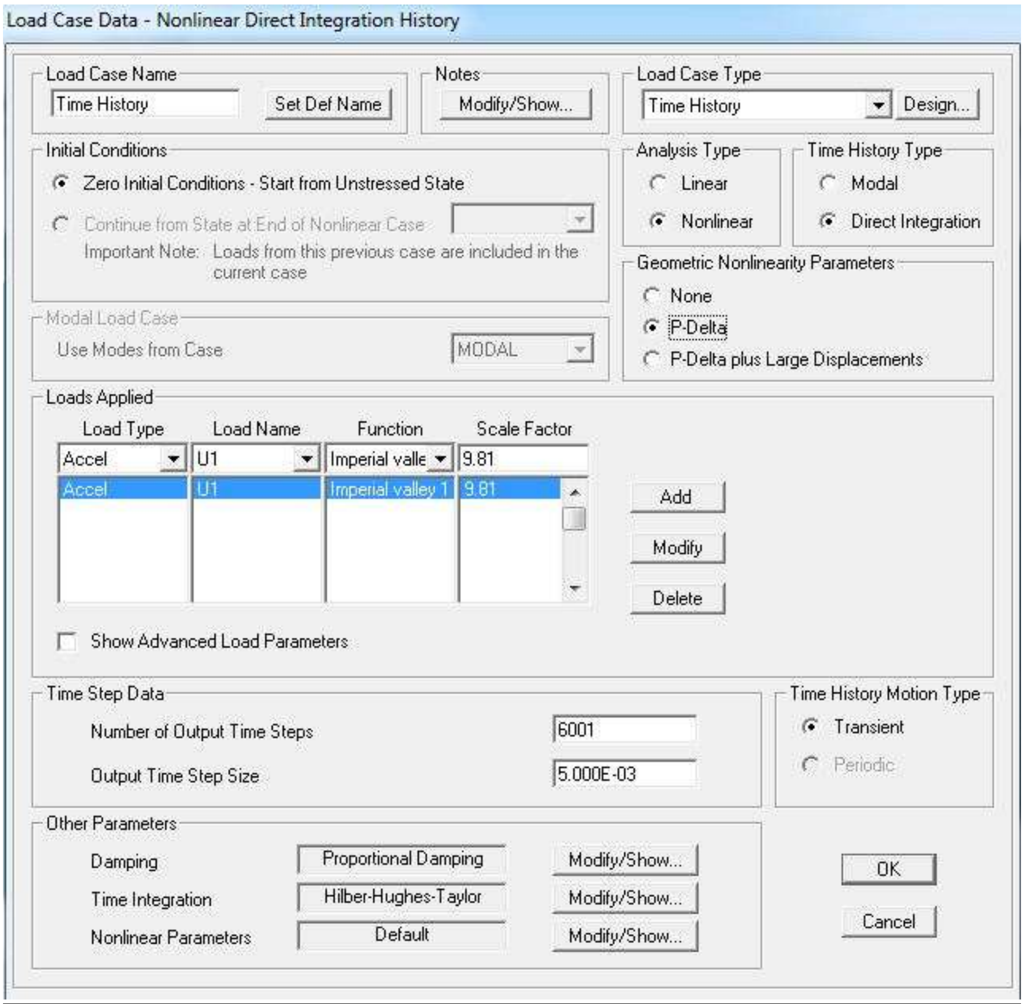


Fig 5.29: Non Linear Direct Integration Time History data for Imperial Valley

III) Sikkim Earthquake (2011)

Sikkim earthquake was occurred in 18th September 2011 at 18:10. Data was recorded at an interval of 0.005 sec and total 33970 data are available for 169.845 sec. Peak ground acceleration was 0.2056 g.

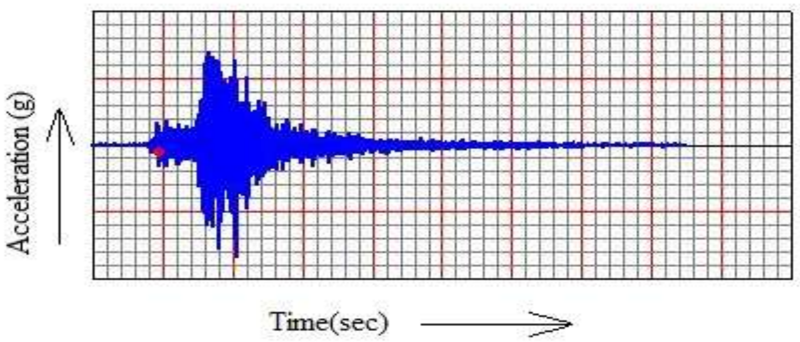


Fig 5.30 : Time vs Acceleration graph for Sikkim earthquake

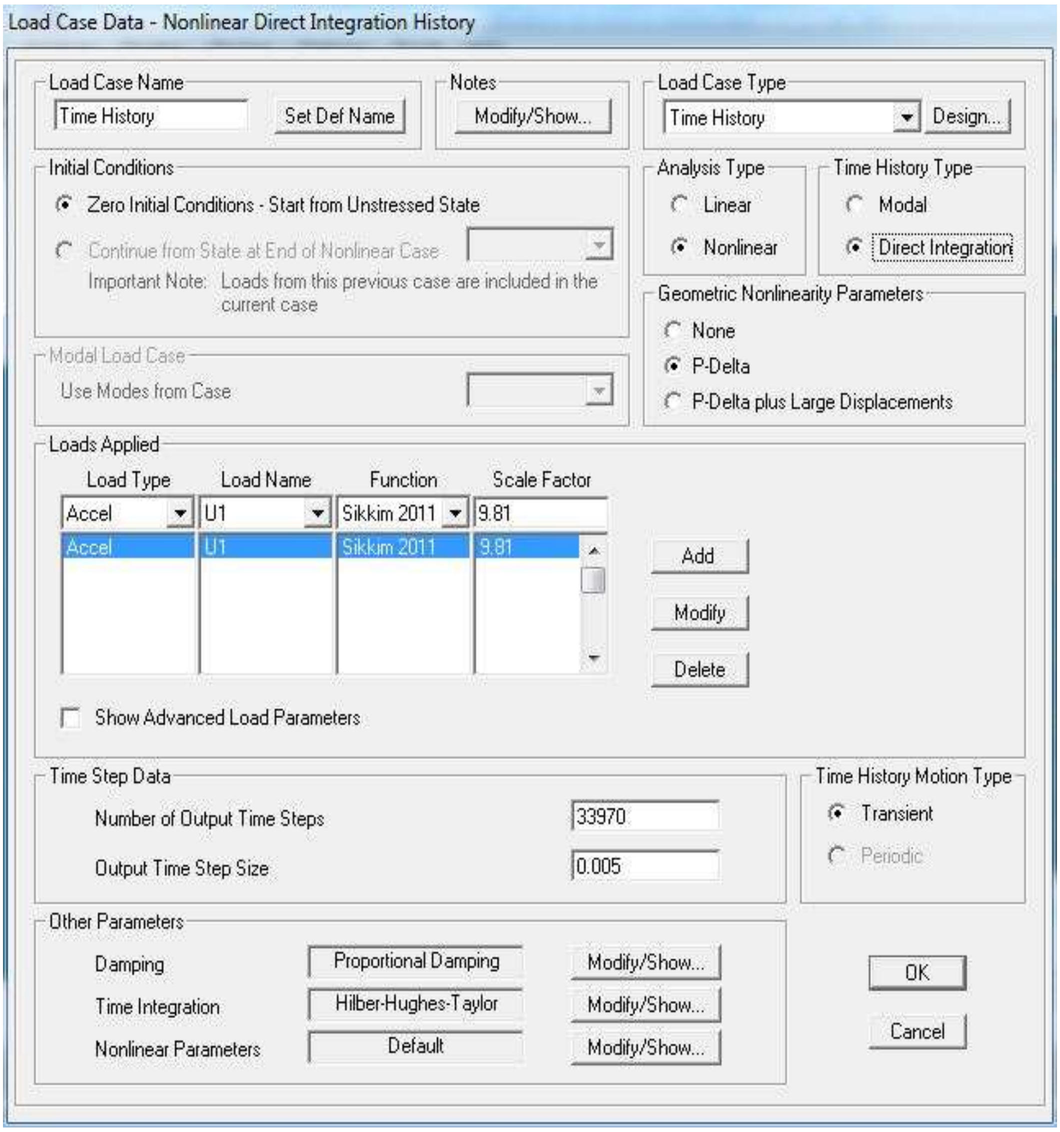


Fig 5.31 : Non Linear Direct Integration Time History data for Sikkim earthquake

Chapter VI

RESULT AND DISCUSSION

6.1 General

This section discusses the results obtained from the analysis phase and performance assessment phase, including floor accelerations, inter-story drifts, structural seismic performance levels, Time history analysis results of absolute acceleration are tabulated below.

A. 7 storied building frame (symmetrical plan)

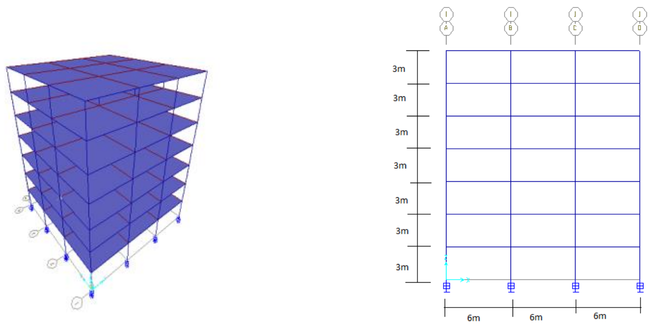


Fig. 6.1 Seven storied building symmetric frame

(I) The aforementioned structure has been analyzed with the Park field earthquake ground motion recorded on 28th June 1966 at 04:26.The seismic response of the structure is tabulated below

Table 6.1. Absolute Accleration of each floor.

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	1.489	0.076	94.89
2	6th floor level	1.436	0.070	95.12
3	5th floor level	1.347	0.063	95.32
4	4th floor level	1.230	0.056	95.44
5	3rd floor level	1.052	0.067	93.63
6	2nd Floor level	0.819	0.073	91.08
7	1st Floor level	0.571	0.078	86.34
8	Base level	0.475	0.078	83.58

Reduction of absolute acceleration of 7 storied symmetric building frameis 91% in average due to incorporating base isolation system for the park field earthquake ground motion. We can see from the above result that base acceleration is 83.58% reduced but the roof acceleration is reduced to above 94%.

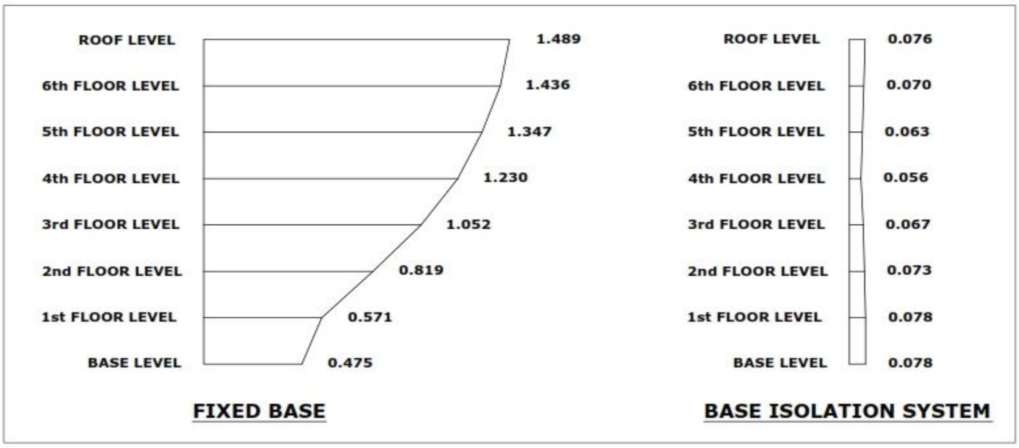
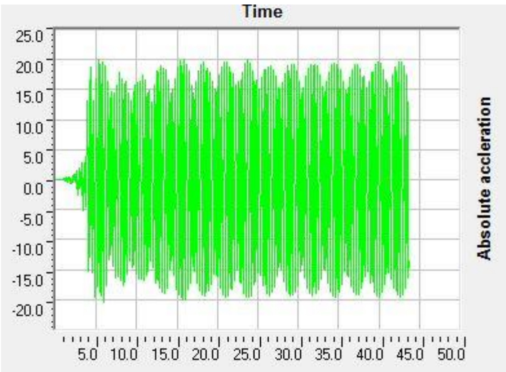


Fig. 6.2. Graphical representation of acceleration at each floor

It was also observed that the acceleration of the roof is magnified 1.489 g with respect to acceleration at the base i.e. 0.475g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.076 g which is lower than the acceleration above the isolator at the base level.



