Placement of Damper for Seismic Response Control of RC Building

BY

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10MCLC01

DEPARTMENT OF CIVIL ENGINEERING

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Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis and Design)

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DECLARATION

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in Civil Engineering(Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Arjun M. Butala

CERTIFICATE

This is to certify that the Major Project entitled "**PLACEMENT OF DAMPER FOR SEISMIC RESPONSE CONTROL OF RC BUILDING**" submitted by **Arjun M. Butala (10MCLC01)**, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering(Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Earthquake produces a disastrous dynamic force on structural systems. The dynamic force is responsible to cause damage to various structural elements or parts of structure and even has potential completely collapse the structure. Understanding of behaviour of structural system to such dynamic force is of prime important, so as structural systems can be made earthquake resistant. Dynamic structural response resulted due to dynamic forces can be controlled by three different approach, namely Passive Control, Active Control and Hybrid Control. The response is primarily control by providing various dampers, which acts either passively or actively. Presently, structural response is controlled by employing passive damper is widely accepted in practice, since they produce resistance damper force by structural motions itself and thus does not require any external energy.

The prime focus of the present study is to evaluate effectiveness of placement of passive damper in a building. A three storey shear building is considered here. Mass and stiffness matrices for the building are obtained considering lumped mass model. Rayleigh damping is assumed considering 5% of critical damping for all modes. A building is subjected to four different types of excitation namely, El Centro (1940), Loma Prieta (1989), Northridge (1994), and Kobe (1995). All possible combination for passive control devices (viscous and viscoelastic damper) are considered for a building, i.e. possible location and nos. of dampers are considered. Equation of motion for each possible combination of damper location and their nos. are derived. Response quantities like displacement, velocity, acceleration, inter storey drift and damper force are extracted for uncontrolled building (without passive damper) and controlled building (with passive damper) using MATLAB.

Comparison among response quantities of uncontrolled building and controlled building shows that later shows moderate reduction in all response quantities. Parametric

study compares of different value of damping coefficient for viscous damper and viscoelastic damper are also carried out. The results shows out of various placement of damper considered, damper placed on each floors works most effectively and reduce responses when only one damper is to be used, results shows damper placed at ground floor provide good response in all response quantities.

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Abbreviation Notation and Nomenclature

ADAS .... Added Damping and Stiffness
DVA .... Dynamic Vibration Absorber
EQ .... Earthquake
FEMA .... Federal Emergency Management Egency
PGA .... Peak Ground Acceleration
PGV .... Peak Ground Velocity
PGD .... Peak Ground Displacement
sgm .... Signum Function
RCC .... Reinforced Cement Concrete
SDOF .... Single Degree of Freedom
VE .... Viscoelastic
GF .... Ground Floor
FF .... First Floor
SF .... Second Floor
a .... Amplitude of Motion
m .... Mass Matrix of Building
k .... Stiffness Matrix of Building
c .... Damping Matrix of Building
f_{ck} .... Characteristic Strength of Concrete
f_y .... Characteristic Strength of Steel
E_d .... Energy Dissipation per Cycle
γ_0 .... Shear Strain Amplitude
V .... Volume of the Viscoelastic Material
P(t) .... Force in an Energy Dissipation Damper as a Function of Time
C_d .... Co-efficient of Damper
α .... Velocity Exponent of Viscous Damper
$\tau(t)$ .... Shear Stress as a Function Time

$\gamma(t)$ .... Shear Strain as a Function Time

f .... Circular Frequency in Radians per Seconds

G' .... Storage Modulus of Viscoelastic Material

G'' .... Loss Modulus of Viscoelastic Material

K_d .... Damper Stiffness

$F(t)$ .... Force in VE Damper as a Function of Time

A .... Shear Area of Viscoelastic Material

t .... Thickness of Viscoelastic Damper

ζ .... Required Damping Ratio of Building

ς .... Target Added Damping Ratio

ξ .... Inherent Damping Ratio of Building

ω .... Fundamental Frequency of Building

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Chapter 1

Introduction

1.1 General

An earthquake is random and rapid shaking of the earth caused by the breaking and shifting of rock beneath the earth's surface. This can collapse buildings and bridges; disrupt gas, electric, and phone service; and sometimes trigger landslides, flash floods, fires, and huge, destructive ocean waves (tsunamis). Buildings foundations resting on unconsolidated landfill and other unstable soil are at risk because they can be shaken off during an earthquake. An earthquake when occurs in a populated area, it may cause deaths and injuries and extensive property damage. After the earthquake, the structures may become the non-functional, which may be problematic to some public structures like hospitals, communications, which need to remain functional after an earthquake. Therefore, it is necessary that structures to be designed to resist earthquake forces, in order to reduce the loss of life.

The seismic design of the structure is required, to prevent the damages in structures or collapse of structures. The earthquake resistant design philosophy currently adopted in the code of practice, almost the world over can be stated as follows:

1. To prevent non-structural damage in minor earthquake ground shaking, which may occur frequently during the service life of the structure.

2. To prevent structural damage and minimize non-structural damage during moderate earthquake ground shaking, which may occasionally occur, say, once in the life time of the structure.

3. To avoid collapse or serious damage during severe earthquake ground shaking, which may rarely occur say the recurrence period of which may be much longer than the life of the structure.[1]

Various technique are derived to reduce examine response of structure when subjected to earthquake. Structure can be provided with various energy dissipation devices like bracing and base isolation. This techniques increases structural stiffness or damping value of the structures, hence, structure remains stable (i.e in working condition) after earthquake. Three basic technologies are commonly used to protect the structure during earthquake, namely, Base Isolation, Passive Energy Devices and Active Control Devices. Earthquake protective system shown in Fig1.1

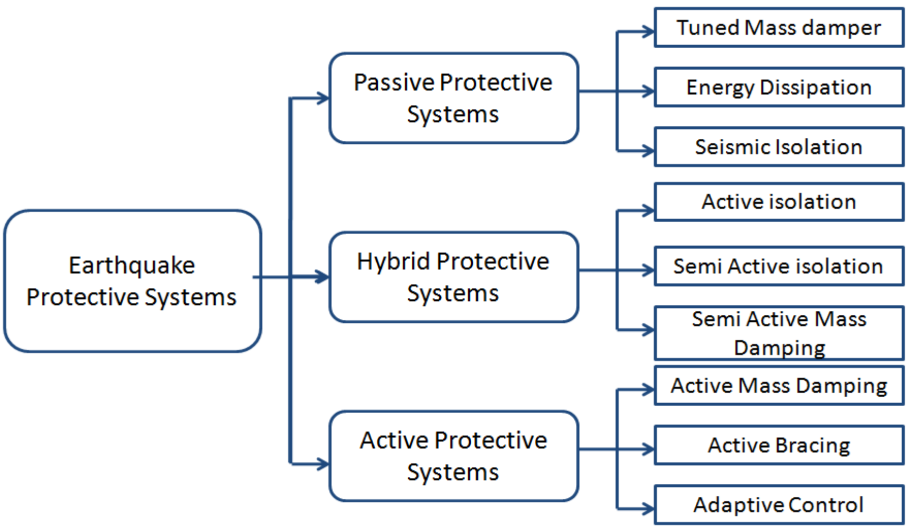

Figure 1.1: Earthquake protective system

Brief description of each of them is given below:

1. Base Isolation: - To isolate the building from the ground in such a way that earthquake motions are not transmitted up through the building or at least greatly reduced.

2. Passive Energy Dissipation: - In passive energy dissipation system the motion of structure is controlled by adding devices to structure in the form stiffness, mass and damping. Passive energy dissipation devices can be effective against winds and earthquake induced motion. It operates on principles such as, yielding of metals, frictional sliding and deformation of viscoelastic (VE) solids or fluids.

3. Active Control System: - Active control systems requires active participation of mechanical devices whose dynamic properties are modified based on feedback from current response measurement. These forces can be used to both add stiffness, damping and dissipate energy in the structure. Thus, Active control system requires external power source to operate. Active control system consists of networking of sensors, actuators and controllers. The active control system includes different types of devices which add stiffness and/or damping.

1.2 Background

Different types of structural protective systems are used to reduce the response of the structure under earthquake excitation like Passive Control dissipation Devices, Active control, Semi active Control and Hybrid Protective Systems. The Passive control Dissipation Systems is most commonly used due to less maintenance and absence of any external power to operate.

A concept of response reduction of structure under the earthquake excitation was developed 100 years ago by Prof. John Miller of Japan. He placed a wooden house on ball bearing to demonstrate isolation from ground shaking. In aircraft, structure sensitive avionics instruments are isolated by providing isolation damper.

Seismic response control of structure is directly influenced by numbers of passive energy dissipation devices and their placement. Seismic response control of building using passive energy dissipation devices like viscous damper, viscoelastic damper and metallic yield damper was studied by Mr. Vijay Chachapara, M.Tech (CASAD), IT, NU during academic year (2010-11). His dissertation work includes seismic response control of building subjected to four different types of earthquake expiation, namely, EL Centro, Kobe, Loma Prieta and Northridge earthquake. Three different passive energy dissipation devices like viscous, VE and metallic yield damper were used. The passive energy dissipation devices was placed at GF of the building. Response quantitative maximum displacement, maximum velocity, maximum acceleration, maximum intensity drift and maximum deformation were extracted for controlled and uncontrolled boundary. The scope of work as mentioned in the dissertation work of Mr. Vijay Chachapara pointed out study of placement of damper for seismic response control. In the present work, influence of damper placement on seismic response center and boundary is aimed to be carried out.

1.3 Objective of Study

The objective of present study is to determine structural response of three story building under strong and pulse type earthquake excitations. The specific objective of work is to study influence of placement of passive energy dissipation devices (Viscous and Viscoelastic) on seismic response control of building. It is aimed to prove the efficiency of passive energy dissipation devices to control the seismic response of building by computing response quantities like, displacement, velocity, acceleration and intensity drift of uncontrolled building.

1.4 Scope of Work

Following scope of work is defined for the present work:

1. Carry out extensive literature review on implementation of passive devices for seismic control of the building.

2. Study, in detail, various mathematical models used for various passive energy dissipation devices and their response characterization.

3. Lumped mass model formulation of the three story building.

4. Formulation and solution of equation of motion for controlled building (with passive energy dissipation devices) and uncontrolled building (without passive energy dissipation devices) using numerical method like Newmark-Beta through MATLAB.

5. Extract response quantities like inter-storey drift, displacement, velocity, acceleration and damper force.

6. Determine the influence of damper placement on seismic response control of the building.

1.5 Organization of Report

The major project is divided into following chapters.
Chapter 2 deals with the details of literature review of various technical papers. It mainly focuses on the mathematical model, behavior and properties of different passive energy dissipation devices and influence of their placement on seismic response control of the structure.
Chapter 3 consists study and characterization of passive devices like viscous and viscoelastic under sinusoidal and random excitations influence of damper dynamics on damper force generative studied.

Chapter 4 includes formulation and solution of equation of motion for building with and without passive devices using Newmark-Beta method under four different excitations through MATLAB. Response quantities like inter storey drift, displacement, velocity, acceleration, damping force of uncontrolled building are extracted.

Chapter 5 covers formation and solution of equation of motion for the shear building equipped with viscous damper using Newmark-Beta under four different earthquake excitations through MATLAB. It also includes extraction of the response quantities. i.e., inter storey drift, displacement, velocity, acceleration and damping force. The response quantities of controlled and uncontrolled building are compared.

Chapter 6 gives formulation and solution of equation of motion of three story building with viscoelastic damper attached. It also covers design of VE damper to achieve derived damping response quantities are extracted for controlled building and are compared with response quantities that are of uncontrolled building.

Chapter 7 includes the summary of the work and conclusions.

Chapter 2

Literature Survey

2.1 General

The damping devices have been developed in order to reduce effectively the seismic response of structures subjected to earthquake excitation. Passive energy dissipation devices have the potential to increase the seismic resistance of a structure by increasing its capabilities to dissipate energy and by reducing the seismic demand of the structure. The usefulness of the devices is a function of where they are located in the structure. Various formulas and literature available about characteristics of passive dissipation devices and capabilities of seismic devices in seismic response validation. Some work showing influence of placement of passive dissipation device on seismic response validation are also available. Following section covers important work related to use of passive damping devices.

2.2 Literature Review

Various papers have been referred for basic understanding of passive devices considered in buildings, their mathematical modeling, behavior and available applications of passive dampers. Some of the important papers are summarized below.

Fujita et al.[2] [2010] had developed two different types of model which are 3N model and N model as shown in Fig 2.1. The relative displacement between each component of damper unit can be defined in 3N Model. The N Model is a simpler model where the whole damper unit is converted to an equivalent frequency dependent Kelvin-Voigt model. Structural model with viscoelastic damper inclusive of supporting member is shown in Fig 2.2. The varied stiffness of dampers varies with the natural frequency of the structure and so the optimal placement for viscoelastic damper is identified with the selection of the optimal stiffness of supporting member. The size or quantity of viscoelastic dampers becomes large, the force acting on the supporting member increases. Simultaneous design consideration of viscoelastic dampers and supporting members is a new aspect of the theoretical development and practicality. For finding out the optimal placement of damper, a gradient based evolutionary optimization technique using the Lagrange multiplier optimization technique was used. The optimality criterion of placement for viscoelastic damper is satisfied by solving 3-storey and 10-storey numerical problem.

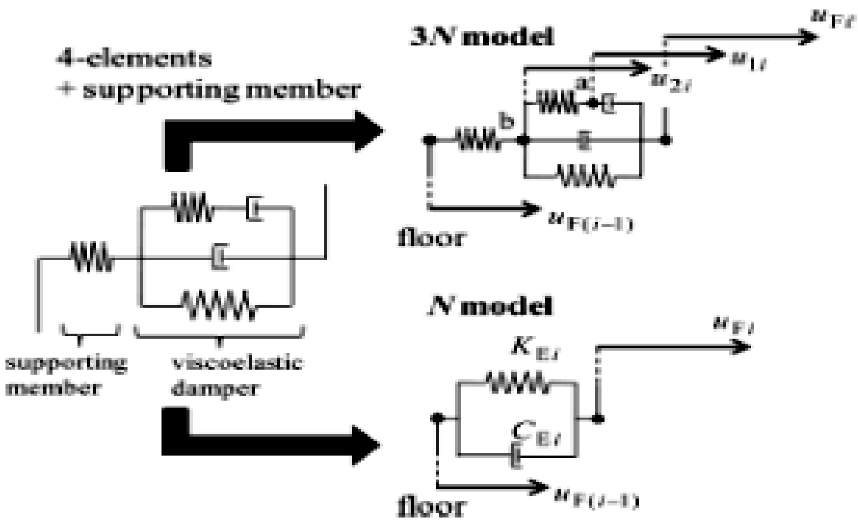

Figure 2.1: Damper models simplified as "3N model" and "N model"

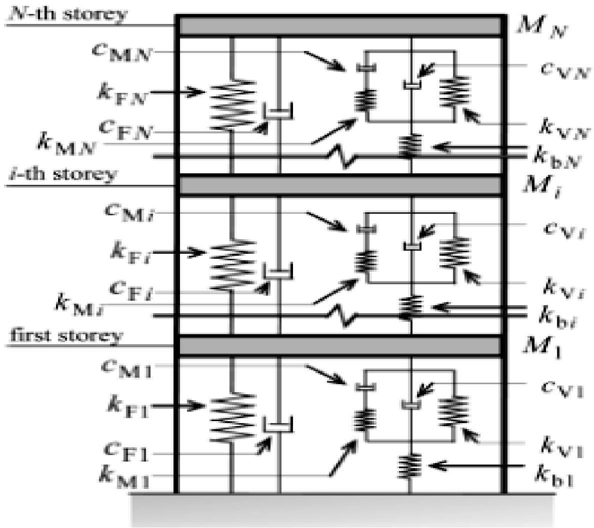

Figure 2.2: Structural model with viscoelastic damper including supporting member

TOVAR and LOPEZ[3] [2004] studied the influence of the number and placement of dampers on the dynamic response of the building frame of five storey structure was considered with varies in number and location of one, three and five dampers in the structure as shown in Fig.2.3, 2.4 and 2.5.

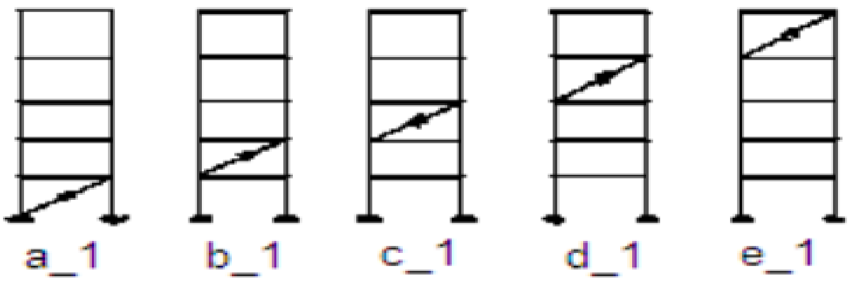

Figure 2.3: Case of study to assess the effect of one damper's location

The usefulness of the devices is a function of where they located in the structure. The main objectives of their work are: (i) to assess how the variation of placement and number of dampers affect the seismic response of a frame structure, and (ii) to evaluate a simplified method to analyze frame structures that have non-classical damping,

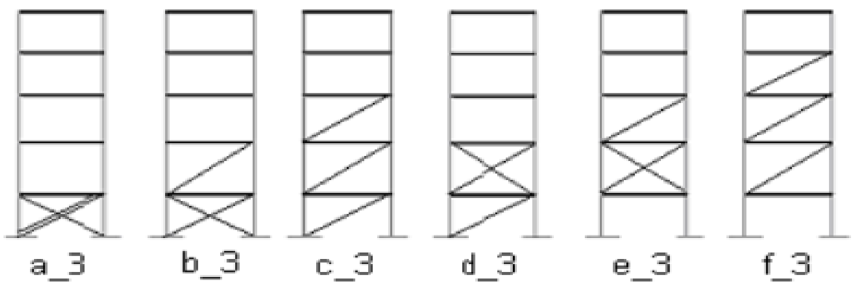

Figure 2.4: Case of study to assess the effect of three damper's location

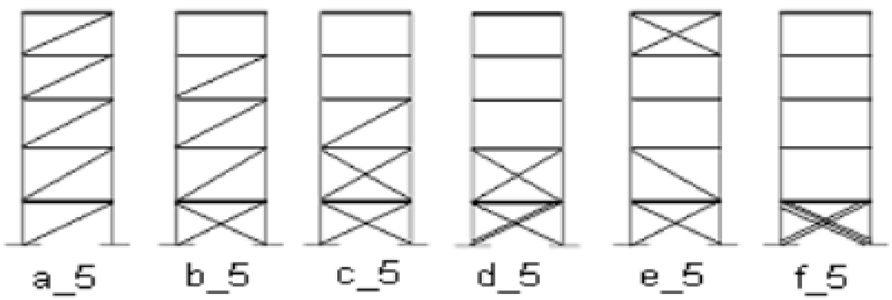

Figure 2.5: Case of study to assess the effect of five damper's location

in order to study how the error in the simplified method is influenced by placement of dampers. They found out drift reduction by eq.2.1 The response of the structure for drift reduction v/s damper location is shown on Fig 2.6,2.7 and 2.8.

Drift reduction

$$\beta_i^k = \frac{\Delta_i^k}{\Delta_i^0} \tag{2.1}$$

where

β_i^k= benefit in drift reduction at storey in case k and

Δ_i^k =maximum drift on storey in case k

Δ_i^0 =maximum drift on storey in case 0

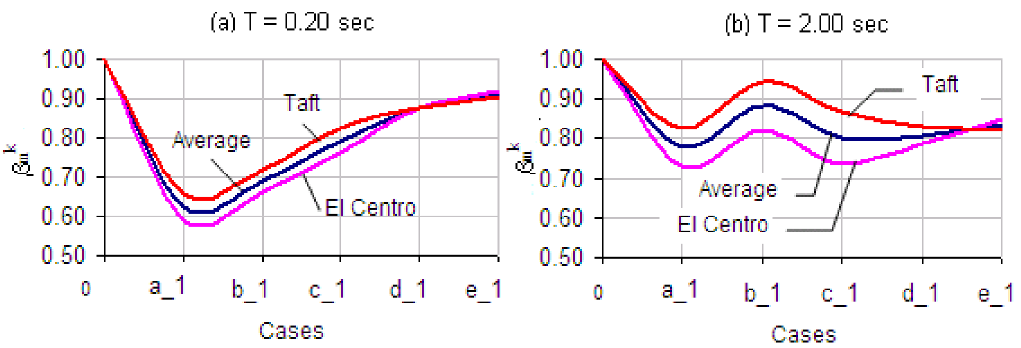

Figure 2.6: Effect of one damper's placement

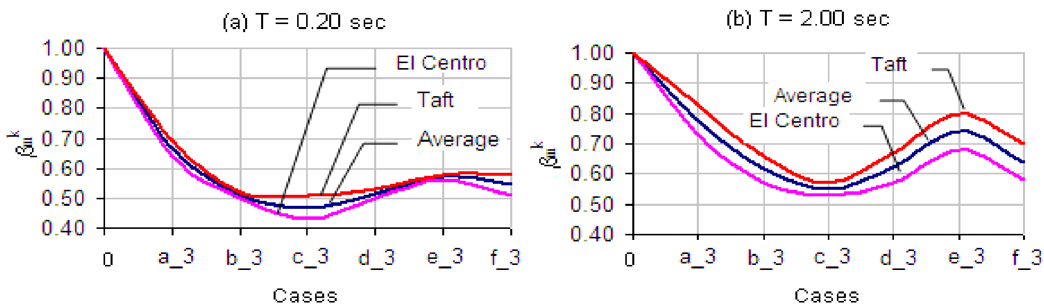

Figure 2.7: Effect of three damper's placement

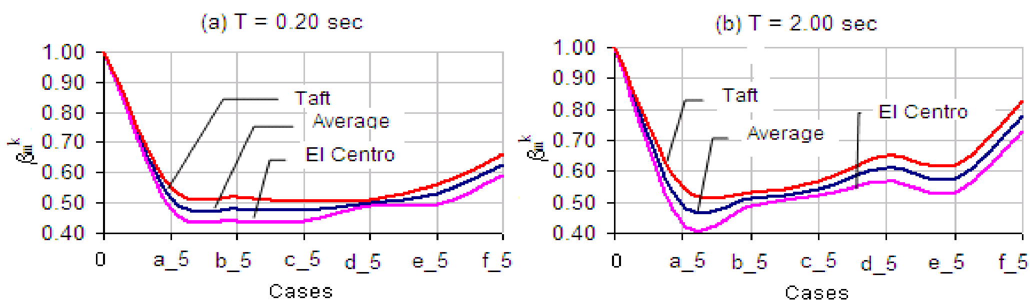

Figure 2.8: Effect of five damper's placement

From this investigation the following conclusions were made:

1. The dampers placement influences significantly the structural response. A large number of dampers do not always leads to the best benefit in terms of drift reduction for all stories. Three dampers lead to the best overall benefit for all stories in this structure.

2. When only one damper is placed this should be located at the first storey in order to obtain the best overall drift reduction. The best damper placement is one damper per storey; if the number of dampers is less than the number of stories, one damper per storey beginning at the lowest storey is the best choice.

3. The simplified method is not recommended for a damper distribution concentrated in a few stories, because large errors in the structural response could be obtained.

4. The analysis considering the simplified method may be used without introducing significant errors in the systems with a more uniform damping distribution, that is, one damper per storey with the same damping constant.

Kokil and Shrikhande[4] [2007] studied effect of Fluid viscous damper in a 3-D 10 storey building model. Dampers were provided in all four directions (North, South, East and West). The sample for damper placement is shown in table 2.1 in which the 1 indicate the damper and '0' indicate absence of damper.

Table 2.1: Sample for the damper location

Direction	Storey Level									
	1	2	3	4	5	6	7	8	9	10
North	1	0	0	0	0	0	0	0	0	0
South	0	0	0	1	0	0	0	0	0	0
East	0	0	1	0	0	0	0	0	0	0
West	0	1	0	0	0	0	0	0	0	0

To seek the optimal location of dampers, a linear combination of maximum inter-storey drift and maximum base shear of the damped structure normalized by their respective undamped counterparts has been taken as the objective function. The supplemental dampers are more effective in reducing the seismic response of a symmetric response of a symmetric building and its effectiveness reduces as rather plan irregularity. The optimal damper placement to minimize the sum of amplitudes of the transfer functions evaluated at the undamped natural frequency of a structural system subjected to constrain on the sum of the damping co-efficient of added dampers. The optimal placement increase the lower mode damping ratio more effectively than uniform placement and that the increase in number of additional dampers does not always reduce the structural response.

Rodrigo and Romero[5] [2003] investigated the effect of linear and nonlinear viscous dampers on the response of a six storey, three bay moment resisting steel frame under seismic loads. A pair of dampers was installed in the middle bay of each storey. They presented a simple methodology for the optimal retrofitting of the structures with nonlinear viscous dampers. First, the optimality of linear viscous damper was based on achieving good performance in terms of FEMA 274 (FEMA [1997]) target displacement levels and minimizing the axial forces developed in the damper. Then, the optimality of nonlinear viscous dampers was based on the equal energy dissipation approach. According to the results of the nonlinear response history analysis, they observed that for large values of the nonlinear parameter, N, about 0.8 or 0.9, the envelope of the response remained almost constant while the damper forces were reduced from the linear damper case. They also concluded that if these devices are designed with moderate nonlinearity level (N 0.8 to 0.9), the same good performance can be obtained as the case of linear viscous dampers but with a reduction of the damper forces more than 35 %.