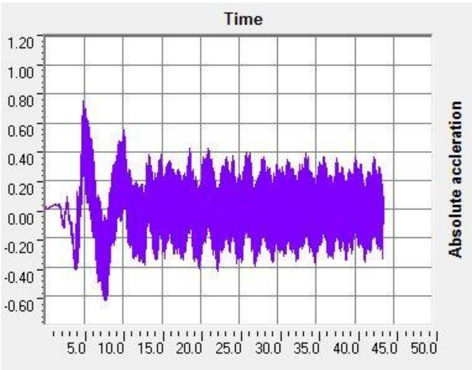


Fig. 6.3. Roof acceleration of Fixed base building (m/sec^2)

Fig. 6.4. Roof acceleration of isolated building (m/sec^2)

Figure 6.3. and 6.4. show the reduction of roof acceleration of the isolated building with respect to fixed base building.

(II) The aforementioned structure has been analyzed with the Imperial Valley earthquake ground motion recorded on 6th June 1938 at 02:42. The seismic response of the structure is tabulated below

Table 6.2. Absolute Accleration of each floor.

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	0.1351	0.00278	97.94
2	6th floor level	0.1320	0.00196	98.52
3	5th floor level	0.1218	0.00131	98.92
4	4th floor level	0.1118	0.00228	99.39
5	3rd floor level	0.1054	0.00321	96.95
6	2nd Floor level	0.0864	0.00298	96.56
7	1st Floor level	0.0398	0.00306	92.32
8	Base level	0.0118	0.00352	70.13

The reduction of absolute acceleration of 7 storied symmetric building frame is 93.84% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 70.13% reduced but the roof acceleration is reduced to above 97.94%.

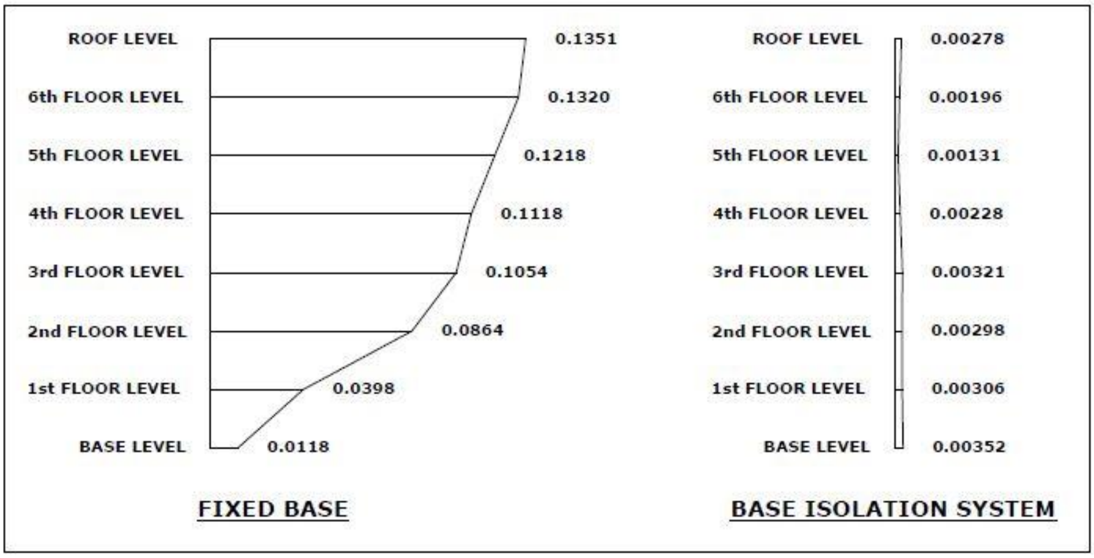


Fig. 6.5 Graphical representation of acceleration at each floor

It was also observed that the acceleration of the roof is magnified 0.1351 g with respect to acceleration at the base i.e. 0.0118g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.00278 g which is lower than the acceleration above the isolator at the base level.

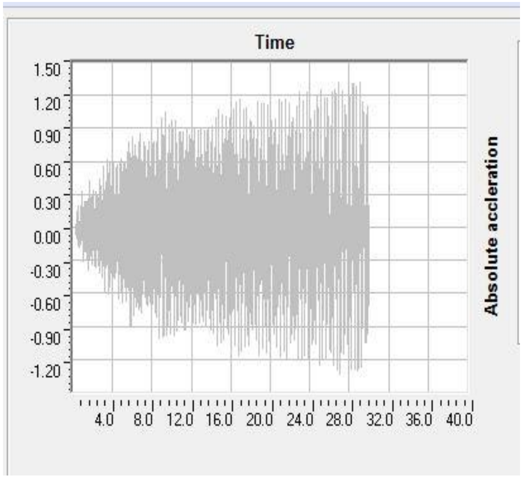


Fig. 6.6. Roof acceleration of Fixed base building (m/sec^2)

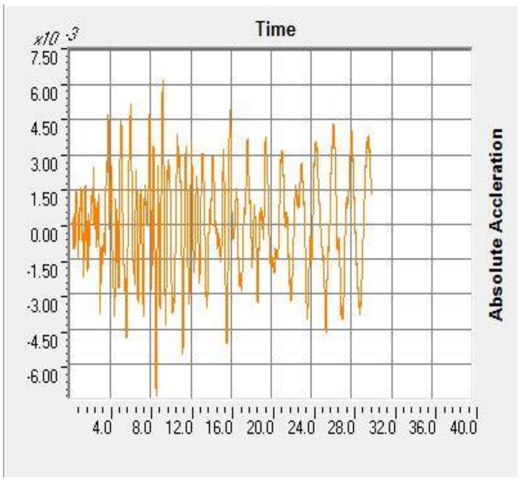


Fig. 6.7. Roof acceleration of isolated building (m/sec^2)

From the above figure 6.6 and 6.7, it was observed that the roof acceleration reduced due to incorporation of base isolation.

(III) The aforementioned structure has been analyzed with Sikkim earthquake ground motion recorded on 18th September 2011 at 18:10. The seismic response of the structure is tabulated below

Table 6.3 Absolute accleration of each floor

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	1.4099	0.10285	92.70
2	6th floor level	1.1163	0.07913	92.91
3	5th floor level	1.0890	0.06055	94.44
4	4th floor level	0.8840	0.06261	92.92
5	3rd floor level	0.8471	0.06382	92.47
6	2nd Floor level	0.7586	0.07143	90.58
7	1st Floor level	0.3634	0.09528	73.78
8	Base level	0.2056	0.15336	42.95

Reduction of absolute acceleration of 7 storied symmetric building frameis 84.09% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 42.95% reduced but the roof acceleration is reduced to above 92.70%.

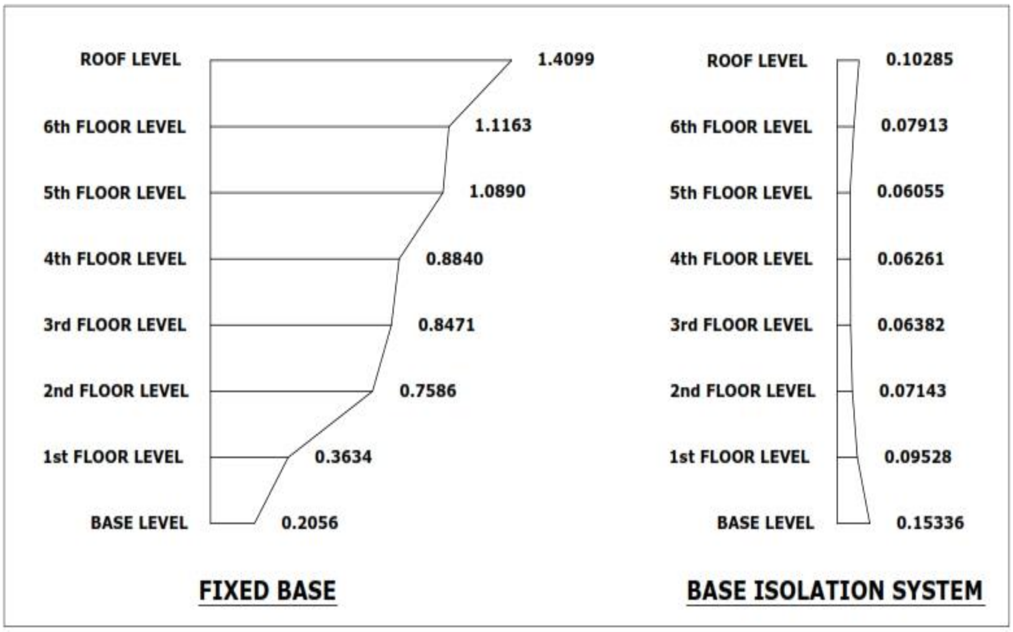
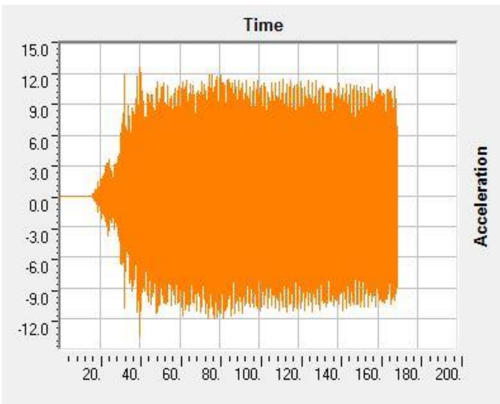


Fig. 6.8 Graphical representation of acceleration at each floor

It was also observed that the acceleration of the roof is magnified 1.4099 g with respect to acceleration at the base i.e. 0.2056g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.10285 g which is lower than the acceleration above the isolator at the base level



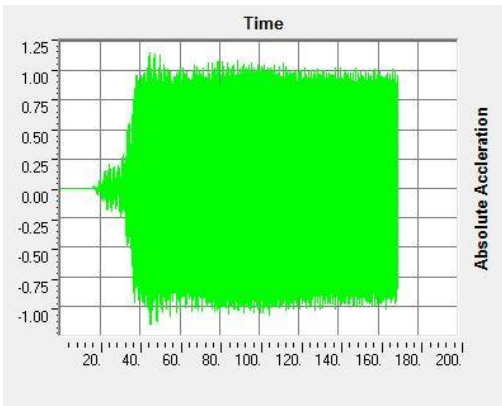


Fig. 6.9 Roof acceleration of Fixed base building (m/sec^2)

Fig. 6.10 Roof acceleration of isolated building (m/sec^2)

Figure 6.9 and 6.10 show the reduction of roof acceleration of the isolated building with respect to fixed base building.

B. 4 storied asymmetric building frame (asymmetrical plan)

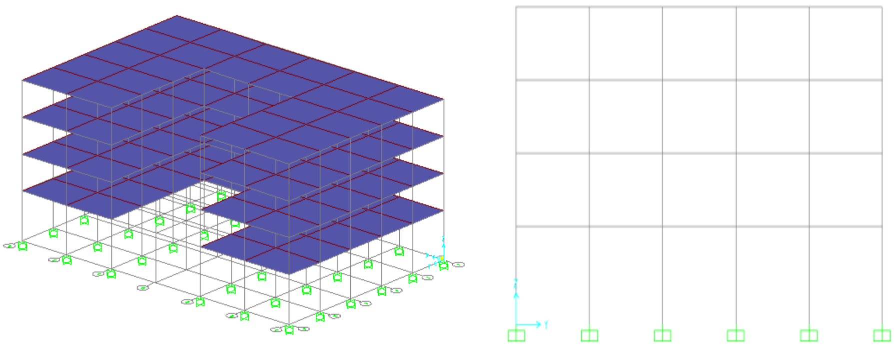


Fig. 6.11

(I) The aforementioned structure has been analyzed with the Park field earthquake ground motion recorded on 28th June 1966 at 04:26.The seismic response of the structure is tabulated below

Table 6.4 Absolute Accleration of each floor

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction in absolute
		Fixed Base	Base Isolation System	
1	Roof Level	0.6891	0.1148	83.34
2	3rd floor level	0.6519	0.1131	82.66
3	2nd Floor level	0.5829	0.1126	80.68
4	1st Floor level	0.5054	0.1133	77.57
5	Base level	0.475	0.1144	75.94

Reduction of absolute acceleration of 4 storied asymmetric building frameis 80.04% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 75.94% reduced but the roof acceleration is reduced to above 83.34%.

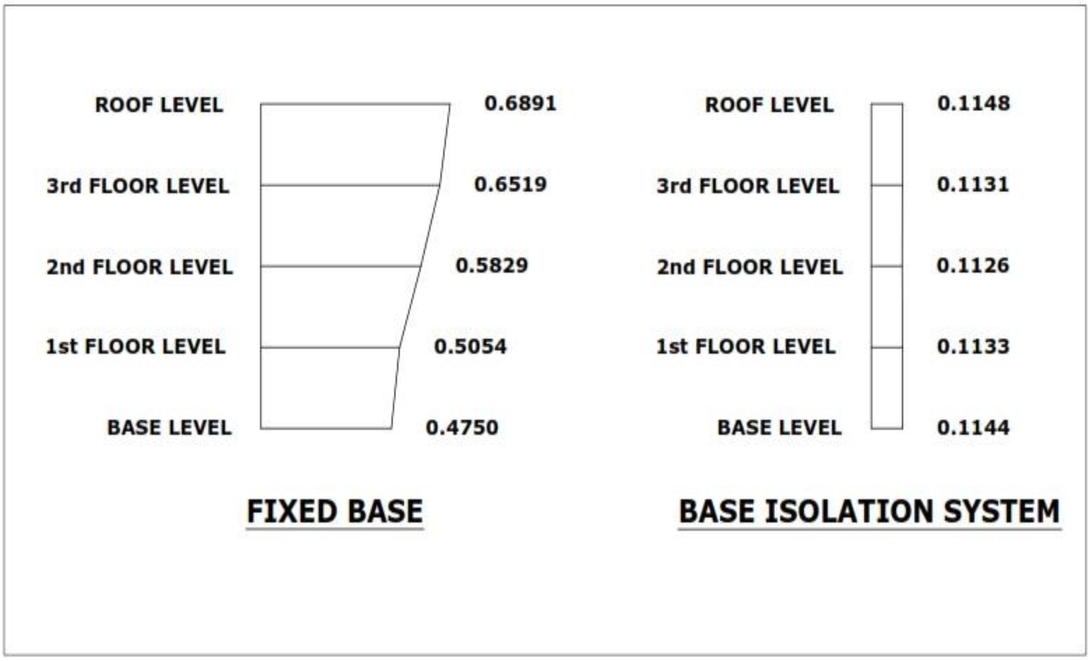
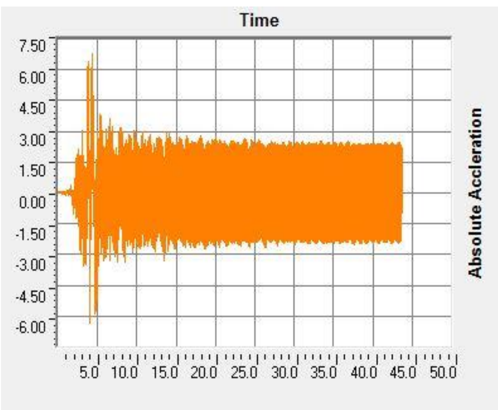


Fig. 6.12 Graphical representation of acceleration at each floor.

It was also observed that the acceleration of the roof is magnified 0.6891 g with respect to acceleration at the base i.e. 0.4750g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.1148 g which is same as the acceleration above the isolator at the base level.



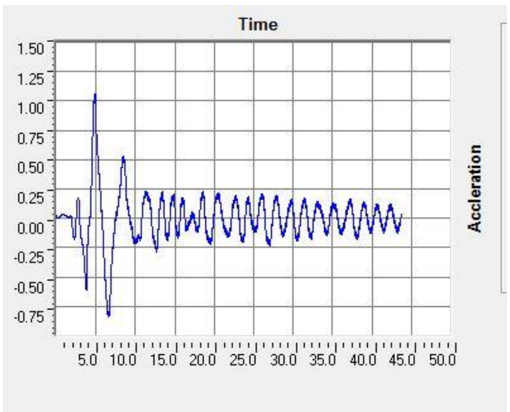


Fig. 6.13 Roof acceleration of Fixed base building (m/sec^2)

Fig. 6.14 Roof acceleration of isolated building (m/sec^2)

From the above figure 6.13 and 6.14, it was observed that the roof acceleration has reduced due to incorporation of base isolation.

(II) The aforementioned structure has been analyzed with the Imperial Valley earthquake ground motion recorded on 6th June 1938 at 02:42. . The seismic response of the structure is tabulated below

Table 6.5 Absolute Accleration of each floor

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	0.0699	0.0074	98.93
2	3rd floor level	0.0617	0.0073	98.81
3	2nd Floor level	0.0488	0.0071	98.54
4	1st Floor level	0.0295	0.0068	97.68
5	Base level	0.0118	0.0065	94.46

Reduction of absolute acceleration of 4 storied asymmetric building frameis 84.09% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 42.95% reduced but the roof acceleration is reduced to above 92.70%.

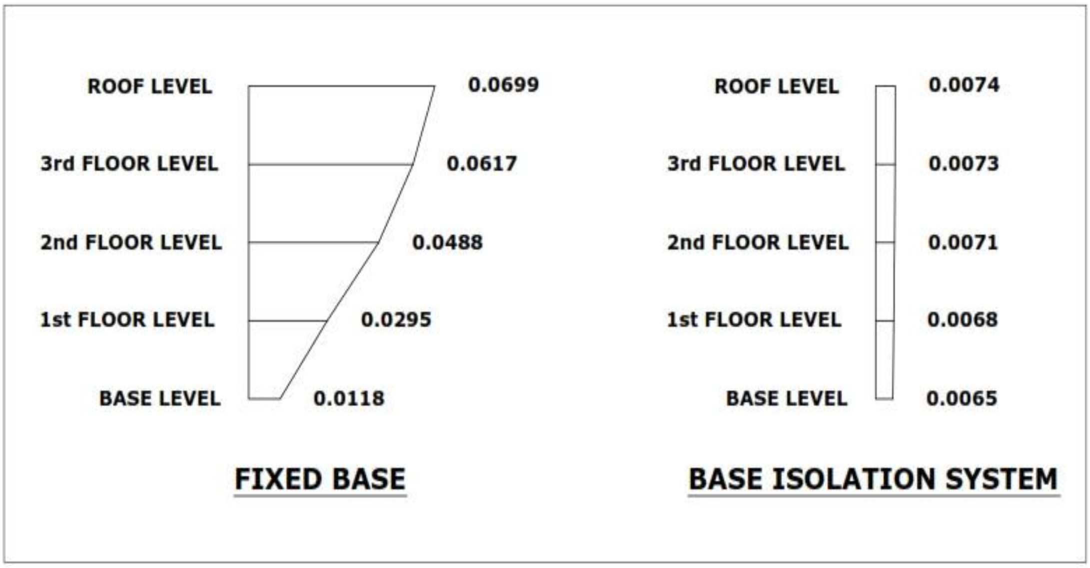
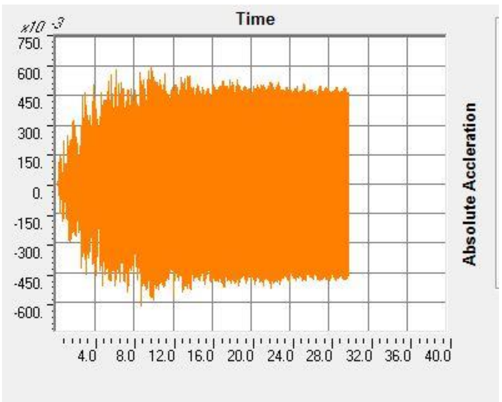


Fig. 6.15 Graphical representation of acceleration at each floor

It was also observed that the acceleration of the roof is magnified 0.0699 g with respect to acceleration at the base i.e. 0.0118g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.0074 g which is same as the acceleration above the isolator at the base level.



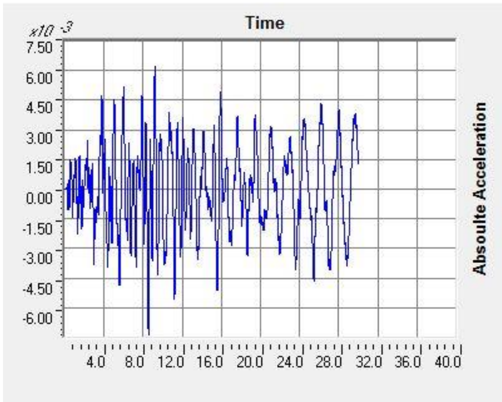


Fig. 6.16 Roof acceleration of Fixed base building (m/sec^2)

Fig. 6.17 Roof acceleration of isolated building (m/sec^2)

Figure 6.16 and 6.17 shows the reduction of roof acceleration of the isolated building with respect to fixed base building.

(III) The aforementioned structure has been analyzed with Sikkim earthquake ground motion recorded on 18th September 2011 at 18:10.The seismic response of the structure is tabulated below

Table 6.6 Absolute accleration of each floor

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	0.6186	0.02186	96.47
2	3rd floor level	0.5575	0.02084	96.26
3	2nd Floor level	0.4969	0.00210	99.58
4	1st Floor level	0.3209	0.02161	93.27
5	Base level	0.2056	0.02247	89.07

Reduction of absolute acceleration of 4 storied asymmetric building frameis 94.93% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 89.07% reduced but the roof acceleration is reduced to above 96.47%.

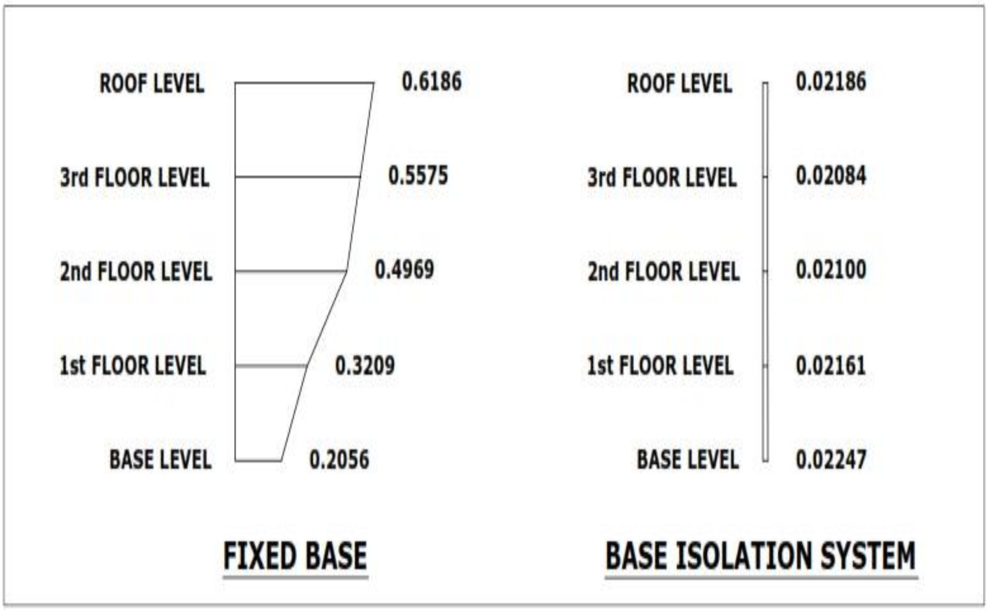


Fig. 6.18 Graphical representation of acceleration at each floor

It was also observed that the acceleration of the roof is magnified 0.6186 g with respect to acceleration at the base i.e. 0.2056g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.02186 g which is same as the acceleration above the isolator at the base level.

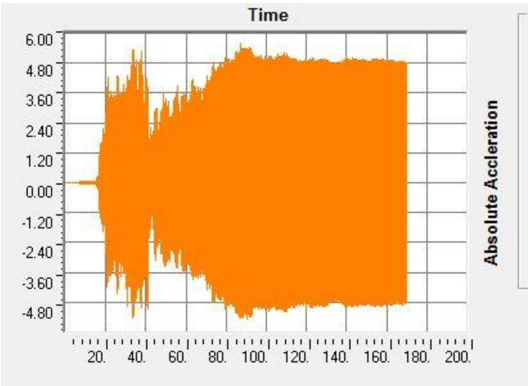


Fig. 6.19 Roof acceleration of Fixed base building (m/sec^2)

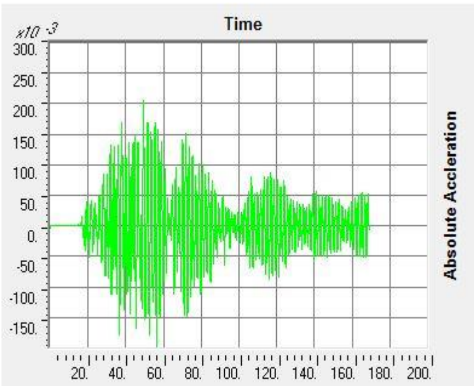


Fig. 6.20 Roof acceleration of isolated building (m/sec^2)

Figure 6.18 and 6.3.,. show the reduction of roof acceleration of the isolated building with respect to fixed base building.

C. 7 storied building frame (Asymmetrical plan)

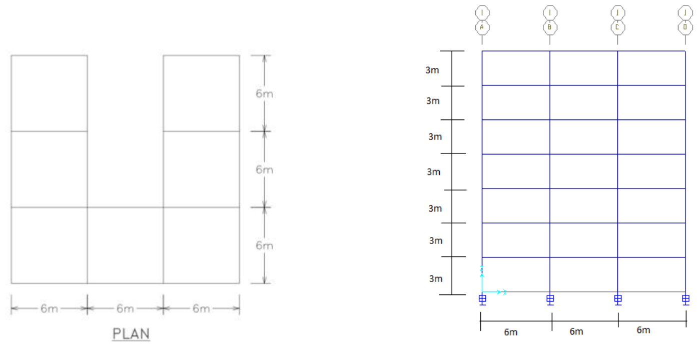


Fig. 6.21 Seven storied Asymmetrical building frame

(I) The aforementioned structure has been analyzed with the Park field earthquake ground motion recorded on 28th June 1966 at 04:26.The seismic response of the structure is tabulated below

Table 6.7 Absolute accleration of each floor.