Singh and Moreschi[6] [2001] presented a method to obtain the amount of viscous and viscoelastic damping required for an elastic structure to obtain the desired response reduction. The required supplemental devices were also optimally distributed in the structure. The method was based on a gradient based optimization approach. To check the applicability of the proposed method, numerical results of a 24-storey building were presented. The reduction of response was expressed in terms of inter-storey drift, base shear, or floor acceleration.

Singh and Moreschi[7] [2002] presented a genetic algorithm to determine the optimal size and location of the frequency dependent and independent viscous dampers as well as the viscoelastic dampers. The genetic approach was used to calculate the required number of a given capacity dampers and their optimal placement locations in a building to achieve the desired reduction in the structural response. According to the genetic approach presented, the response reduction can be expressed in terms of many functions such as base shear, bending moment at column bases, or floor acceleration. To illustrate the applicability of the genetic approach, they presented some numerical results for a shear building model and torsional building model with three types of damping devices. The shear building was a 24-storey building with non-uniform structural properties. The details of this building are shown in Chang et al. (1991). The second building was six storey torsional building. The stiffness and mass center of the building were not coinciding. The structures were assumed to be elastic assuming the added viscoelastic dampers were designed to prevent inelastic deformations.

Aiken, et al.[8][1992] conclude that, the response quantity of drift with inter storey is similar for Moment Resisting Frame (MRF) and structures attached to Viscoelastic (VE) damper and Friction Damper (FD). The response quantity acceleration is similar for Concentrated Braced Frame (CBF) and controlled structures. For Controlled structures, the displacement and acceleration is similar. It was also concluded that, for VE damper the hysteresis loop is regular and it has no thresh old value or activation force level so they dissipate energy for all levels of earthquake excitation. For FD, the characteristic of hysteretic loop is regular and repeatable. No change in slip load was observed during earthquake excitation, hence it can't slip or dissipate energy. Comparing both FD and VE damper, former does not achieved for small scale earthquake, i.e. low PGA earthquake and motion.

Ajeet Shukla and Datta[9] [1999] investigated the control of the seismic response of multi-storey building frames using optimally placed viscoelastic dampers. The responses were obtained in the frequency domain using spectral analysis for narrow and broadband stationary random ground motions. To determine the optimal location of the dampers, a controllability index as shown in eq. 2.2, based on the root mean square value of the inter-storey drift, was used.

$$x_L = max[\frac{\sigma_x(L)}{h(L)}] \tag{2.2}$$

Where, x(L) and $\sigma(L)$ are location index and RMS value of interstorey drift at L^{th} storey. h(L) is L^{th} storey height.
Three mathematical models of viscoelastic dampers, namely linear, Kelvin and Maxwell models were used. Three alternative schemes of damper placement were studied. To apply the proposed strategy of optimization, they considered twenty storey shear frame models with the viscoelastic dampers as shown in Fig 2.9. The results showed that the scheme with optimally placed dampers provides more reduction in storey drifts than other schemes. It was found that the optimal placement of dampers are sensitive to the nature of the excitation force, total quantity of viscoelastic material used, and the modeling of the viscoelastic damper.

Zhang and Soong [10][1992] developed a sequential procedure for optimally placing viscoelastic dampers in structures. The method was based on the concept of degree of controllability. They verified the method experimentally through a five storey steel model structure (Chang et al. [1991]). To apply the method, they carried out a numerical example of a ten-storey steel frame. It was shown that for the ten storey

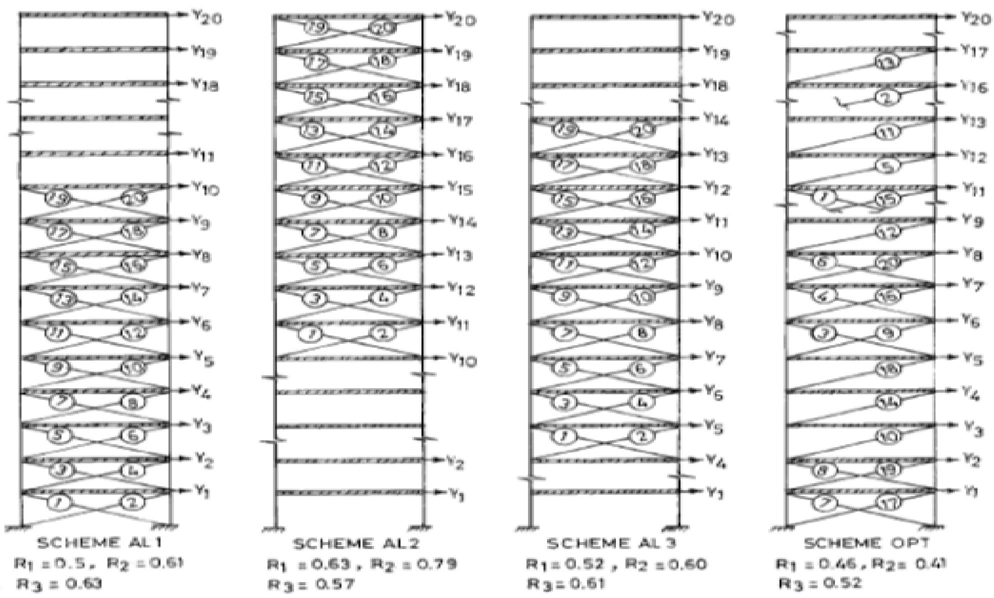

Figure 2.9: Different Placement Schemes of VEDs in Building Frame

frame, a saving of 2-5 dampers could be obtained when considering the storey drifts as the criterion of the response reduction. According to the results of the experimental and numerical examples, it was found that the economical use of the viscoelastic dampers could be obtained by placing the dampers at the optimal locations found by the procedure. They presented a design procedure that can determine the damper dimensions, number and locations needed to achieve the desired additional damping according to the structural parameters, and the structural response reduction requirements. A design for the viscoelastic dampers for a 24-storey steel frame as an example was obtained. The distribution of the viscoelastic dampers shown in Fig 2.10.

From practical point of view, there are many uncertainties to validate a certain optimum placement of dampers in structures. The optimization in dampers configurations is based on a certain ground motion that will not occur again. Moreover, the structure is assumed to behave elastically, which will almost certainly not be true especially under strong ground motions.

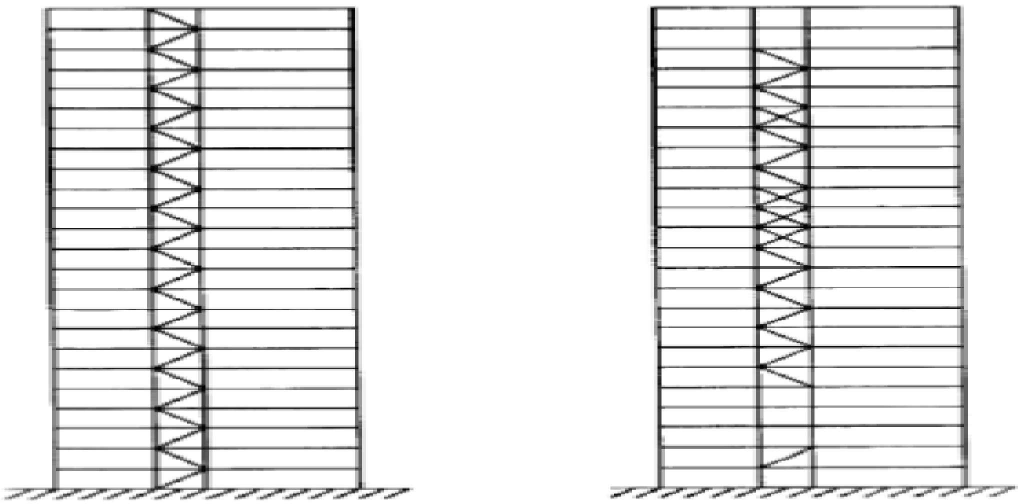

Figure 2.10: 5 Different Placement Schemes of VEDs in Building Frame

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes mathematical modeling, hysteresis behavior and properties of various passive dampers. Numerical example of building attached with passive energy dissipation devices and important results are quoted.

Chapter 3

Passive Control Systems

3.1 Introduction

Passive energy dissipation devices have the potential to increase the seismic resistance of a structure by increasing its capability to dissipate energy and by reducing its effect on the structure. The main role of a passive energy dissipater is to increase the hysteretic damping in the structure. The collapse of the structure cause due to vibration or dynamic loading in the structures. Due to these vibrations the large amount of energy is imparted. To reduce or dissipate the energy, it is requiring the energy is absorb by the structure. Research is under way to reduce the response of the structures resulting due to dynamic loading. A widely considered strategy consists of incorporating external elements to the structure to control its dynamic response. For passive control devices, the device generates control forces at the points of attachments and the power needed to develop such forces are originated from the motion of the points of attachments. As with purely passive devices, the forces in the device are developed from the motion of the attachments points. The amplitude and direction of these forces are determined by the relative motion of these points. To reduce structural response due to earthquake, wind and other dynamic loads the passive energy dissipation system is required. Passive control system develops control forces at the point of attachment of the system. The power needed to generate these forces is provided by the motion of the points of attachment during dynamic excitation. Passive energy dissipation systems encompass a range of materials and devices for enhancing damping, stiffness and strength, and can be used both for natural hazard mitigation and for rehabilitation of aging or deficient structures.

3.2 Classification of Energy Dissipation Devices

Concerning the passive devices, they are usually categorized into two main categories; displacement dependent and velocity dependent devices and also Dynamic Vibration Absorber Figure 3.1 shows the classification of passive energy dissipation devices. The displacement dependent devices, like steel plate dampers and friction dampers, dissipate energy through yielding of the damper elements or through sliding friction. Velocity dependent devices, like viscous fluid dampers and viscoelastic dampers, rely on Viscoelasticity in dissipating energy. The velocity dependent devices provide damping and (optionally) stiffness to the structures and are used to dissipate energy for all levels of excitations. On the other hand, displacement dependent devices provide added stiffness to structures and they dissipate energy under moderate and strong excitations only.

3.2.1 Displacement Dependent Devices

Displacement dependent devices dissipate energy through sliding friction, like friction dampers, or through the inelastic behavior of the damper elements, like metallic dampers. These devices provide lateral stiffness to the structure and consequently reduce its deformation demand. However, increasing the stiffness may damage the building contents due to the excessive accelerations. Higher stiffness often increases base shear, and bending moment at column bases as well. A variety of hysteretic devices has been proposed and developed to enhance structural safety. These devices generates rectangular hysteresis loop in general. This shows that the behaviour of

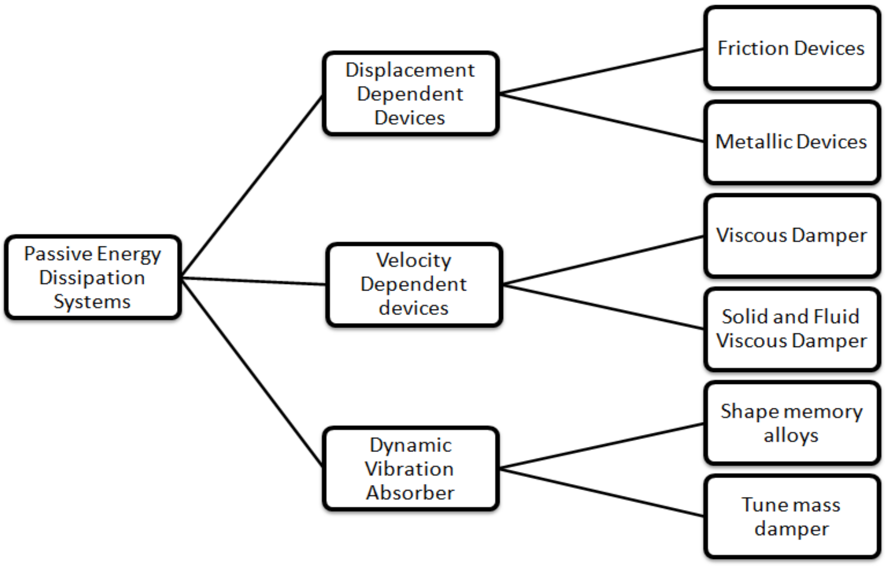

Figure 3.1: Classification of Passive Energy Dissipation Systems

friction dampers is close to that of coulomb friction. The simplest models of hysteretic behavior involve algebraic relation between force and displacement. Hence, hysteretic devices are often called displacement dependant.

Metallic Dampers

To achieve a good structural design to resist seismic excitations, it is usually necessary to rely on inelastic behavior of structural elements. This behavior results in dissipation of the seismic energy transferred to the structures, which improves the structural response. However, the structural elements may experience severe yielding that affects the post event usability of the structure. The basic concept of the metallic dampers is to introduce specific metallic elements in order to absorb the energy during the seismic excitations through their inelastic behavior.Kelly et al.(1972)[11] and Skinner et al.(1975) [12]were the first researchers to consider the idea of using separate metallic dampers for energy dissipation. They considered different types of these dampers such as torsional beam, flexural beam and U-strip dampers. The most well known and used types of these dampers are the X-shaped and triangular plate dampers.