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	2.6527	0.08581	96.77
2	6th floor level	2.3506	0.07664	96.74
3	5th floor level	1.8963	0.06758	96.44
4	4th floor level	1.7559	0.06249	96.44
5	3rd floor level	1.4927	0.06627	95.56
6	2nd Floor level	1.1243	0.07268	93.54
7	1st Floor level	0.6976	0.06351	90.90
8	Base level	0.4758	0.07955	83.28

Reduction of absolute acceleration of 7 storied symmetric building frame is 93.71 % in average due to incorporating base isolation system for the park field earthquake ground motion. We can see from the above result that base acceleration is 83.28% reduced but the roof acceleration is reduced to above 96%.It was also observed that the acceleration of the roof is magnified 2.6527 g with respect to acceleration at the base i.e. 0.475g of the conventional fixed base frame. Whereas;

for base isolated structure, the acceleration at the roof is 0.085 g which is approximately same as acceleration above the isolator at the base level.

(II) The aforementioned structure has been analyzed with the Imperial Valley earthquake ground motion recorded on 6th June 1938 at 02:42. .The seismic response of the structure is tabulated below

Table 6.8 Absolute accleration of each floor.

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	0.0865	0.00267	96.92
2	6th floor level	0.0650	0.00212	96.74
3	5th floor level	0.0639	0.00127	98.02
4	4th floor level	0.0618	0.00084	98.63
5	3rd floor level	0.0565	0.00099	98.24
6	2nd Floor level	0.0418	0.00177	95.75
7	1st Floor level	0.0222	0.00235	89.39
8	Base level	0.0117	0.00277	76.36

The reduction of absolute acceleration of 7 storied symmetric building frame is 93.76% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 76.36% reduced but the roof acceleration is reduced to above 96.92%.

It was also observed that the acceleration of the roof is magnified 0.0865 g with respect to acceleration at the base i.e. 0.0117g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.00267 g which is lower than the acceleration above the isolator at the base level.

(III) The aforementioned structure has been analyzed with Sikkim earthquake ground motion recorded on 18th September 2011 at 18:10. The seismic response of the structure is tabulated below

Table 6.9 Absolute accleration of each floor.

Sl. no.	LEVEL	Absolute acceleration (in terms of g)		Percentage reduction
		Fixed Base	Base Isolation System	
1	Roof Level	2.2648	0.06972	96.92
2	6th floor level	1.6354	0.05298	96.76
3	5th floor level	1.6256	0.03526	97.83
4	4th floor level	1.5416	0.01657	98.92
5	3rd floor level	1.5325	0.04003	97.39
6	2nd Floor level	1.3177	0.04851	96.32
7	1st Floor level	0.6342	0.06373	89.95
8	Base level	0.2056	0.07389	64.05

Reduction of absolute acceleration of 7 storied symmetric building frame is 92.97% in average due to incorporating base isolation system for the imperial earthquake ground motion. We can see from the above result that base acceleration is 64.05% reduced but the roof acceleration is reduced to above 96.92%.

It was also observed that the acceleration of the roof is magnified 2.2648 g with respect to acceleration at the base i.e. 0.2056g of the conventional fixed base frame. Whereas; for base isolated structure, the acceleration at the roof is 0.06972 g which is lower than the acceleration above the isolator at the base level.

Time history analysis results of Inter Story drifts are tabulated below.

A. 7 storied building frame (Symmetric plan)

(I) The aforementioned structure has been analyzed with the Park field earthquake ground motion recorded on 28th June 1966 at 04:26. Inter storey drifts of the structure is tabulated below

Table 6.10 Inter storey drift of 7 storied symmetrical building at each floor.

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-6th floor level	0.3	0.1
2	6th floor level-5th floor level	3.1	0.3
3	5th floor level-4th floor level	2.2	0.1
4	4th floor level-3rd floor level	9	0.6
5	3rd floor level-2nd Floor level	10	0.1
6	2nd Floor level-1st Floor level	7	0.1
7	1st Floor level- Base level	3	1.1

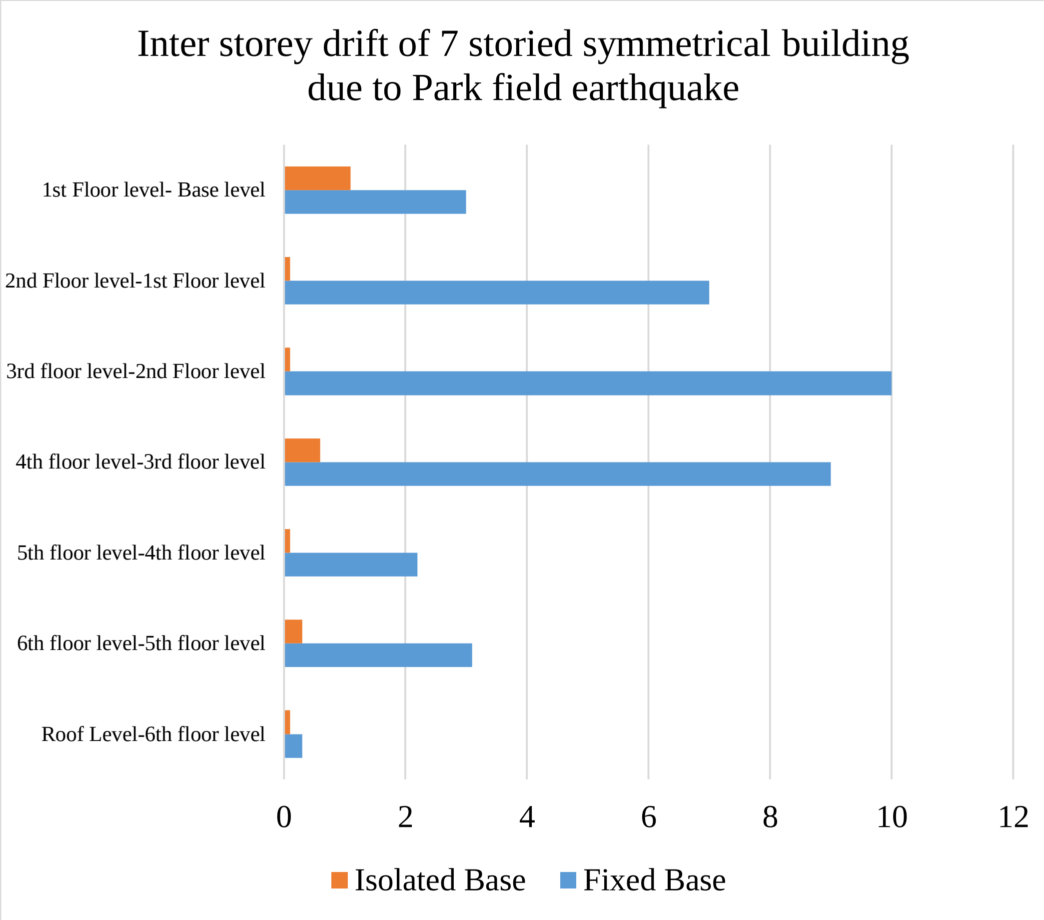


Fig. 6.22 Graphical representation of inter storey drift at each floor

(II) The aforementioned structure has been analyzed with the Imperial Valley earthquake ground motion recorded on 6th June 1938 at 02:42. . The inter storey drift of the structure is tabulated below

Table 6.11 Inter storey drift of 7 storied symmetrical at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-6th floor level	0.2	0.002
2	6th floor level-5th floor level	0.3	0.005
3	5th floor level-4th floor level	0.1	0.001
4	4th floor level-3rd floor level	0.3	0.01
5	3rd floor level-2nd Floor level	0.3	0.008
6	2nd Floor level-1st Floor level	0.4	0.001
7	1st Floor level- Base level	0.2	0.011

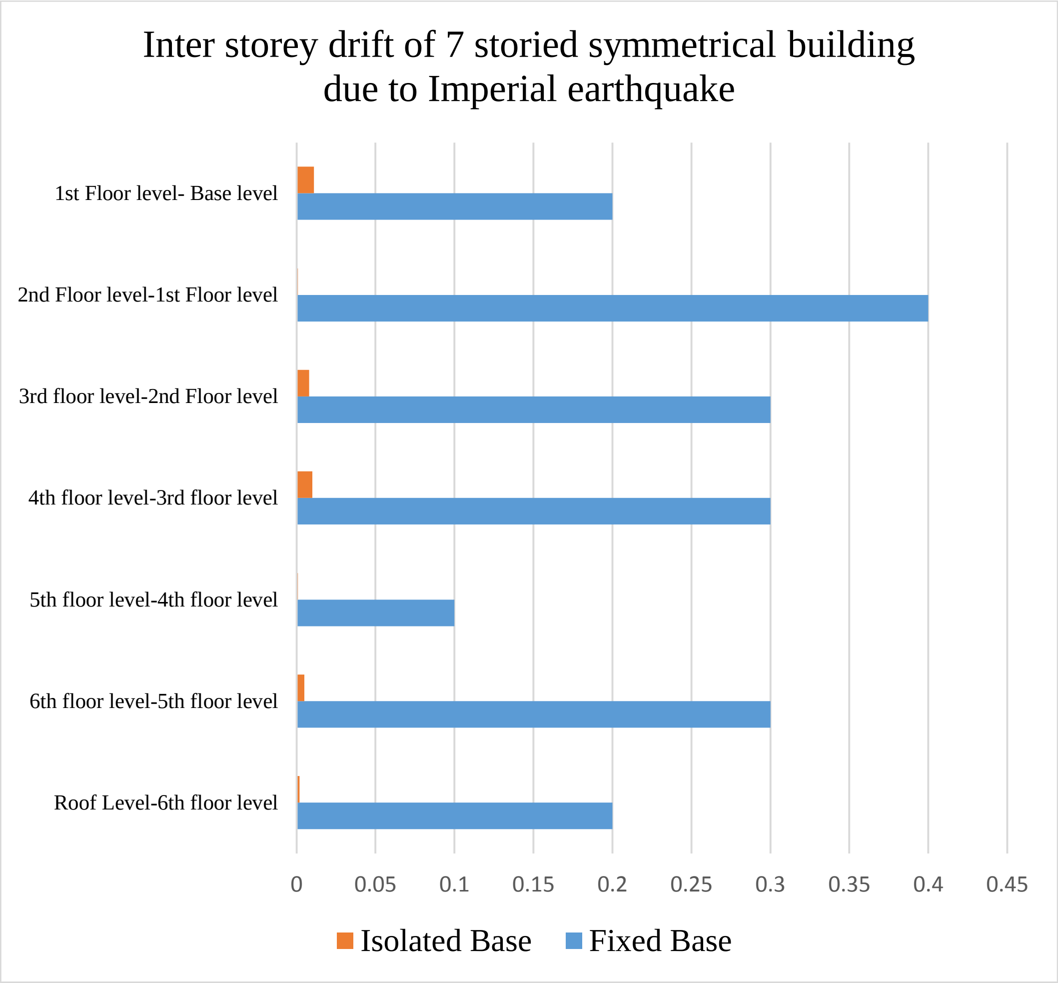


Fig. 6.23 Graphical representation of inter storey drift (mm) at each floor

(III) The aforementioned structure has been analyzed with Sikkim earthquake ground motion recorded on 18th September 2011 at 18:10. .Inter storey drift of the structure is tabulated below

Table 6.12 Inter storey drift drift of 7 storied symmetrical at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-6th floor level	1.4	0.2
2	6th floor level-5th floor level	2.2	0.3
3	5th floor level-4th floor level	3.6	0.4
4	4th floor level-3rd floor level	3.6	0.2
5	3rd floor level-2nd Floor level	5	0
6	2nd Floor level-1st Floor level	5.3	0
7	1st Floor level- Base level	3.2	0

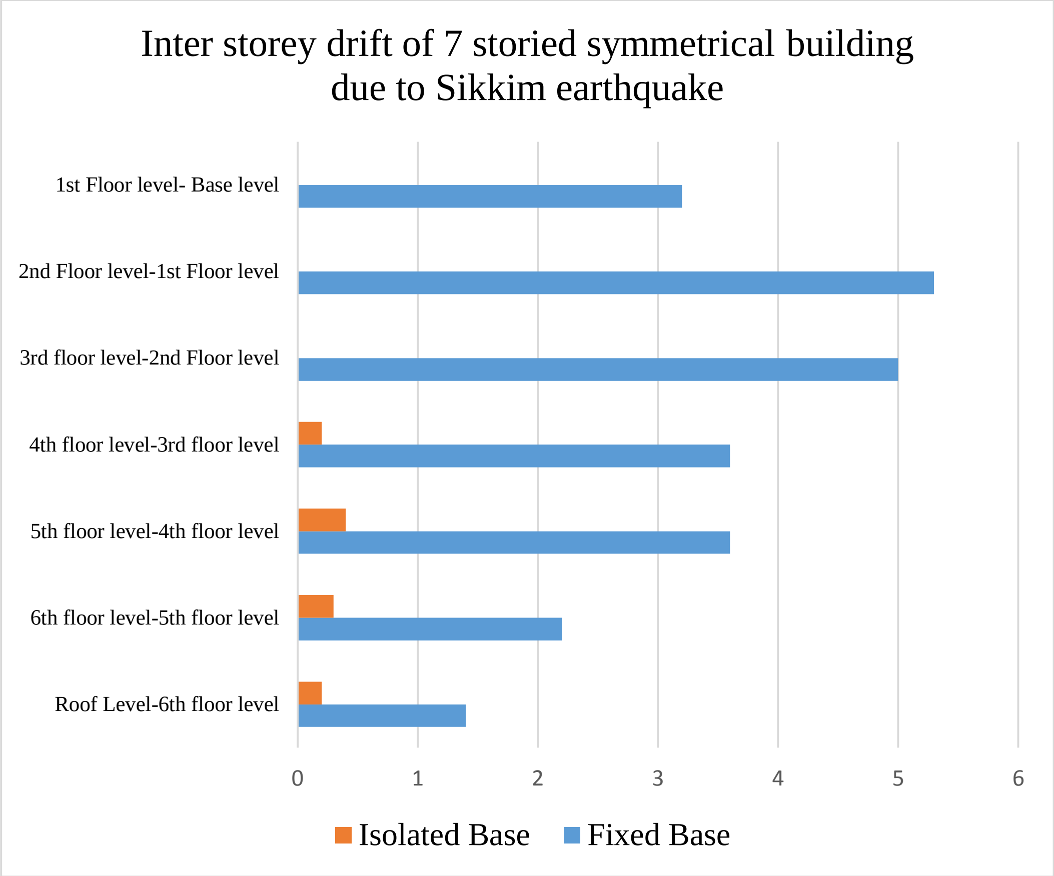


Fig. 6.24 Graphical representation of inter storey drift (mm) at each floor

B. 4 storied asymmetric building frame (Asymmetric Plan)

(I) The aforementioned structure has been analyzed with the Park field earthquake ground motion recorded on 28th June 1966 at 04:26. Inter storey drifts of the structure is tabulated below

Table 6.13 Inter storey drift of 4 storied asymmetric building at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-3rd floor level	0.2	0.1
2	3rd floor level-2nd Floor level	0.3	0.1
3	2nd Floor level-1st Floor level	0.3	0.1
4	1st Floor level- Base level	0.3	0.2

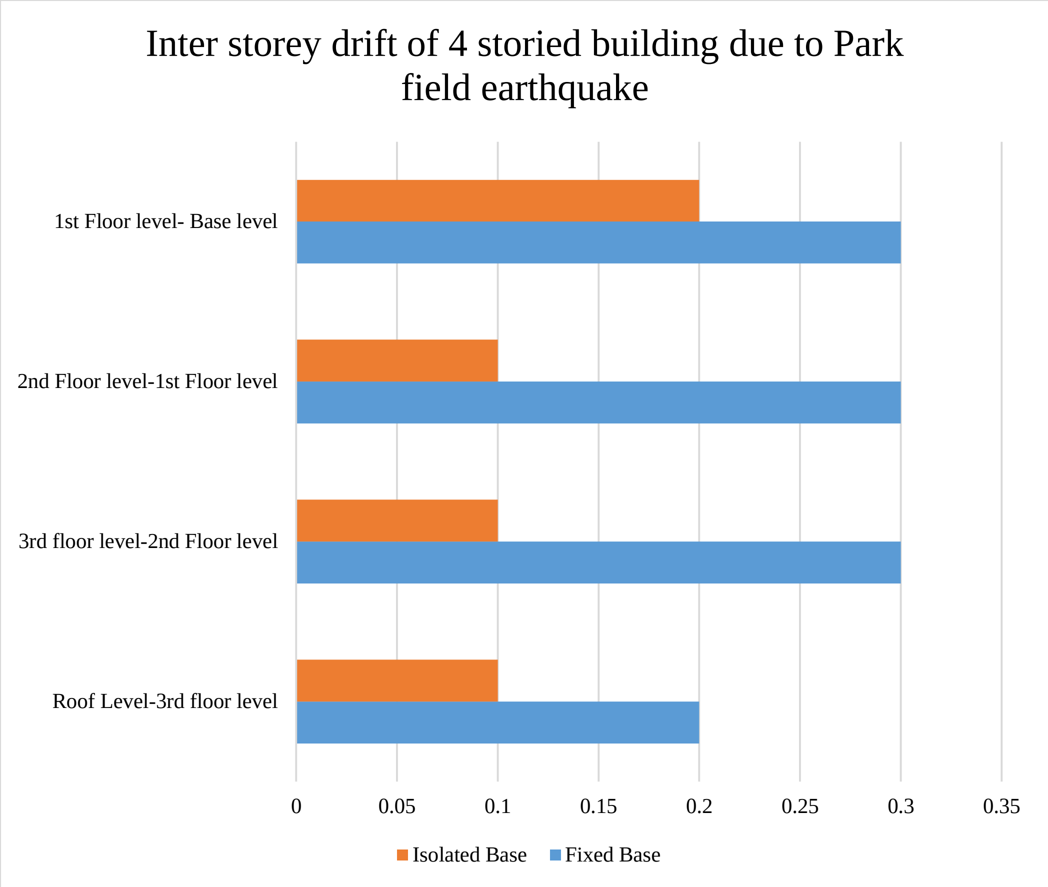


Fig. 6.25 Graphical representation of inter storey drift (mm) at each floor

(II) The aforementioned structure has been analyzed with the Imperial Valley earthquake ground motion recorded on 6th June 1938 at 02:42. . The inter storey drift of the structure is tabulated below.

Table 6.14 Inter storey drift of 4 storied asymmetric building at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-3rd floor level	0.07	0.001
2	3rd floor level-2nd Floor level	0.03	0
3	2nd Floor level-1st Floor level	0.1	0
4	1st Floor level- Base level	0.01	0.001

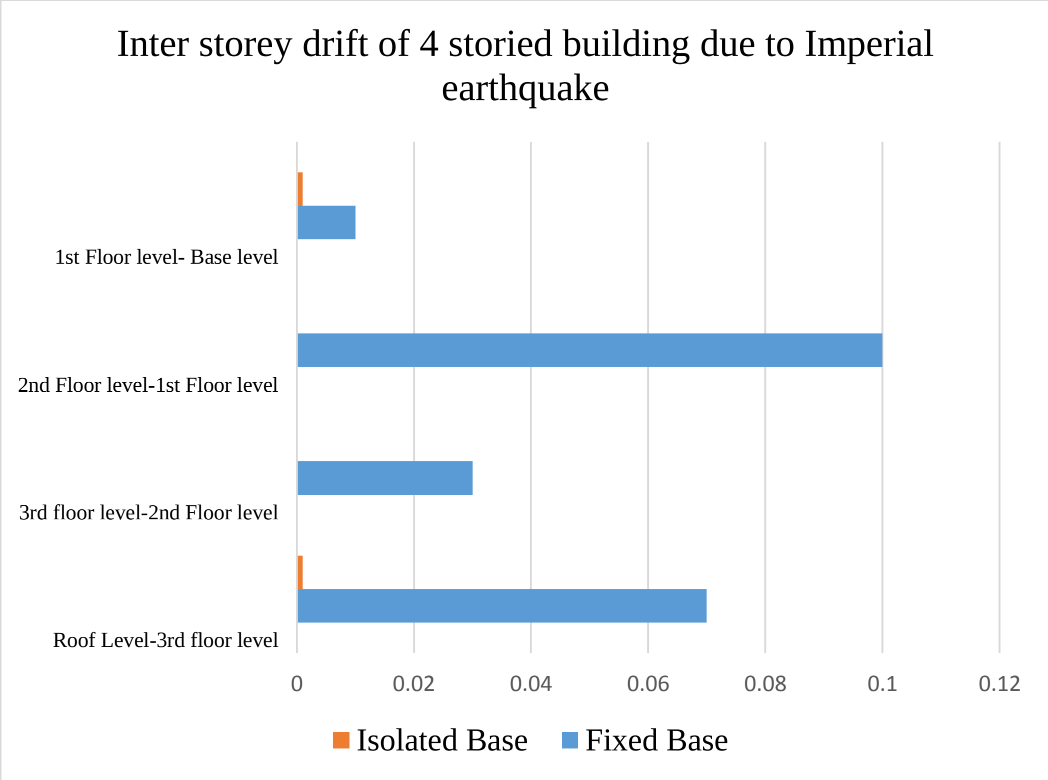


Fig. 6.26 Graphical representation of inter storey drift (mm) at each floor

(III) The aforementioned structure has been analyzed with Sikkim earthquake ground motion recorded on 18th September 2011 at 18:10. Inter storey drift of the structure is tabulated below

Table 6.15 Inter storey drift of 4 storied asymmetric building at each floor.

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-3rd floor level	0.8	0.048
2	3rd floor level-2nd Floor level	0.8	0.021
3	2nd Floor level-1st Floor level	0.9	0.161
4	1st Floor level- Base level	1.4	0.01

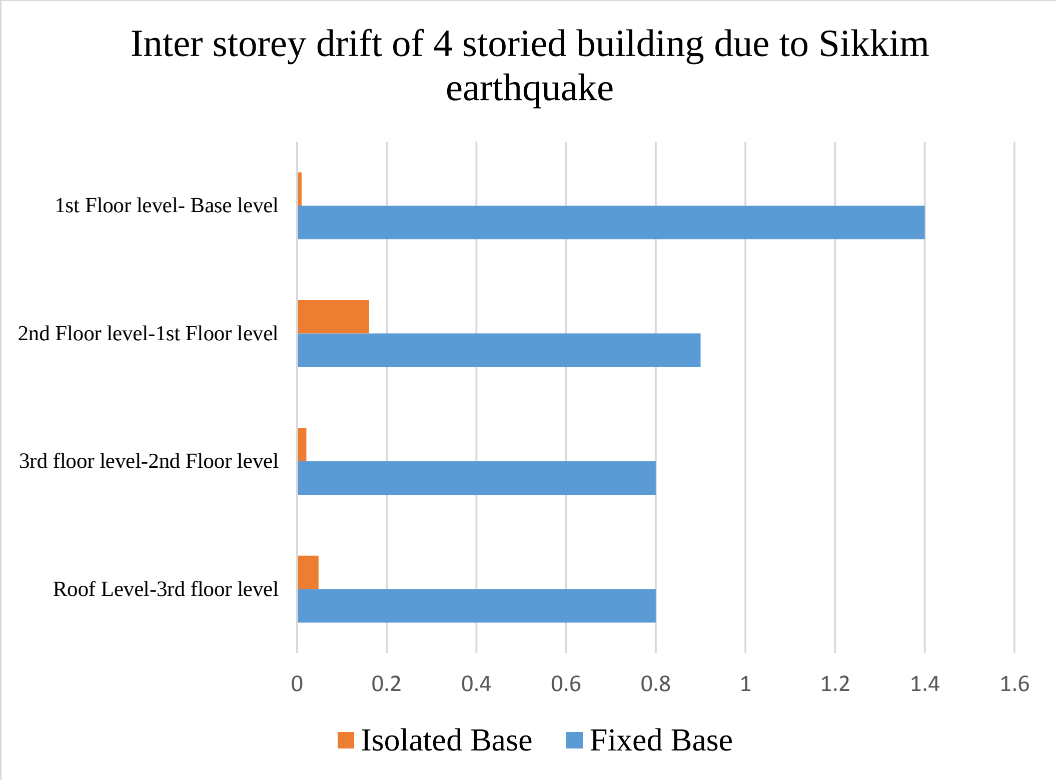


Fig. 6.27 Graphical representation of inter storey drift (mm) at each floor

From the above result it was observed that the inter story drift is significantly reduced due to incorporation of base isolation system. In symmetrical building reduction of inter-story drift is more than the asymmetric building.