X-shaped Plate Dampers

The flexural plate steel dampers are the most used types of the metallic dampers. This type includes the X-shaped and V-shaped plate dampers. In 1987, Bergman and Goel had performed cyclic tests on X-shaped and V-shaped plate dampers. The geometry of these types is shown in Figure3.1 The dampers were attached to a full-scale single-storey building. Three dampers were tested under constant displacement amplitude. Different displacement amplitudes up to 1.5 in. were applied with a forcing frequency of 0.33 HZ for 10 cycles. The X-shaped dampers performed better than the V-shaped dampers regarding the energy dissipation as well as their durability. Another type of the **X-shaped** dampers, known as the **Added Damping and Stiffness** (ADAS) was introduced in 1990 by Xia et al.[13] The ADAS device is an assemblage of steel plates. When installed in a building frame, the devices are connected to the beams so that the storey drifts cause relative horizontal displacements, which lead to energy dissipation through the yielding of large volume of steel. A typical configuration of ADAS devices and how they deform when subjected to shear forces due to storey drifts are shown in Figure.3.2

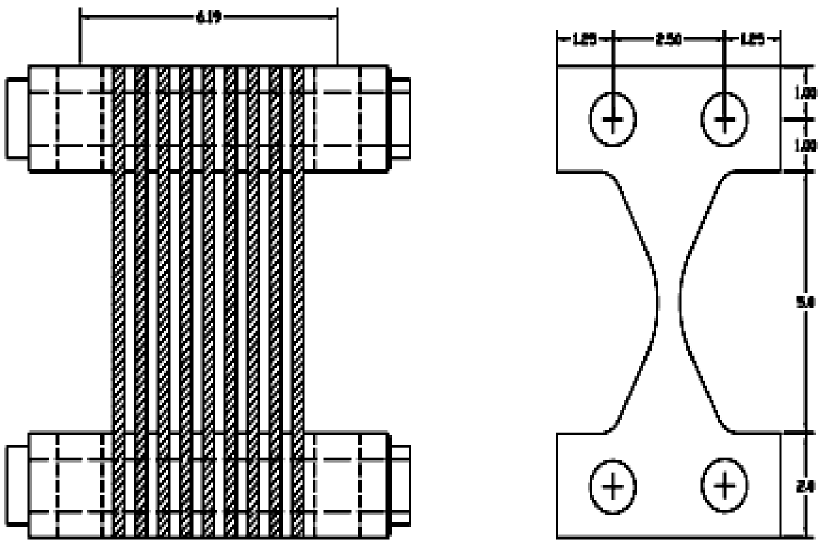

Figure 3.2: X shaped Metallic Damper

The main advantages of these devices are:

- Large inelastic deformations are constrained in ADAS elements, which are designed for this purpose. After moderate to strong earthquake excitations, these devices can be easily replaced.

- The devices considerably increase the equivalent viscous damping in structures, which results in a reduction in structural response under vibrations. Also, the energy dissipation demands for other structural elements are reduced.

- ADAS devices can be used in new structures as well as in existing structures for retrofitting.

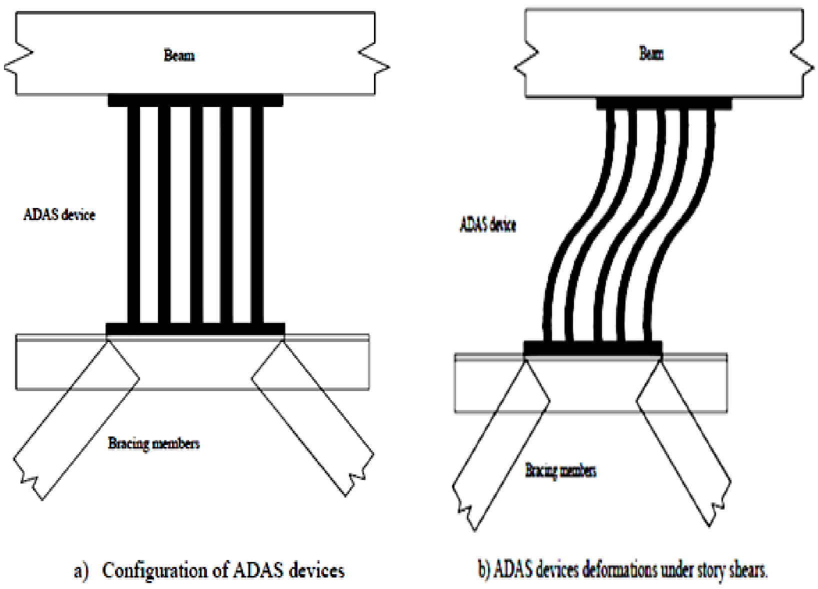

Figure 3.3: ADAS Damper

3.2.2 Velocity-dependent devices

Velocity dependent Devices, like viscous fluid dampers and viscoelastic dampers, dissipate energy through forces proportional to the velocity of the motion. Velocity dependent devices provide damping and stiffness to the structures and are used to dissipate energy for all levels of excitation while displacement dependent devices usually provide stiffness and the energy dissipation takes place under moderate and strong excitations only.

A.) Viscous Fluid Dampers

Recently, viscous fluid dampers were widely used in structures as passive control devices. The energy is dissipated through the viscous fluid dampers by moving a piston that forces a viscous fluid through orifices in the piston head. The force developed in the damper is proportional to the velocity of the moving piston.

Types of Viscous Dampers

Viscous dampers dissipate energy through the moving of a body in a viscous fluid. Different ideas have been developed having the same concept in order to improve the response of buildings under seismic and wind loads. One design approach of viscous dampers is to dissipate energy through the conversion of mechanical energy to a heat. This can be applied through deforming highly viscous substance by a moving piston (Schwahn and Delinic, 1988)[14]. An example of this type is shown in Figure 3.4(a). Another design approach depends on moving a viscous damping wall in its plane through a viscous fluid contained in a narrow rectangular steel container. The wall is attached to the upper floor while the container is fixed to the lower floor. Figure 3.4(b) shows an example of this type. The most common type of viscous dampers is shown in Figure3.4(c). In this type, the energy dissipation is developed through a moving piston that forces a fluid to pass through small orifices around and through the piston head (Constantinou and Symans (1993))[15]. Fluid velocity is very high in the other side of the piston head, thus the upstream pressure energy converts almost entirely into kinetic energy. Accordingly, the fluid expands into the full volume on the other side of the piston head and slows down and then loses its kinetic energy into turbulence. The difference in pressure between the upstream and downstream produces a large damper force.

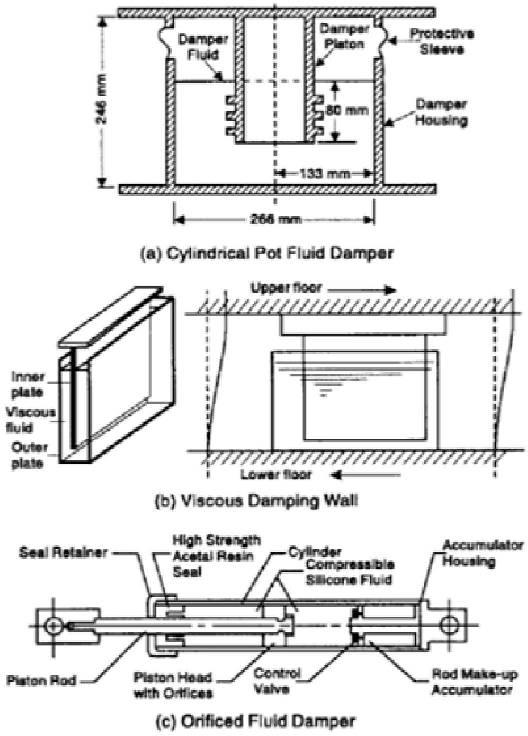

Figure 3.4: Types of Viscous Fluid Damper

Effect of Nonlinearity in Viscous Dampers

In general the force developed in a viscous damper can be determined by the following relation:

$$F = Csign(v)|V|^N \tag{3.1}$$

where F is the damper force, C is an arbitrary constant, V is the relative velocity of the piston, and N is an exponent that can range from 0.3-1.95 and remains constant over the full range of velocities.

B.) Viscoelastic Dampers

The first application of Viscoelastic Damper was applied in 1969 to the twin tower of the World Trade Center in New York City, to reduce motion under wind loads. The viscoelastic damper is also used in Columbia Seafirst Building. This Columbia tower is 76 storey building rising to a height of 938 ft above ground level. To reduce the wind-induced motions, the designers used 260 viscoelastic dampers to be installed alongside the main diagonal bracing members in the core of the building. Viscoelastic dampers typically consist of a solid viscoelastic material sandwiched between steel plates. Energy is dissipated through large shear strains in the viscoelastic material. Implementation of viscoelastic dampers causes a small increase in structural stiffness due to the inherent storage stiffness of the viscoelastic material. One of the primary advantages of the viscoelastic dampers is that they dissipate energy under all levels of ground motion.

Configuration of Viscoelastic Damper

The typical configuration of the viscoelastic damper is as shown in Figure3.5, which is constructed from two viscoelastic layers bonded by three rigid surfaces. The damper is placed in the structure where vibration is expected to cause shear deformations in the viscoelastic material.

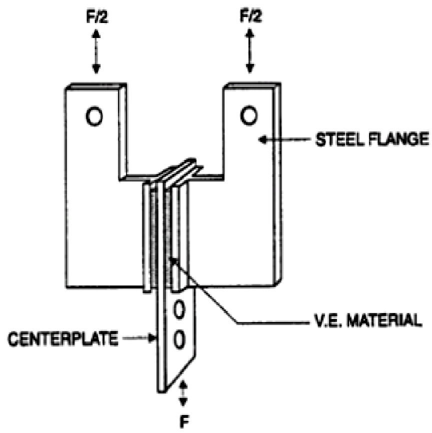

Figure 3.5: Viscoelastic Damper

Design of Viscoelastic Dampers

The behaviour of the viscoelastic damper is greatly influenced by different factors. These factors are the environmental temperature, the number of load cycles, the amount of strain and the excitation frequency. The temperature should be considered in two ways; the environmental temperature and the rise in temperature within the material due to the cyclic loads. Keel and Mahmoodi (1969)[16] performed a series of tests to investigate the effect of temperature on the behavior of viscoelastic materials. He found that the energy dissipation by the viscoelastic material per one cycle varies inversely with the temperature. Also, he observed that the relationship between the temperature and the number of load cycles is linear in the first 100 cycles and after about 400 cycles, the relationship can be considered constant. The results of the experiments indicated that only 2-4 % of the energy dissipated by the viscoelastic dampers is stored in the material, which means that the generated heat in the viscoelastic material is dissipated quickly so that the temperature of the viscoelastic material does not rise to high values. The main goal of using the viscoelastic dampers is to dissipate energy from the vibrating structure. It is recommended in the design of viscoelastic dampers that the dampers dissipate energy as much as possible to reduce the damage of the surrounding elements. Samali and Kwok (1995)[17] summarized the research work relevant to viscoelastic dampers and identified the factors that can improve their performance. For a viscoelastic material subjected to a sinusoidal shear loading, the total energy dissipated in one cycle can be calculated from [Samali and Kwok (1995)]

$$Er = \pi\gamma_o^2 G^{''} V \tag{3.2}$$

where γ_o is the shear strain, V is the volume of viscoelastic material, and $G^{''}$ is the shear loss modulus.

According to equation3.2, they indicated that the parameters that can improve the effectiveness of a viscoelastic material in viscoelastic dampers are the shear strains and volume of viscoelastic material. The following comments were presented:

- Viscoelastic dampers should be placed in locations that are subjected to large deformations in order to increase the shear strains developed in the viscoelastic material. In practical applications, there are only few locations, which are suitable for placing dampers. Hence, increasing the effectiveness of viscoelastic dampers through increasing shear strains is limited.

- Any increase in the viscoelastic volume should be considered along with the stiffness requirement and the thermal properties of the system. Any change in the volume of the viscoelastic material, whether through the shear area, thickness or width, will affect the stiffness of the damper. The energy dissipation through the viscoelastic material to the surrounding environment depends on the viscoelastic material thickness, the available heat conduction area, and the overall thermal conductivity of the system.

Effectiveness of Viscoelastic and Viscous Fluid Dampers

Both viscoelastic and fluid viscous dampers improve the response of structures under different dynamic loads. Viscoelastic dampers provide stiffness and damping while viscous fluid dampers provide damping only under conditions of low frequency movement.

The viscoelastic dampers are very effective in reducing the response of tested structures due to the seismic excitations. They noticed that at 77^oF, the viscoelastic dampers reduced the response by 80 %, while at higher temperatures 107^oF the dampers were still able to reduce the response by 40 %.

Viscoelastic dampers are typically placed in structures in temperature controlled environment so the performance of viscoelastic dampers will not be affected due to environmental temperature change.

Optimization of Viscoelastic and Viscous Fluid Dampers

Over the past few years, many researchers considered the optimization of viscous and viscoelastic dampers in order to get the best performance of structures using these dampers. Zhang and Soong (1992)[18]developed a sequential procedure for optimally placing viscoelastic dampers in structures. The method was based on the concept of degree of controllability. They found that, to optimize the no. of damper to the structure the storey drift is an important factor. Then, they presented a design procedure that can determine the damper dimensions, number and locations needed to achieve the desired additional damping according to the structural parameters, and the structural response reduction requirements. From practical point of view, there are many uncertainties to validate a certain optimum placement of dampers in structures. The optimization in dampers configurations is based on a certain ground motion that will not occur again. Moreover, the structure is assumed to behave elastically, which will almost certainly not be true especially under strong ground motions.

Some Applications of Viscoelastic and Viscous Fluid dampers

Due to the effectiveness of the viscous fluid and viscoelastic dampers in reducing the response due to the seismic excitations and the wind loads, many buildings were constructed with these dampers.

- One of the most famous buildings in the world, the World Trade Center, New York 1969, had about 20,000 viscoelastic dampers in the two towers. The viscoelastic dampers were used to increase the resistance of the tubular steel frame against the wind induced building oscillations. The dampers were distributed throughout the building from the 10^{th} to the 110^{th} floor. Mahmoodi et al (1987) presented the configuration and the position of these dampers.

- Another use of the viscoelastic damper was in the Columbia Seafirst Building [Keel and Mahmoodi (1986)]. This 76-storey building rises to a height of 940 ft above ground. To reduce the wind-induced motions, 260 viscoelastic dampers were installed alongside the main diagonal bracing members in the core of the buildings.

- A three-storey reinforced concrete structure was upgraded by viscoelastic dampers [Soong et al. (1997)]. The building is located in San Diego, CA. The lateral load resisting system was based on 8-in. reinforced concrete walls on the perimeter of the building. The results of seismic analysis of the building showed insufficiency of resisting against expected seismic loads. Accordingly, a seismic upgrade with an objective of reducing storey drifts was carried out using 64 viscoelastic dampers. Each damper consisted of four dampers forming a K-brace configuration.

- In 1995, viscous fluid dampers were used in the base isolation systems for the San Bernardino Medical Center in California. The five storey building is close to two major faults. One hundred and eighty six nonlinear fluid dampers having an output force of 315 kips and stroke of ± 24 in. were used [Soong et al. (1997)].

Different Locations of Dampers

The dampers can be installed in different locations in a building in order to improve the performance under different dynamic loads. The possible locations of the dampers in structures are:

1. In parallel with base isolators
   This is useful for structures where there are large displacements at the base.

2. Diagonal member
   The damper can be installed like a conventional diagonal brace. This type is effective for refurbishments.

3. Chevron brace
   The damper can be installed in both legs of a chevron brace. This configuration has the same effects as the diagonal members.

4. Horizontally at the top or bottom of a chevron brace
   The damper can be installed horizontally to a conventional chevron brace, whether at the top or bottom of the brace.

5. Horizontally between adjacent structures.
   If there are two structures very close to each other, the damper can be installed

horizontally between them to prevent pounding.

6. Toggle brace

   The basic idea behind this system is that the damper is placed diagonally and linked to two steel linkage elements, which are not collinear. Accordingly, any small inter-storey drift is amplified in the direction of the damper, which increases the effect of the dampers Toggle brace damper systems were used recently in the United States. Reverse toggle brace damper systems were used in the 37-storey Yerba Buena tower in San Francisco and the 37-storey Millenium Place in Boston for reducing the wind-induced vibrations.

3.3 Mathematical model and It's Behaviour

3.3.1 Mathematical Model for Viscous Damper and Viscoelastic Damper

The behavior of any viscoelastic material can be represented by a combination of elastic and viscous behaviour. For linear elastic material, the normal stress is given by:

$$\sigma = E\varepsilon \tag{3.3}$$

where E is the modulus of elasticity or $Young's$ modulus, and ε is the axial strain. For such a material, the stress and strain are time-independent. This behavior can be modeled by a linear spring.

For linear viscous material, the resisting force is proportional to the motion velocity. The stress-strain relationship for a bar with a cross sectional area, A, and length, l, can be described by:

$$\sigma A = C\dot{u} \tag{3.4}$$

where C is the proportionality constant or the damping constant. In this case, the stress and strain are time dependent. This relation can be modeled by a dashpot.There are two well known models usually used in modeling viscoelastic materials; Maxwell model and Kelvin model.

1) Maxwell Model :

In this model, the material is modeled by a spring and a dashpot in series. The model is shown in Figure3.6. The stress and strain are given by:

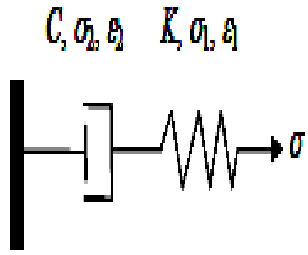

Figure 3.6: Maxwell Model

$$\sigma = \sigma_1 = \sigma_2 \tag{3.5}$$

$$\varepsilon = \varepsilon_1 + \varepsilon_2 \tag{3.6}$$

where $\sigma_1$and ε_1 are the stress and strain in the linear spring while $\sigma_2$and ε_2 are the stress and strain in the dashpot.

The force in the fluid viscous damper may be expressed as,

$$P(t) = Cd|\dot{u}|^{\alpha} sgn(\dot{u}) \tag{3.7}$$

Where, C_d is the damping coefficient for the damper, α is the velocity exponent for the damper that ranges from 0.1 to 2, u is the relative velocity between each end of the device and sgn is the signum function that, define the sign of the relative velocity term. A value of $\alpha = 1.0$ represents the linear viscous damper. Structural dampers usually have α values ranging from 0.3 - 1.0. The main advantage of the linear viscous dampers is that there is very little interaction between damper forces and structural forces. Maximum structural forces occur at maximum displacement, at which the damper forces are zero because the deformational velocity in the damper is near zero. The value of the resisting force in linear viscous fluid damper varies with respect to the translational velocity of the damper at any point in time is given by,

$$P(t) = C_d|\dot{u}|t \tag{3.8}$$

Where, P(t) is the resistance force for linear viscous damper. C_d and $\dot{u}$ are the damping coefficient and displacement of the dampers respectively. The energy dissipation by the damper can be find out from the following equation,

$$E_d = \int |P(t)|du \tag{3.9}$$

The area contained within the hysteretic loop present in Figure3.7 , measures the energy dissipated per cycle in the viscous damper

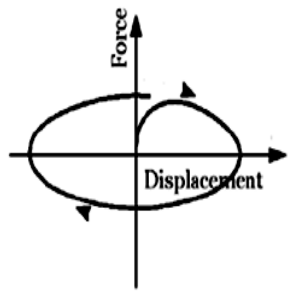

Figure 3.7: Hysteretic loop for Maxwell Model

2) Kelvin Model :

In this model, the material is modeled by a spring and a dashpot in parallel. The model is shown in Figure3.8. The stress and strain are given by:

$$\sigma = \sigma_1 = \sigma_2 \tag{3.10}$$

$$\varepsilon = \varepsilon_1 + \varepsilon_2 \tag{3.11}$$

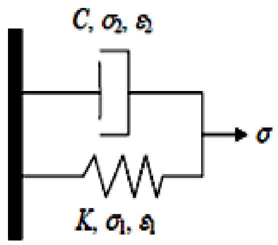

Figure 3.8: Kelvin Model

The addition of dampers into a structure not only increases the stiffness of the structure but also provides a significant amount of damping. It is thus necessary to take into account such changes in the analysis and design of the structure with added dampers. Furthermore, the increased application of velocity dependent dampers in structures will depend on the availability of simplified methods for the analysis and design. Energy is dissipated through large shear strains in the viscoelastic material. Implementation of viscoelastic dampers causes a small increase in structural stiffness due to the inherent storage stiffness of the viscoelastic material. One of the primary advantages of the viscoelastic dampers is that they dissipate energy under all levels of ground motion. As suggested in the FEMA-273 [14] guidelines, solid viscoelastic devices may be modeled using a classical Kelvin Model in which a linear spring is placed in parallel with a viscous dashpot.

Most of the mechanical properties of viscoelastic materials are rather complex and may vary with environmental temperature and excitation frequency. The best method of evaluating the properties of the damper is to generate the hysteresis loop by subjecting the center part of the damper to a periodic displacement then plotting this and the corresponding shear force on an x-y recorder as shown in Figure3.9 for one cycle. The area of the hysteresis loop represents the actual energy lost or damped.

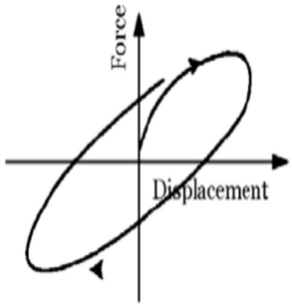

Figure 3.9: Hysteretic loop for Kelvin Model

3.3.2 Response of Viscous Damper and Viscoelastic Damper

Fluid viscous device may be modeled using a spring and dashpot in series, i.e, Maxwell Model. For characterization, the damping coefficient of viscous fluid damper Cd=160 kNsec/cm is considered as given in literature. The plots of force v/s displacement, force v/s velocity and force v/s acceleration for 1 Hz frequency and different value of amplitude (A) as shown in Figure3.10. Same way, for different frequency with constant amplitude, we can also plot the graph for force v/s displacement, force v/s velocity and force v/s acceleration as shown in Figure3.11

In general, earthquakes have different properties such as Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), Peak Ground Displacement (PGD), duration of strong motion and ranges of dominant frequencies; hence they have different influence on the structures. The time history data was taken from Pacific Earthquake Engineering Research Institute" (PEER). The earthquake time history records, which are selected for this study to investigate the dynamic response of viscous damper models are summarized in Table3.1 below.

Table 3.1: Time History Data for Various Earthquakes

Earthquake	Year	PGA (g)	PGV (cm/sec^2)	PGD (cm)	Damping
El centro	1940	0.3129	43.8	18.3	0.05
Kobe	1995	0.6936	37.3	9.52	0.05
Lomaprieta	1989	0.6437	94.8	41.18	0.05
Northridge	1994	1.585	103.9	23.8	0.05

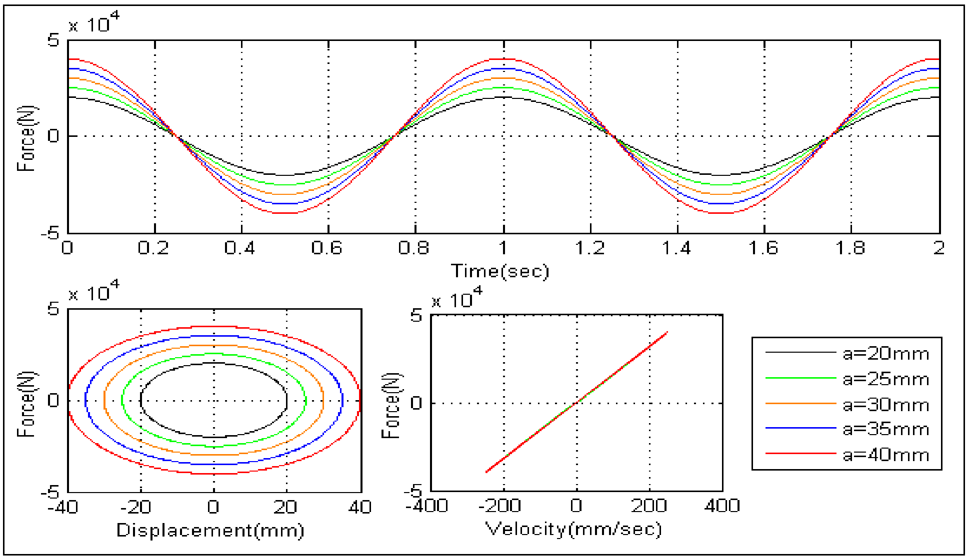

Figure 3.10: response for different amplitude of motion

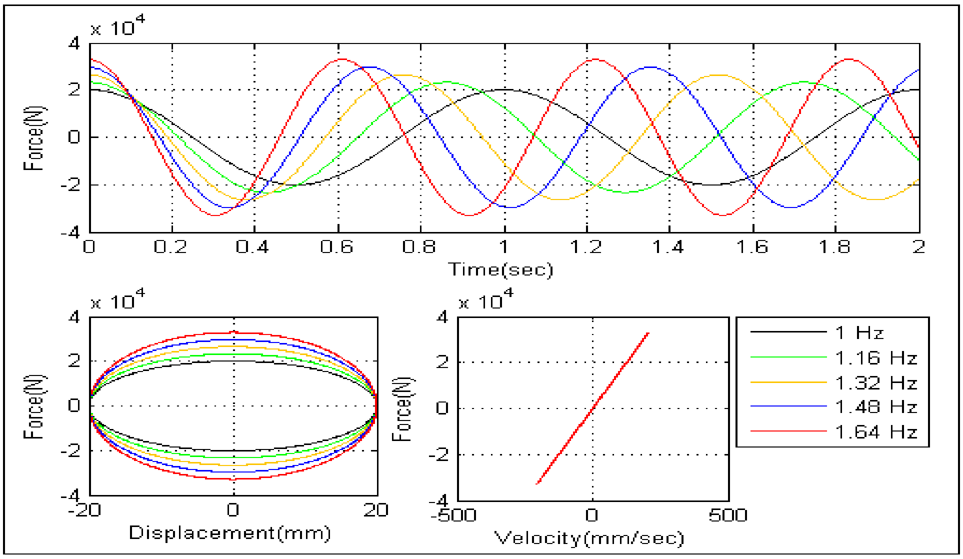

Figure 3.11: response for different amplitude of motion

The response of damper under different earthquake excitation is different with varies the different properties such as peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), duration of strong motion and ranges of dominant frequencies. The response of Viscous damper under El Centro earthquake excitation is shown in Figure3.12 and similarly the response of viscous damper under other earthquake like Kobe,Loma Prieta and North Ridge are also found out.