C. 7 storied building frame (Asymmetric plan)

(I) The aforementioned structure has been analyzed with the Park field earthquake ground motion recorded on 28th June 1966 at 04:26. Inter storey drifts of the structure is tabulated below

Table 6.16 Inter storey drift of 7 storied Asymmetric building at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-6th floor level	6.536	0.272
2	6th floor level-5th floor level	9.863	0.326
3	5th floor level-4th floor level	12.86	0.402
4	4th floor level-3rd floor level	14.501	0.593
5	3rd floor level-2nd Floor level	15.509	0.505
6	2nd Floor level-1st Floor level	13.51	0.752
7	1st Floor level- Base level	8.57	0.995

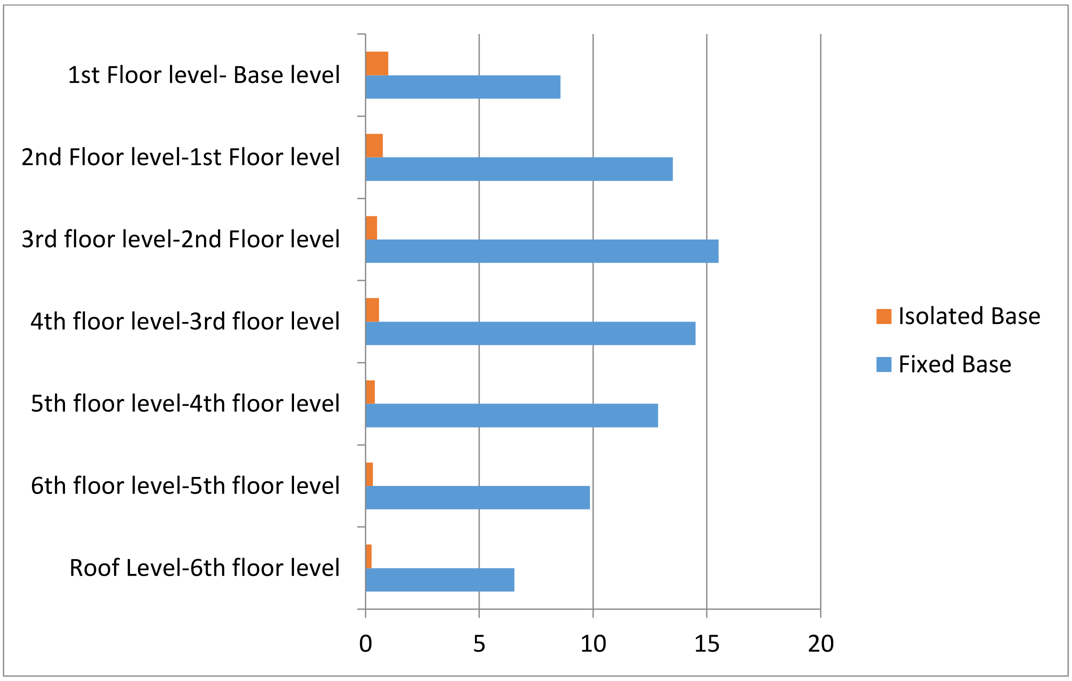


Fig. 6.28 Graphical representation of inter storey drift 7 storied assymetric buildingfor park field earthquake at each floor

(II) The aforementioned structure has been analyzed with the Imperial Valley earthquake ground motion recorded on 6th June 1938 at 02:42. . The inter storey drift of the structure is tabulated below

Table 6.17 Inter storey drift of 7 storied Asymmetric building at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-6th floor level	0.119	0.01
2	6th floor level-5th floor level	0.182	0.01
3	5th floor level-4th floor level	0.255	0.04
4	4th floor level-3rd floor level	0.407	0.05
5	3rd floor level-2nd Floor level	0.352	0.03
6	2nd Floor level-1st Floor level	0.411	0
7	1st Floor level- Base level	0.244	0.02

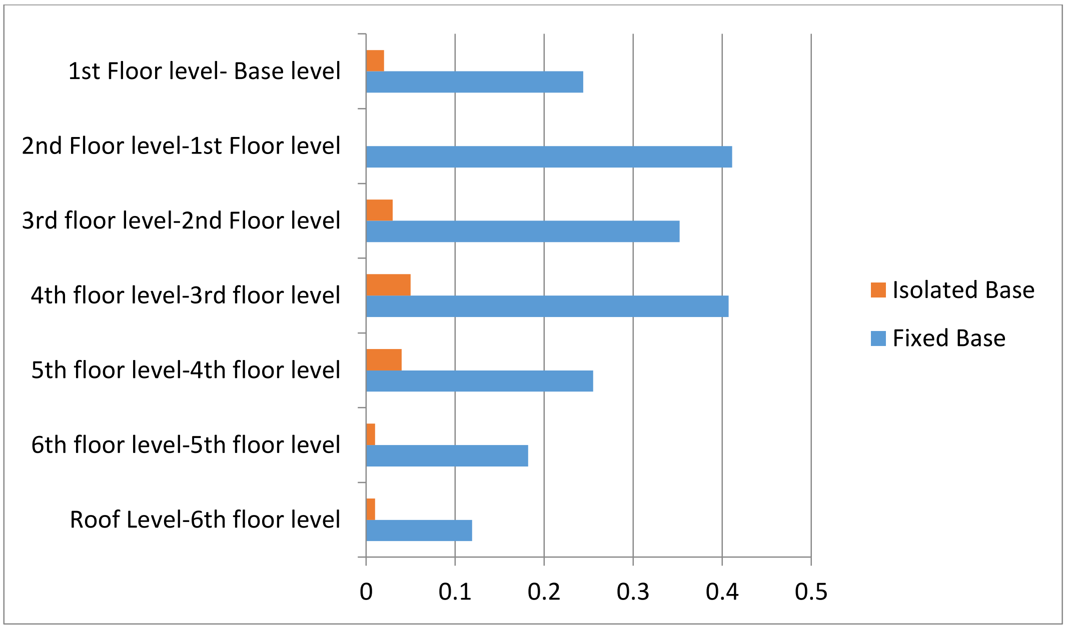


Fig. 6.29 Graphical representation of inter storey drift (mm) at each floor

(III) The aforementioned structure has been analyzed with Sikkim earthquake ground motion recorded on 18th September 2011 at 18:10. .Inter storey drift of the structure is tabulated below

Table 6.18 Inter storey drift of 7 storied Asymmetric building at each floor

Sl. no.	LEVEL	Inter Story Drift (mm)	
		Fixed Base	Base Isolation System
1	Roof Level-6th floor level	4.117	0.06
2	6th floor level-5th floor level	6.15	0.04
3	5th floor level-4th floor level	7.75	0.54
4	4th floor level-3rd floor level	6.905	0.07
5	3rd floor level-2nd Floor level	8.358	0.27
6	2nd Floor level-1st Floor level	8.436	0.26
7	1st Floor level- Base level	4.564	0.36

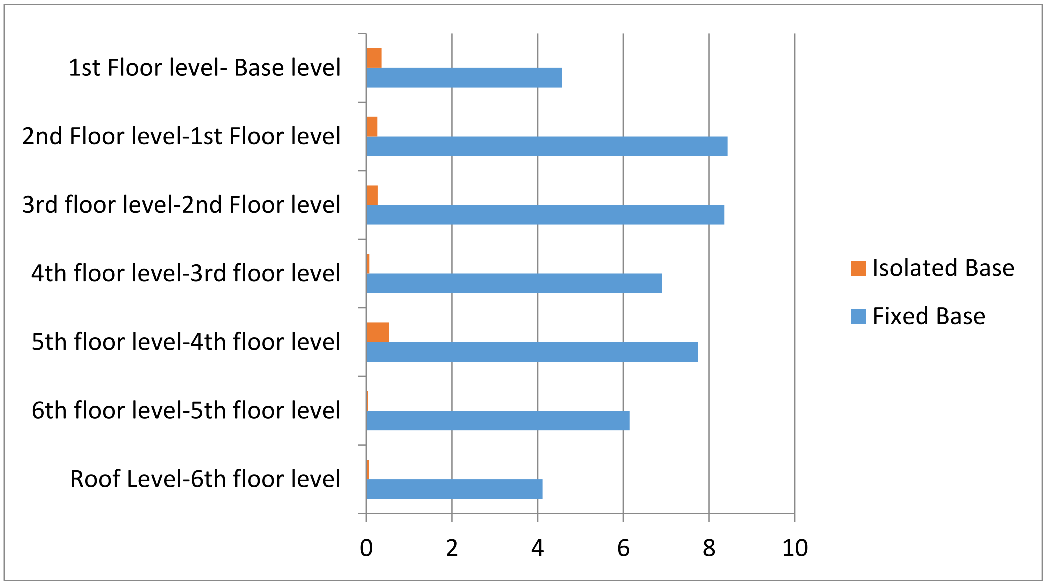


Fig. 6.30 Graphical representation of inter storey drift (mm) at each floor

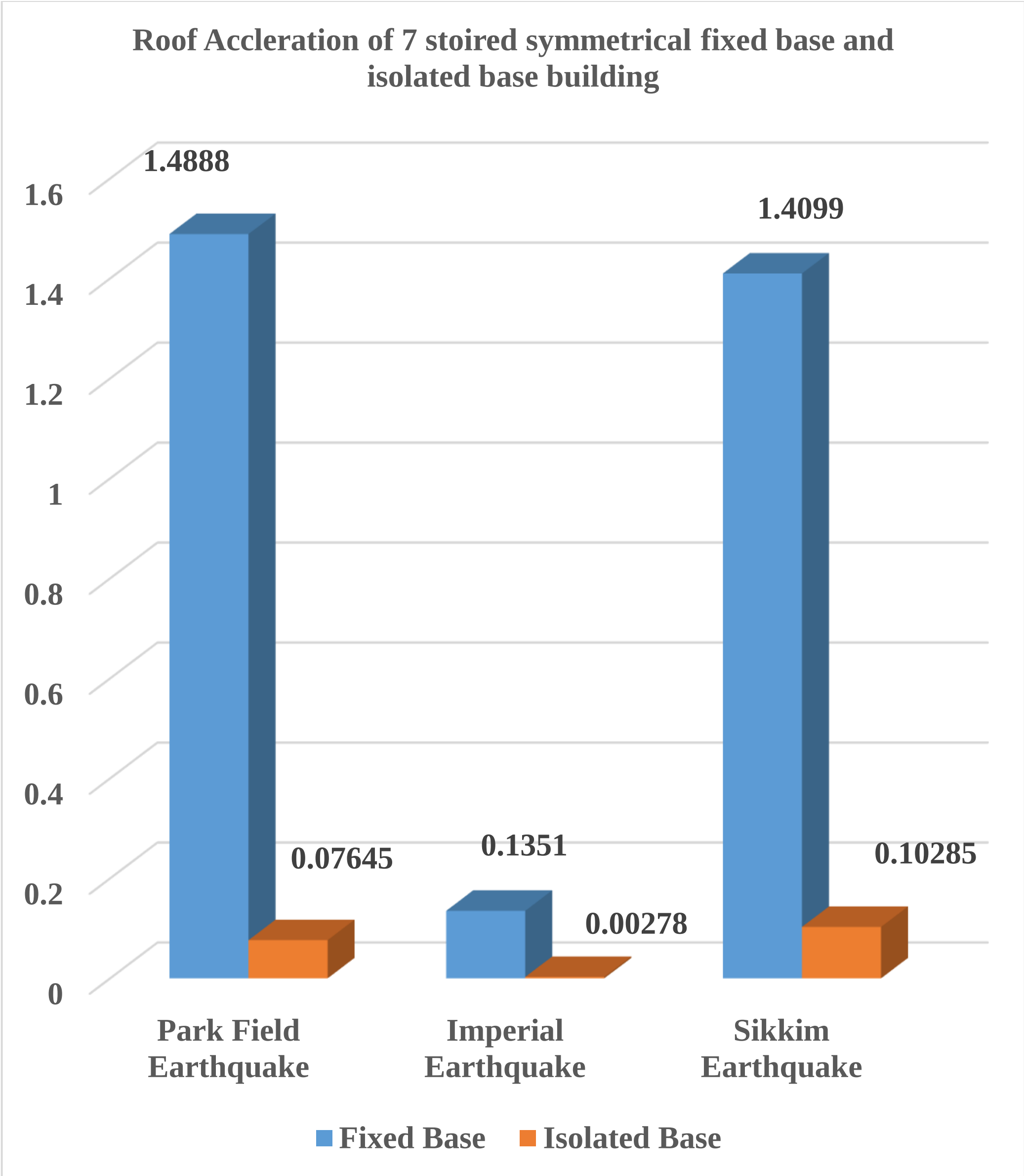


Fig. 6.31 Roof Accleration (in terms of g) of 7 stoired symmetrical fixed base and isolated base building

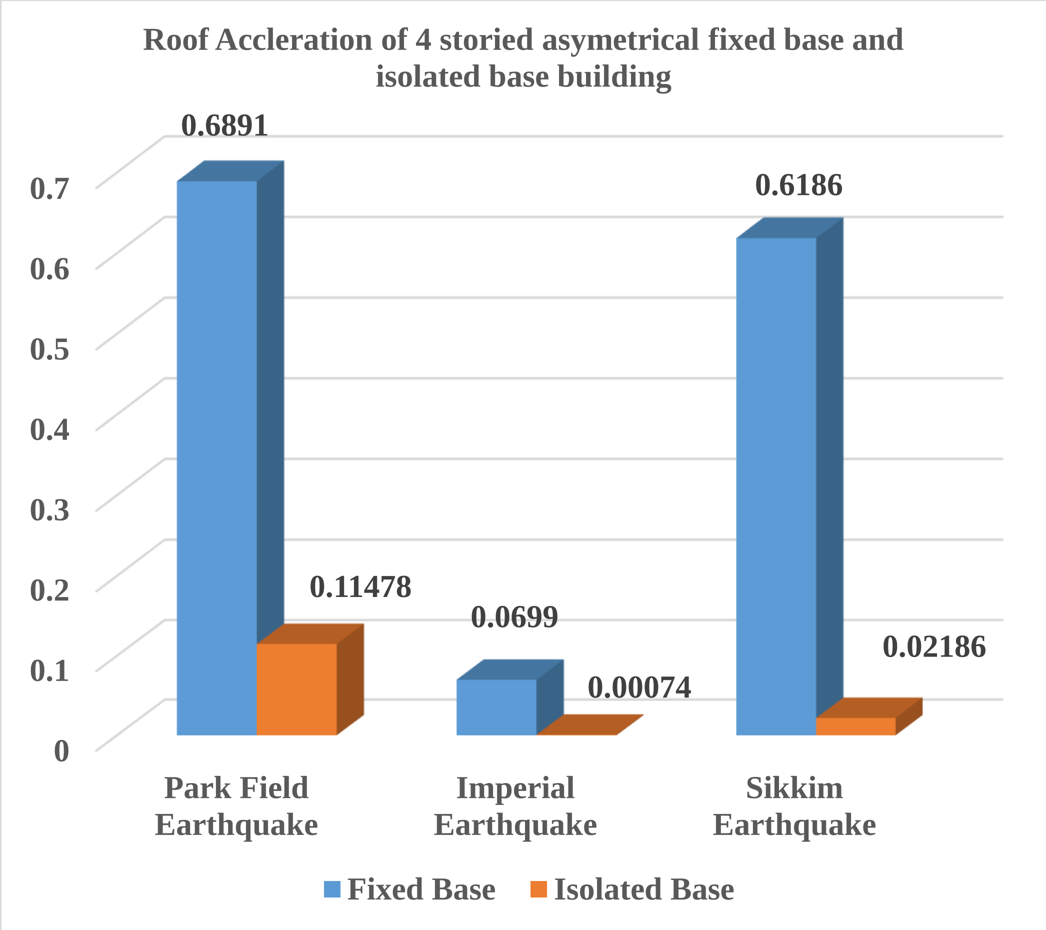


Fig. 6.32 Roof Accleration (in terms of g) of 4 storied asymmetrical fixed base and isolated base building

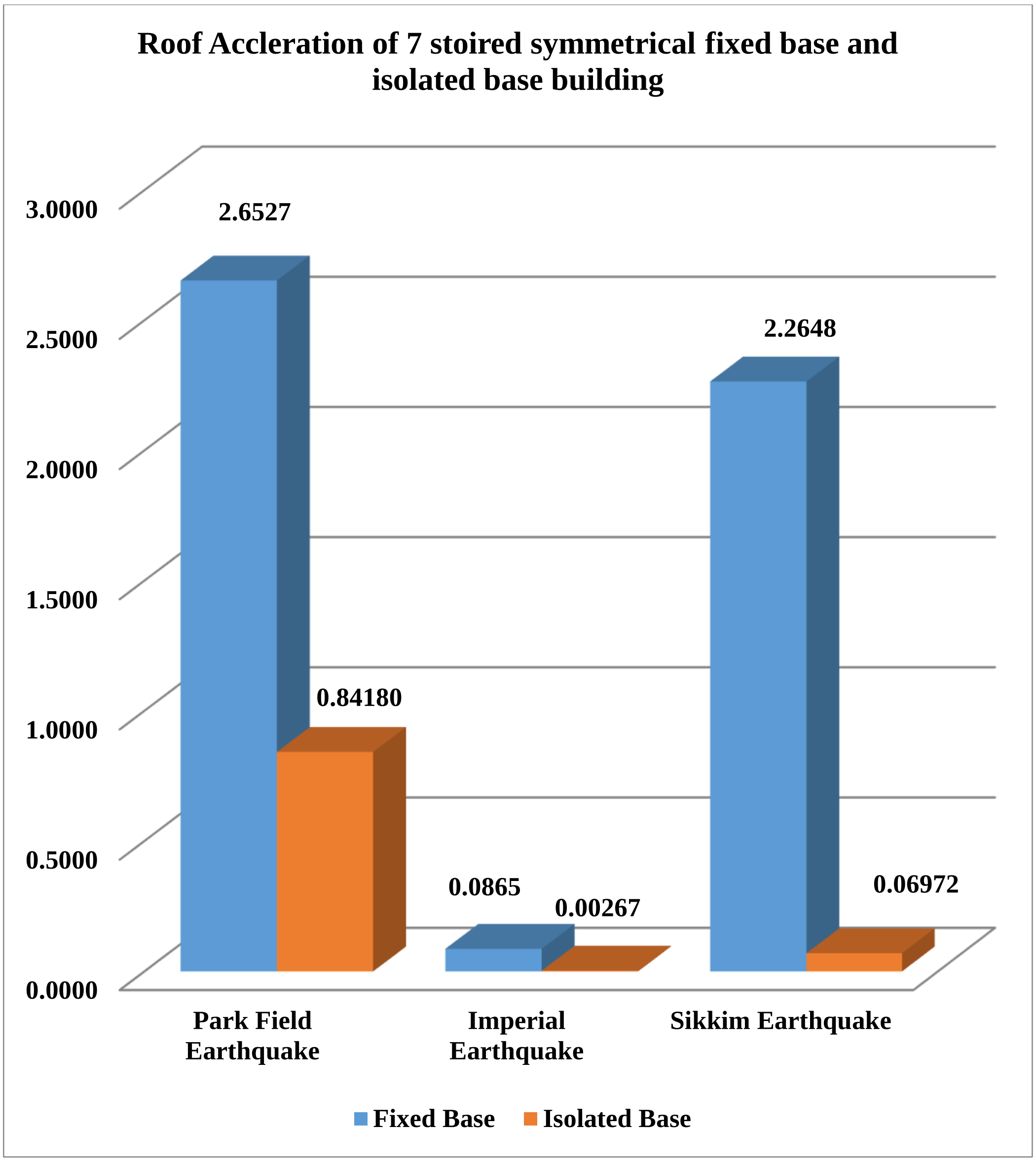


Fig. 6.33 Roof Accleration (in terms of g) of 7 storied asymmetrical fixed base and isolated base building

Base isolation system sigfianctly reduced the roof accleration for both symmetrical and asymmetrical building frame with different ground motion recoreded from past earthquake.

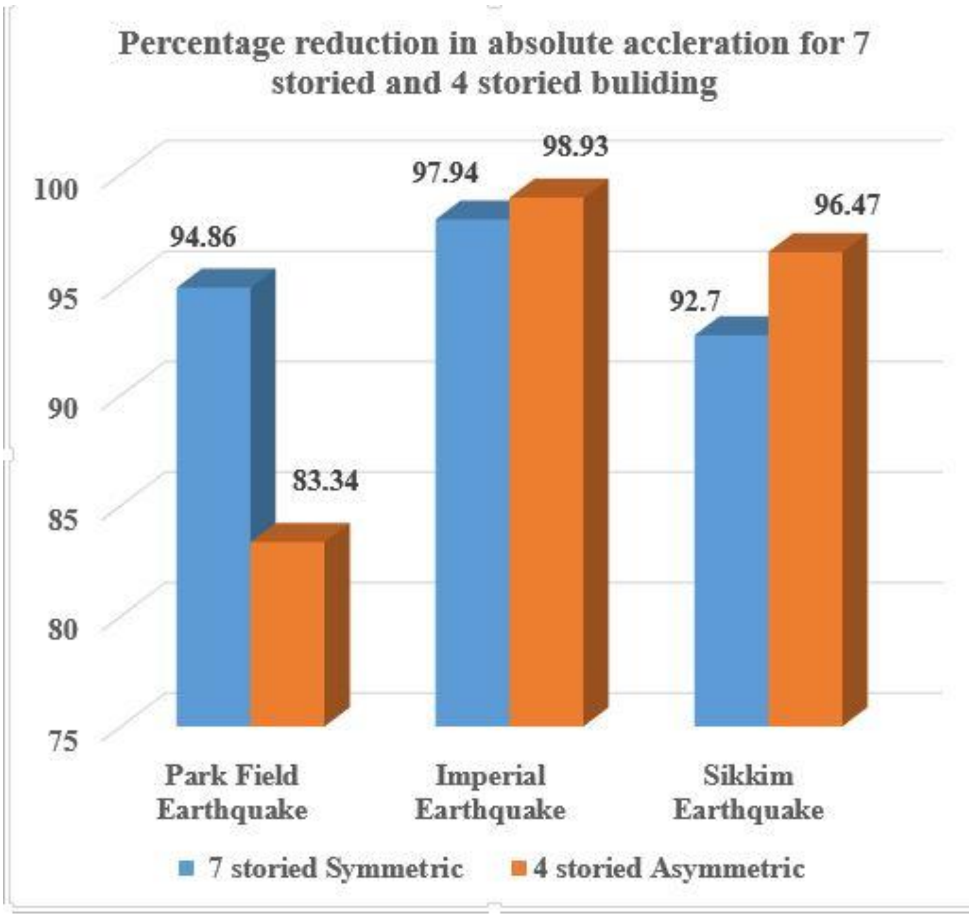


Fig. 6.34 Percentage reduction in absolute accleration at the roof for 7 storied and 4 storied building.

Base isolation system is more effective for 7 storied symmetric building than 4 storied reduced for Park field earthquake. But in case of Imperial and Sikkim earthquake isolator more effectively reduced the ground acceleration in 4 storied asymmetric building with compare to 7 storied symmetric building.

Comparative study of seismic responses of different earthquake ground motion with symmetric and asymmetric building frames is shown in table below.

Table 6.19 Seismic response of different building frames for fixed base and isolated base RCC building frames

Earthquake		Park field (1966)	Imperial Valley (1938)	Sikkim (2011)
Magnitude		6.0	4.8	6.9
Duration of earthquake recorded (in seconds)		43.690	30.050	169.845
PGA (in terms of g)		0.475	0.0118	0.2056
7 story Symmetric building	Roof acceleration for fixed base building (in terms of g)	1.489	0.1351	1.409
	Roof acceleration for isolated base building(in terms of g)	0.076	0.00278	0.10285
	Roof displacement for fixed base building(in mm)	63.277	2.117	36.509
	Roof displacement for isolated base building(in mm)	288.552	14.4	195.25
	Maximum Inter-story drift for fixed base building	10	0.4	5.3
	Maximum Inter-story drift for isolated base building(in mm)	1.1	0.011	0.4
4 story asymmetric building	Roof acceleration for fixed base building (in terms of g)	0.6891	0.49	0.6186
	Roof acceleration for isolated base building(in terms of g)	0.1148	2.69	0.02186
	Roof displacement for fixed base building(in mm)	25.87	0.0699	23.58
	Roof displacement for isolated base building(in mm)	302.44	0.0074	345.12
	Maximum Inter-story drift for fixed base building (in mm)	0.3	0.1	1.4
	Maximum Inter-story drift for isolated base building(in mm)	0.2	0.001	0.161
7 story Asymmetric building	Roof acceleration for fixed base building (in terms of g)	2.6527	.0865	2.2648
	Roof acceleration for isolated base building(in terms of g)	0.08581	.00267	0.0697
	Roof displacement for fixed base building(in mm)	82.589	1.97	46.938
	Roof displacement for isolated base building(in mm)	257.939	11.88	168.84
	Maximum Inter-story drift for fixed base building	15.509	0.411	8.436
	Maximum Inter-story drift for isolated base building(in mm)	0.995	0.005	0.5

Comparison of different parameters between symmetric and asymmetric building configuration of same grade line are shown in figure…….