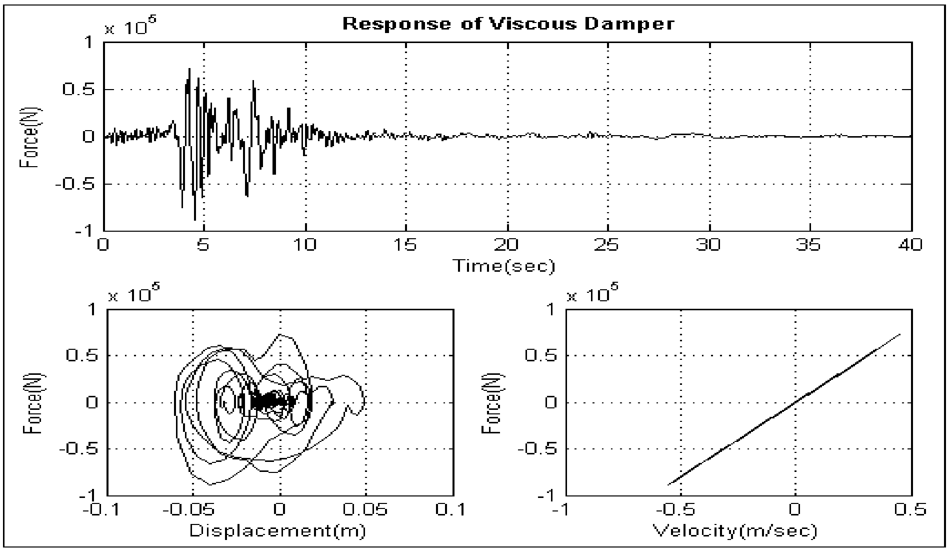

Figure 3.12: Response under Elcentro EQ

3.3.3 Response of Viscoelastic Damper

For this study, VE damper size (A=50.8mm ∗38.1) mm and thickness (t=7.62mm) is consider as specified in literature **??** Damper properties storage modulus G0=958370.25 N/m^2), loss modulus (G00=1151423.25 $N = m^2$) and loss factor(η=1.2) are taken for excitation frequency 1 Hz, ambient temperature 240 C and damper strain 20 %. From these data stiffness coefficient (kd=468.85 N=mm) and damping coefficient (Cd=93.093 NSec/mm) for a viscoelastic damper are calculated as per Appendix B. The plot of Force Vs Time, Force Vs Displacement, and Force Vs Velocity of VE damper subjected to sinusoidal excitations with fixed frequency of 1 Hz and different value of amplitude (a= 0.75, 1, 1.25, 1.50, 1.75 and 2 mm) are obtained out through

MATLAB as shown in Figure 3.13. By considering the same damper dimension and properties, simulation of the damper response under sinusoidal excitations for 1 *mm* amplitude and different excitation of frequencies (1, 1.16, 1.32,1.48, 1.64 and 1.80 Hz) are carried out and the same are shown in Figure3.14.

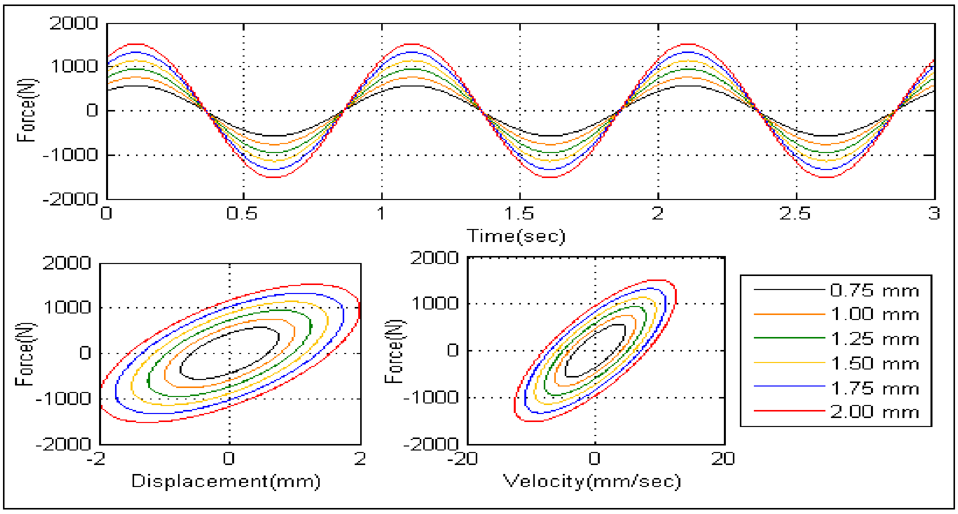

Figure 3.13: response for different amplitude of motion

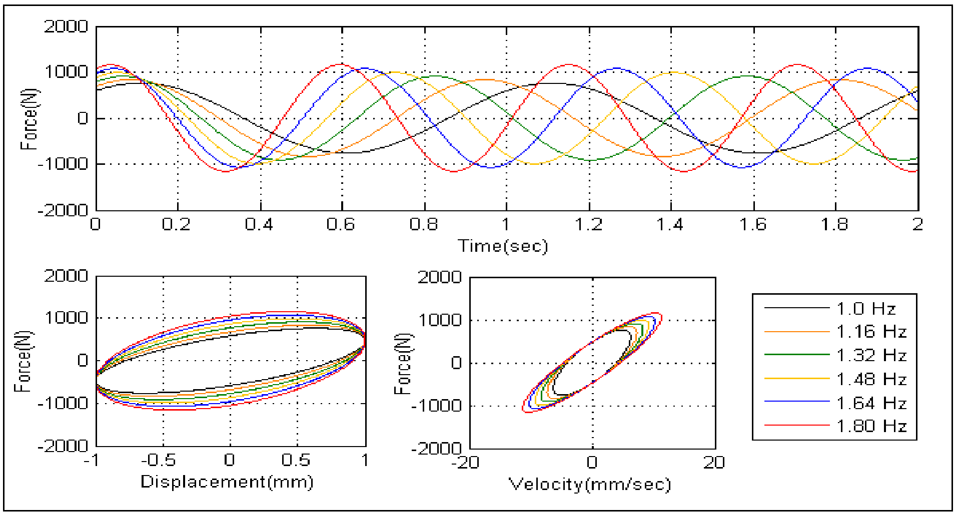

Figure 3.14: response for different amplitude of motion

The response of damper under different earthquake excitation is different with varies the different properties such as peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), duration of strong motion and ranges of dominant frequencies. The response of Viscous damper under El Centro earthquake excitation is shown in Figure3.15 and similarly the response of viscous damper under other earthquake like Kobe,Loma Prieta and North Ridge are also found out.

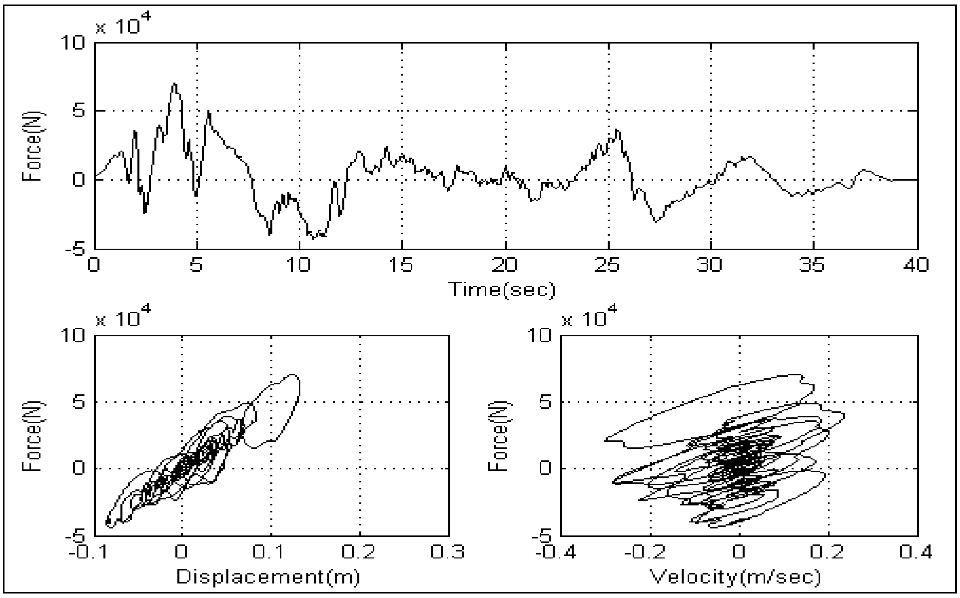

Figure 3.15: response of VE damper under El centro EQ

3.4 Summary

This chapter deals with the detail of mathematical model of passive control devices like viscous and viscoelastic damper. To understand the behaviour of viscous and viscoelastic damper and it's characterization have been carried out through MATLAB under sinusoidal and different earthquake excitations namely, El Centro, Kobe, Loma Prieta and North Ridge excitations and Force v/s time, Force v/s displacement and Force v/s velocity plots are obtained.

Chapter 4

Three Storey Shear building problem

4.1 General

3-storey Reinforced concrete building is considered for dynamic analysis by using MATLAB. The equation of motion for uncontrolled structure (without damper) is derived. Then equation of motion for controlled structure (with damper) with varies damper location is de-rived. Then equation of motion for viscous damper and Viscoelastic damper at various storey of the structure is derived. With the help of different earthquake excitation we can calculate different response quantity like displacement, velocity, acceleration and inter storey drift.

4.2 Building Configuration

The building configurations are as under:

- No. of Storey = G+2 Storey
- storey Height = 3 m
- No. of Bays in (X and Y) Direction = 3 m

- Bay Width in both (X and Y) Direction = 4 m
- Slab Thickness. = 120 mm
- Column Size = 0.3 m * 0.3 m
- Beam Size = 0.23 m * 0.3 m
- f_{ck}= 25 N/mm^2 (M 20 grade of concrete)
- f_y = 415 N/mm^2 (Fe 415 grade of steel)
- Live Load on Typical Storey = 3 kN/m^2

As shown in Figure4.1 the no. of bays and width on both (X and Y) direction are same.Using the lumped mass system the dynamic property of the structure like mass matrix and stiffness matrix are derived.With the help of Rayleigh's damping the damping matrix is also derived. The detail calculation of mass, stiffness and damping matrix are given in Appendix-A.

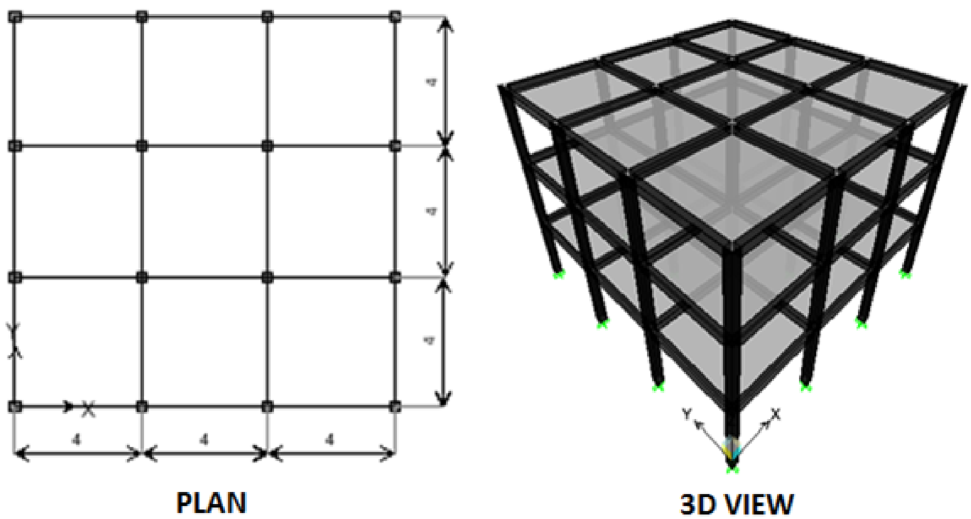

Figure 4.1: Three storey buildings plan and 3d view

4.3 Equation of Motion for Uncontrolled Building

Three storey reinforced concrete building is considered for dynamic analysis. For simplicity, the building is considered as rigid so replace the distribution mass into the lumped mass model. Linear or angular coordinates are used to describe mass model. Assume that the components of the structures are connected as a spring mass model. The 3-D building is continuous system and its require no. of degree of freedom so, the slab is considered as rigid diaphragm.

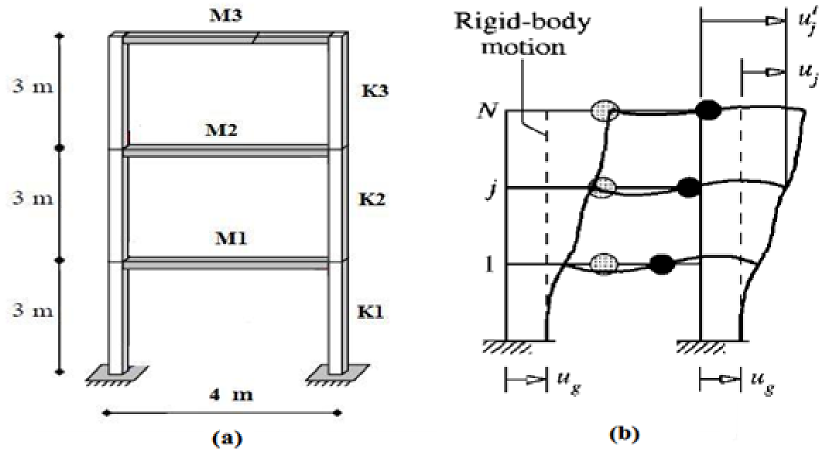

Figure 4.2: Three Storey Building: a) Lumped Mass Model, b) Building Frame under Ground Excitation

Figure4.2shows the simplified model of building with degree of freedom associated for present study.For finding out the damping forces and inertia forces, considered the building is subjected to the earthquake ground motion. The building is rigidly connected with each other so, the beam and floor are also connected rigid in flexure. The mass is distributed throughout the building, but it is idealized as concentrated at the floor levels. The building is as shown in Figure 4.2 has lumped mass at each floor level with three degree of freedoms and the lateral displacement for the same is u_1, u_2 and u_3 in the direction of x axis.

$D'Alembert's$ principle which states that a system may be set in a state of dynamic equilibrium by adding to the external forces a fictitious force which is commonly known as inertial force. It is based on the motion of a fictitious inertia force, a force equal to the product of mass times its acceleration and acting in a direction opposite to the acceleration. It states that with inertia force included, a system is in equilibrium at each time instant. Thus a free body diagram of a moving mass can be drawn, and principles of statics can be used to developed the equation of motion. The displacement of ground is denoted by u_g the total or absolute displacement of mass by u^t and the relative displacement between the mass and ground by u at each instant of time, these displacements are related by,

$$u^t(t) = u(t) + u_g(t) \tag{4.1}$$

Both u^t and u_g refer to the same inertial frame of reference and their positive directions coincide. Similarly for all number of masses, it can be combined in vector form:

$$u^t(t) = u(t) + u_g(t)l \tag{4.2}$$

Where, the influence vector 'l' represents the displacement of the masses resulting from the static application of a unit ground displacement.
The equation of motion for the building of Figure4.2 subjected to earthquake excitation can be derived by concept of dynamic equilibrium from the free body diagram including the inertia force. From the free body diagram as shown in Figure4.3, the equation of dynamic equilibrium is

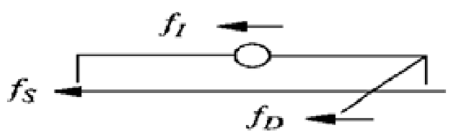

Figure 4.3: Free Body Diagram for Force

$$f_{(I)} + f_{(D)} + f_{(c)} = 0 \tag{4.3}$$

Only the relative motion u between the mass and the base due to structural deformation produces elastic and damping forces. Thus for a linear system the damping force is,

$$f_{(D)} = C\dot{u} \tag{4.4}$$

And elastic resisting force is,

$$f_{(s)} = Ku \tag{4.5}$$

The inertia force f_I is related to the total acceleration $\ddot{u}^t$ at the mass by,

$$f_{(I)} = m\ddot{u}^t \tag{4.6}$$

Substituting Equation4.4, 4.5 and 4.6, in equation4.3, and using equation4.2.,

$$m\ddot{u}^t + c\dot{u} + k(u) = 0 \tag{4.7}$$

The above equation is known as the equation of motion for the building subjected to earth-quake excitation. Where $\ddot{u}_g$(t) is the ground motion acceleration and m, c, and k are the mass, damping and stiffness matrix respectively. For building with n degree of freedom, the size of matrix are $[m]$,$[c]$,and$[k]$ is n*n.

4.4 Equation of Motion for Building with Passive Devices

The use of Passive energy dissipation devices are well accepted for the reducing the response of the structures exposed to earthquake induced ground motion. Due to the addition of passive devices, the total structural damping and structural stiffness is increase which helps us to reduce the effect of the earthquake excitation. At here, the damper is located for following condition:

1. Damper at Ground Floor (GF)
2. Damper at First Floor(FF)
3. Damper at Second Floor (SF)
4. Damper at Ground Floor and First Floor (GF FF)
5. Damper at Ground Floor and Second Floor (GF SF)
6. Damper at First Floor and Second Floor (FF SF)
7. Damper at Ground Floor, First Floor and Second Floor (GF FF SF)

For a shear building with added passive dampers subjected to earthquake excitation, the equation of motion of the system combining building and dampers can be written as,

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -ml\ddot{u}_g(t) - BF \tag{4.8}$$

where,
m, k and c are the mass matrix, stiffness matrix and damping matrix respectively.
u = The vector of the relative displacements of the floors of the building.
l= Influence vector.
$\ddot{u}_g$= The earthquake acceleration excitation.
B = the matrix derived based on placement of passive devices in the building.
F = $[F1, F2, F3...Fn]$T is the vector of control forces produced by passive dampers.Here n is the number of floor of the building.= $C_d\dot{u}(t)$

The equations of motion of the multi-storey structure with viscous damper under the external excitation that is earthquake ground motion, then P_{eff}=-ml$\ddot{u}_g$(t), in which $\ddot{u}_g$(t) is the earthquake ground acceleration and 'l' is an identity matrix so Equation 4.8 can then be expressed as,

$$m\ddot{u}(t) + c\dot{u}(t) + Ku(t) = -ml\ddot{u}_g(t) - Bc_d\dot{u}(t) \tag{4.9}$$

$$m\ddot{u}(t) + (c + Bc_d)\dot{u}(t) + Ku(t) = -ml\ddot{u}_g(t) \tag{4.10}$$

Equation4.10 is the equation of motion for multi degree of structure with viscous damper. Depending on the damper diameter and orifice area, the damping coefficient c_d can be determined and is an important variable in Equation4.10 Where, c is the matrix due to structural inherent damping Bc_d is The additional damping due to viscous damper in the building. Similarly for viscoelastic damper, the control force F produces due to stiffness coefficient k_d and damping coefficient c_d. The equation of motion for the multi degree of freedom shear type building with viscoelastic damper can then be expressed as,

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -ml\ddot{u}_g(t) - B[kdu(t) + cd\dot{u}(t)] \tag{4.11}$$

$$m\ddot{u}(t) + (c + B_{cd})\dot{u}(t) + ku(t) + (k + Bk_d)u(t) = -ml\ddot{u}_g(t) \tag{4.12}$$

Where,

k and c are the matrix due to structural storey stiffness and structural inherent damping, respectively. Bk_d and B$c_d$are the matrix due to the addition of viscoelastic dampers stiffness and damp-ing respectively, in the building. Based on all above condition, with varies the damper location the equation of motion is also changed. The equation of motion for all above condition is found out as below:

4.4.1 Equation of motion when damper is at ground floor

The damper is provided to the ground floor as shown in Figure4.4. The vector matrix is varies with change the damper location. When the damper is at ground floor the vector matrix B is as shown below,

$$B = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{4.13}$$

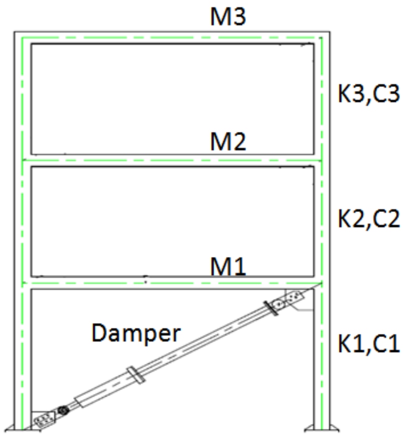

Figure 4.4: Damper at GF

Based on above B matrix, the equation of motion for viscous damper from equation4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -c2 & 0 \\ -c2 & c2+c3 & -c3 \\ 0 & -c3 & c3 \end{bmatrix} \times \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \quad (4.14)$$

Similarly, the equation of motion for viscoelastic damper from Equation4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -c2 & 0 \\ -c2 & c2+c3 & -c3 \\ 0 & -c3 & c3 \end{bmatrix} \times \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+kd & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \quad (4.15)$$

4.4.2 Equation of motion when damper is at first floor

The damper is provided to the first floor as shown in figure4.5. The vector matrix is varies with change the damper location. When the damper is at first floor the vector matrix B is

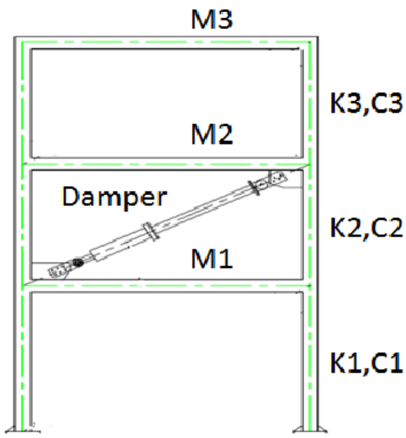

Figure 4.5: Damper at FF

$$B = \begin{bmatrix} 1 & -1 & 0 \\ -1 & 1 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{4.16}$$

Based on above B matrix, the equation of motion for viscous damper from equation 4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -(c2+cd) & 0 \\ -(c2+cd) & c2+c3+cd & -c3 \\ 0 & -c3 & c3 \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.17}$$

Similarly, the equation of motion for viscoelastic damper from Equation 4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -(c2+cd) & 0 \\ -(c2+cd) & c2+c3+cd & -c3 \\ 0 & -c3 & c3 \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+kd & -(k2+kd) & 0 \\ -(k2+kd) & k2+k3+kd & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \quad (4.18)$$

4.4.3 Equation of motion when damper is at Second floor

The damper is provided to the second floor as shown in figure4.6. The vector matrix is varies with change the damper location. When the damper is at ground floor the vector matrix B is

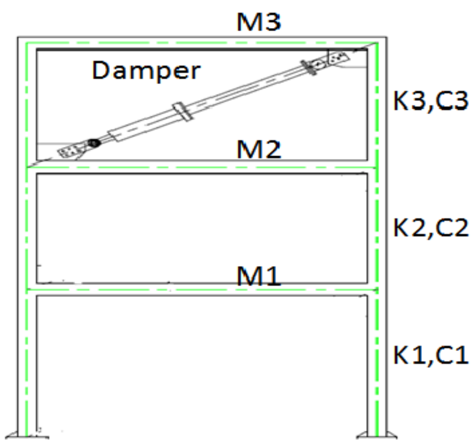

Figure 4.6: Damper at SF

$$B = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 1 & -1 \\ 0 & -1 & 1 \end{bmatrix} \quad (4.19)$$

Based on above B matrix, the equation of motion for viscous damper from equation 4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x1} \\ \ddot{x2} \\ \ddot{x3} \end{bmatrix} + \begin{bmatrix} c1+c2 & -c2 & 0 \\ -c2 & c2+c3+cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x1} \\ \dot{x2} \\ \dot{x3} \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \quad (4.20)$$

Similarly, the equation of motion for viscoelastic damper from Equation4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x1} \\ \ddot{x2} \\ \ddot{x3} \end{bmatrix} + \begin{bmatrix} c1+c2 & -c2 & 0 \\ -c2 & c2+c3+cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x1} \\ \dot{x2} \\ \dot{x3} \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3+kd & -(k3+kd) \\ 0 & -(k3+kd) & k3+kd \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \quad (4.21)$$