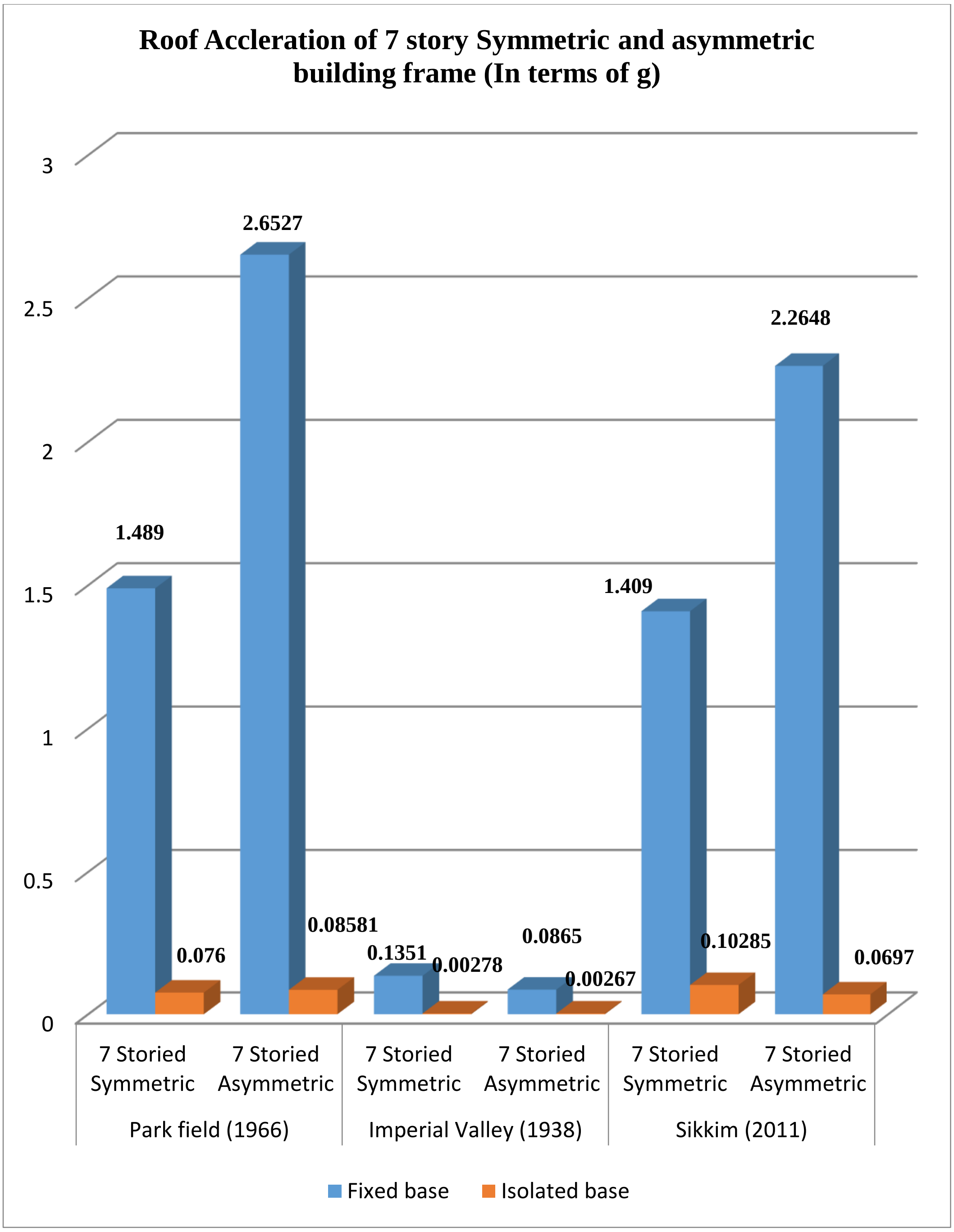


Fig. 6.35 Absolute accleration at the roof for 7 storied symmetrical and asymmetrical building frame.

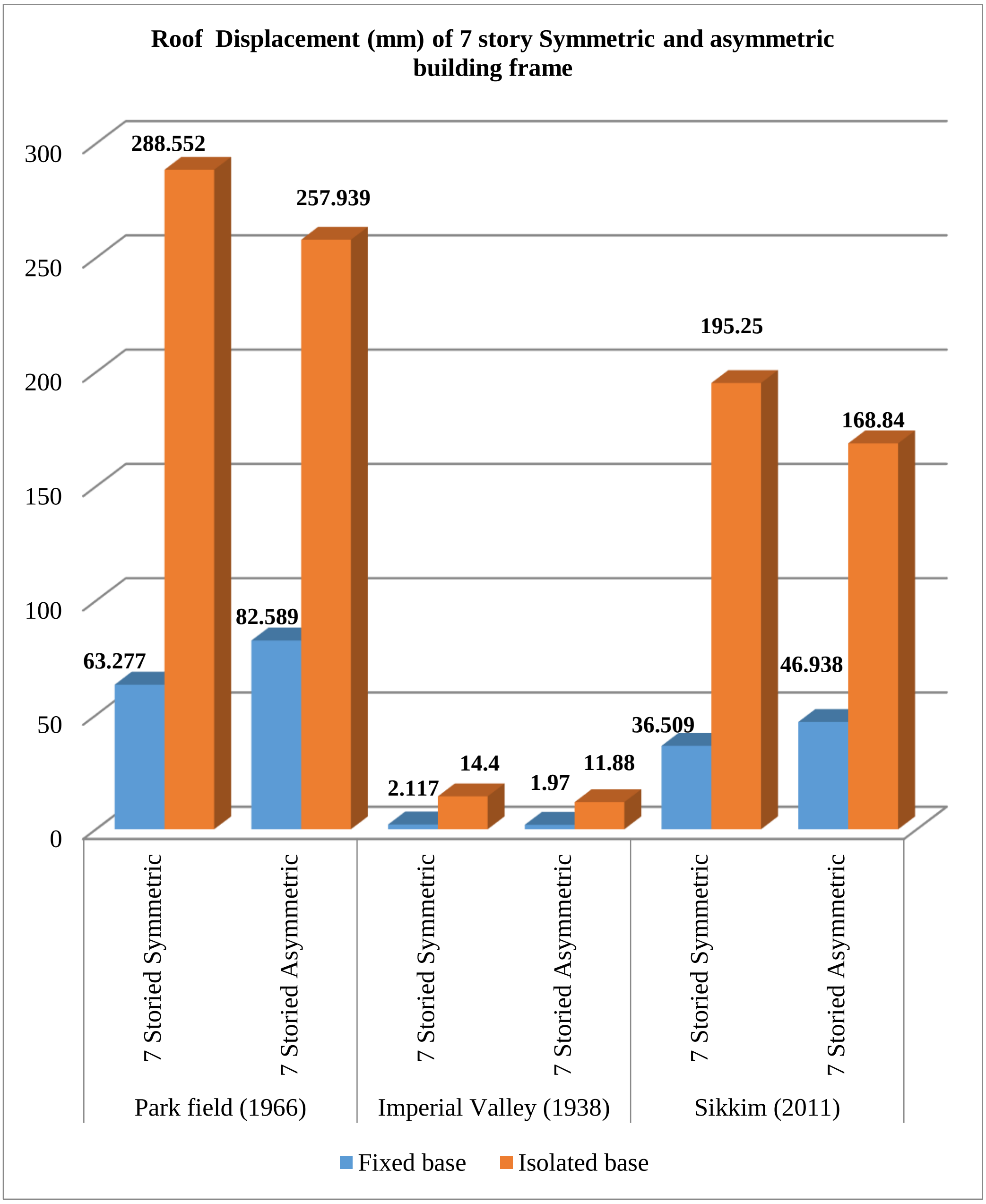


Fig. 6.36 Displacement (mm) at the roof for 7 storied symmetrical and asymmetrical building frame.

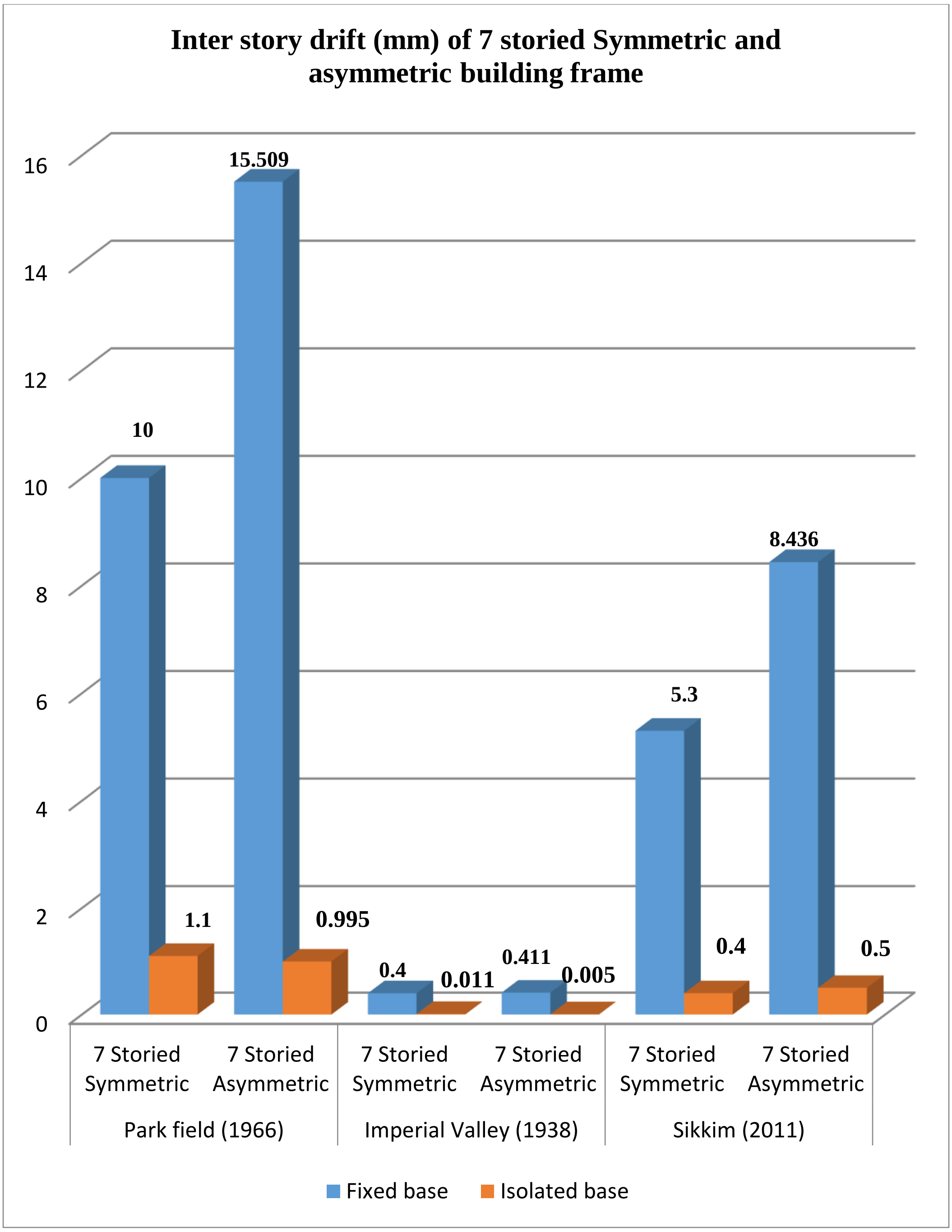


Fig. 6.37 Maximum Inter story drift (mm) for 7 storied symmetrical and asymmetrical building frame.

It seems that there are considerable amount of reduction in roof acceleration and inter-story drift for both symmetric and asymmetric building frames. Similarly the roof displacement also increased to a great extent in the both cases. Thus base isolation system seems to be a good choice of seismic disaster mitigation.

Chapter VII

Conclusion

7.1 General

Based on the numerical study of symmetric and asymmetric stiff building frames subjected to different time history ground motion recorded from past earthquakes, the following conclusion may be drawn

I. Base isolation substantially reduces the transmission of the earthquake forces and energy into the structure which leads to reduction of the structural and non-structural damage to a building subjected to seismic forces to a great extent.

II. The degree of reduction of **roof acceleration** for symmetrical building configurations is in tune for all the earthquakes considered. However, there are variations in the reductions of roof acceleration in case of asymmetric buildings for different earthquakes. The degree of variations depends on the degree of asymmetry.

III. The **roof displacement** increased to a great extent due to the flexibility introduced by the isolator system. The increments of roof displacement are also in tune for symmetrical buildings for different earthquakes. However the increase of roof displacement for asymmetric building frames varies for different time history ground acceleration, which may be attributed to the randomness of the ground motion combined with the asymmetry considered.

IV. The maximum **inter-story drift** also substantially reduced with the incorporation of isolation system, resulted in reduced internal forces. The nature of reductions for symmetric and asymmetric buildings due to different earthquakes follows the same trend as stated earlier. It seems that detail study is required for particular asymmetric building prior to adopt any base isolation system.

V. It is observed that there are considerable amount of **reduction in roof accelerations and inter-story drift** for both symmetric and asymmetric building configurations. Similarly, **roof displacement has increased** to a great extent in both the cases due to the incorporation of the isolator system. The design of the isolation ratio will depend on the **allowable displacement considering the force-displacement trade off**.

VI. Thus the **base isolation system** seems to be an **effective measure for earthquake disaster mitigation** for stiff building frames as evident from the **non-linear dynamic analysis** of numerical models.

7.2 Future Scope of Work

The present study is limited to the numerical study of only three types of buildings with three types of ground acceleration recorded from past earthquake. However this study can be extended to following aspects.

I. Time history analysis can be performed with **variations of building height** with more types of **geometric asymmetry**.

II. **More time history ground** motion can be considered for the analysis of structural models with base isolation system.

III. **Comparison** of time history analysis results with Non-linear static i.e. push over and response spectrum analysis with base isolation system.

IV. Numerical study for **optimization of number of isolators** in frames for cost effective use can be performed.

V. Different **types of isolator** system can be simulated and analyzed for different earthquake ground motion for better understanding of the behavior and efficiency of the isolator system.

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