4.4.4 Equation of motion when damper is at Ground Floor and First Floor

The damper is provided to the ground floor and first floor as shown in figure4.7. The vector matrix is varies with change the damper location. When the damper is at ground floor the vector matrix B is

$$B = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & -1 \\ 0 & -1 & 1 \end{bmatrix} \quad (4.22)$$

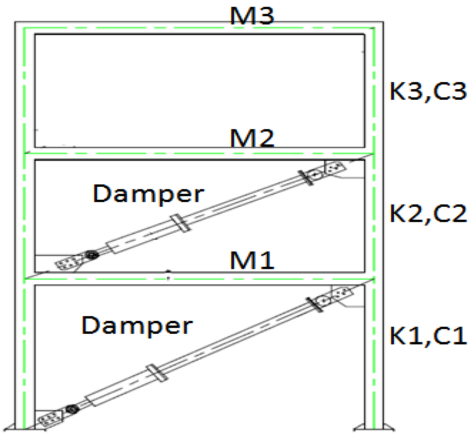

Figure 4.7: Damper at GF and FF

Based on above B matrix, the equation of motion for viscous damper from Equation4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+2cd & -c2+cd & 0 \\ -c2+cd & c2+c3+cd & -c3 \\ 0 & -c3 & c3 \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.23}$$

Similarly, the equation of motion for viscoelastic damper from equation4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+2cd & -c2+cd & 0 \\ -c2+cd & c2+c3+cd & -c3 \\ 0 & -c3 & c3 \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+2kd & -(k2+kd) & 0 \\ -(k2+kd) & k2+k3+kd & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.24}$$

4.4.5 Equation of motion when damper is at Ground Floor and Second Floor

The damper is provided to the ground floor and second floor as shown in figure4.8. The vector matrix is varies with change the damper location. When the damper is at ground floor the vector matrix B is

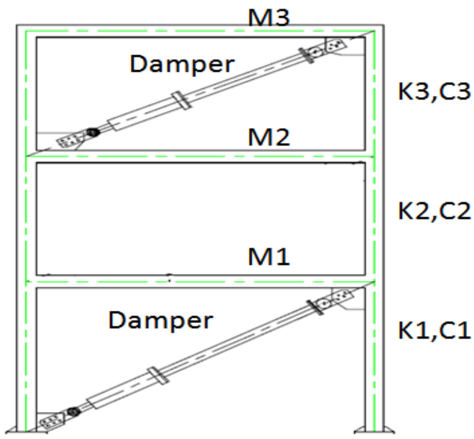

Figure 4.8: Damper at GF and SF

$$B = \begin{bmatrix} 2 & -1 & 0 \\ -1 & 1 & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{4.25}$$

Based on above B matrix, the equation of motion for viscous damper from equation4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -c2 & 0 \\ -c2 & c2+c3+cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+kd & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.26}$$

Similarly, the equation of motion for viscoelastic damper from equation4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -c2 & 0 \\ -c2 & c2+c3+cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+kd & -k2 & 0 \\ -k2 & k2+k3+kd & -(k3+kd) \\ 0 & -(k3+kd) & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.27}$$

4.4.6 Equation of motion when damper is at First Floor and Second Floor

The damper is provided to the first floor and second floor as shown in figure4.9. The vector matrix is varies with change the damper location. When the damper is at ground floor the vector matrix B is

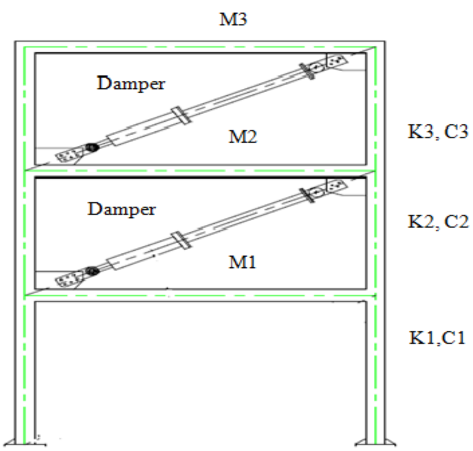

Figure 4.9: Damper at FF and SF

$$B = \begin{bmatrix} 1 & -1 & 0 \\ -1 & 2 & -1 \\ 0 & -1 & 0 \end{bmatrix} \tag{4.28}$$

Based on above B matrix, the equation of motion for viscous damper from equation4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -(c2+cd) & 0 \\ -(c2+cd) & c2+c3+2cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.29}$$

Similarly, the equation of motion for viscoelastic damper from equation4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+cd & -(c2+cd) & 0 \\ -(c2+cd) & c2+c3+2cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+kd & -(k2+kd) & 0 \\ -(k2+kd) & k2+k3+2kd & -(k3+kd) \\ 0 & -(k3+kd) & k3+kd \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.30}$$

4.4.7 Equation of motion when damper is at Ground Floor , First Floor and Second Floor

The damper is provided to the ground floor, first floor and second floor as shown in Figure4.10. The vector matrix is varies with change the damper location. When the damper is at ground floor the vector matrix B is

$$B = \begin{bmatrix} 2 & -1 & 0 \\ -1 & 2 & -1 \\ 0 & -1 & 1 \end{bmatrix} \tag{4.31}$$

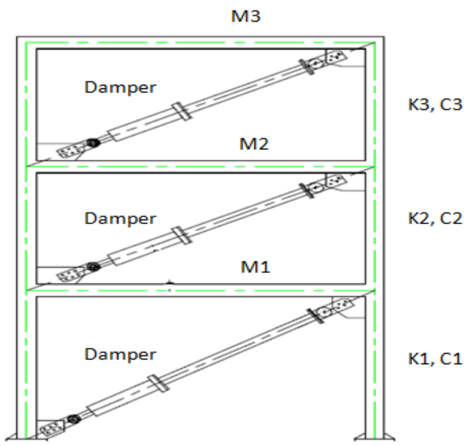

Figure 4.10: Damper at GF FF and SF

Based on above B matrix, the equation of motion for viscous damper from equation4.10 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+2cd & -(c2+cd) & 0 \\ -(c2+cd) & c2+c3+2cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2 & -k2 & 0 \\ -k2 & k2+k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.32}$$

Similarly, the equation of motion for viscoelastic damper from equation4.12 becomes,

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \times \begin{bmatrix} \ddot{x}1 \\ \ddot{x}2 \\ \ddot{x}3 \end{bmatrix} + \begin{bmatrix} c1+c2+2cd & -(c2+cd) & 0 \\ -(c2+cd) & c2+c3+2cd & -(c3+cd) \\ 0 & -(c3+cd) & c3+cd \end{bmatrix} \begin{bmatrix} \dot{x}1 \\ \dot{x}2 \\ \dot{x}3 \end{bmatrix} + \begin{bmatrix} k1+k2+2kd & -(k2+kd) & 0 \\ -(k2+kd) & k2+k3+2kd & -(k3+kd) \\ 0 & -(k3+kd) & k3+kd \end{bmatrix} \times \begin{bmatrix} x1 \\ x2 \\ x3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{4.33}$$

4.5 Solution of Equation of Motion using Numerical Method

Analytical solution of equation of the motion for a multi degree of freedom system is usually not possible if the excitation-applied force or ground acceleration varies arbitrarily with time. Such problem can be solved by the numerical time-stepping methods for integration of differential equations. There are two basic approaches to numerically evaluate the dynamic response. The first approach is numerical interpolation of the excitation and the second is numerical integration of the equation of motion. Both approaches are applicable to linear systems but the second approach is related to non-linear systems.

There are number of numerical methods are available for solving the problem of equation of motion which describe in previous section. But all the numerical integration method has two basic characteristics. First, they do not satisfy differential equation at all time t, but only at discrete time intervals, say Δ(t) apart. Secondly, within each time interval (t), a specific type of variation of the displacement u, velocity $\dot{u}$, and acceleration $\ddot{u}$ is assumed. Thus, several numerical integration methods are available depending on the type of variation assumed for u, $\dot{u}$ and $\ddot{u}$ within each time interval Δ(t).

4.5.1 Time stepping Methods

Equation of motion in the case of base excitation due to earthquake is given as,

$$m\ddot{u}(t) + c\dot{u}(t) + Ku(t) = -ml\ddot{u}_g(t) \tag{4.34}$$

Now, subject to initial conditions

$$u_o = u(o) and \dot{u}_o = \dot{u}(o) \tag{4.35}$$

Assumed the system is linear damping, but other forms of damping, including non-linear damping can be considered. The applied force at discrete time intervals and the time increment Δt_i=t_{i+1}-t_i is usually take to be constant, although this is not necessary. The response is determining at discrete time instants t_i, denoted as time i; the displacement, velocity, and acceleration at the i^{th} step are denoted by u_i, $\dot{u}_i$ and $\ddot{u}_i$ respectively. These values are assumed to satisfy Equation4.36 at time i: as,

$$m\ddot{u}_i + c\dot{u}_i + Ku_i = p_i \tag{4.36}$$

Where ku_i is the resisting force at time i; for linearly elastic but would depend on the prior history of displacement and velocity at time i if the system were inelastic. In subsequent section numerical procedure is presented, which enable us to determine the response quantities u_{i+1}, $\dot{u}_{i+1}$ and $\ddot{u}_{i+1}$at time (i+1) step that satisfy Equation4.36 at time i+1:

$$m\ddot{u}_{i+1} + c\dot{u}_{i+1} + Ku_{i+1} = p_{i+1} \tag{4.37}$$

If the numerical procedure is applied successively with i = 0, 1, 2, 3 ... The time stepping procedure gives the desired response at all times with the known initial conditions u_0 and $\dot{u}_0$.

Types of Time Stepping Methods

Three types of time stepping procedures are as follows:

1. Method based on the interpolation of the excitation function.
2. Method based on finite difference expressions for the velocity and acceleration.
3. Method based on the assumed variation of acceleration.

In a direct integration method, the system of equation of motion is integrated successively by using step by step numerical method. No transformation of equation of motion is needed prior to integration and using difference formulas that involve one or more increments of time usually approximates time derivatives. Basically two principle approaches used in the direct integration method: Explicit and implicit schemes. In an explicit scheme, the response quanti-ties are expressed in terms of previously determined value of displacement, velocity, and acceleration. In an implicit scheme the difference equations are combine with the equation of motion, and the displacements are calculated directly by the solving the equation.

4.5.2 Newmark Beta Method[20, 21]

The well known Newmark direct integration method is quite often used to compute the structural response, and hence in this section we intend to formulate a procedure that incorporates the Newmark type numerical scheme in solving the equation of motion with and without passive devices under the earthquake excitations.

The Newmark Beta integration method is based on the assumption that the acceleration varies linearly between two instants of time. Two parameter α and β are used in this method, which can be suit the requirement of the particular problem. Newmark [20] presented a family of time-step methods for the solution of structural dynamics problem for both blast and seismic loading. In order to illustrate the use of this numerical integration method, consider the solution of linear dynamic equilibrium equations of motion as given in Equation4.37. Newmark developed a family of time-stepping methods based on the following equations:

$$\dot{u}_{i+1} = \dot{u}_i + [(1-\gamma)\Delta_t]\ddot{u}_i + (\gamma\Delta_t)\ddot{u}_{i+1} \tag{4.38}$$

$$u_{i+1} = u_i + (\Delta t)\dot{u}_i + [(0.5-\beta)(\Delta t)^2]\ddot{u}_i + [\beta(\Delta t)^2]\ddot{u}_{i+1} \tag{4.39}$$

Newmark used Equations4.37,4.38 and 4.39 iteratively for each time step, for each displacement DOF of the structural system. The parameter β and γ define the variation of acceleration over a time step and determine the stability and accuracy characteristics of the method. Typical selection for γ is 1/2 and $1/6 \leq \beta \leq 1/4$ is satisfactory from all points of view, including that of accuracy. These two equations, combined with the equilibrium Equation4.37 at the end of the time step, provide the basis for computing u_{i+1}, $\dot{u}_{i+1}$ and $\ddot{u}_{i+1}$ at time (i+1) from the known u_i, $\dot{u}_i$ and $\ddot{u}_i$ at time i. Iteration is required to implement these computations because the unknown

$\ddot{u}_{i+1}$ appears in the right side of Equation4.38 and 4.39. The parameter γ and β indicate how much acceleration enters into the displacement and velocity equations at the end of the interval Δt. Therefore, γ and β are chosen to obtain the desired integration accuracy and stability. When γ =1/2 and β =1/6, Equations4.38 and 4.39 correspond to the linear acceleration method. Whenγ =1/2 and β =1/4, this correspond to the assumption that the acceleration remain constant. The complete algorithm using the Newmark Beta integration method is given below.

Newmark's Direct Integration Method[20, 21]

1) Initial calculation

(1.1) Form static stiffness matrix [k], mass matrix [m] and damping matrix [c]

(1.2) Specify integration parameter γ and β

(1.3) Select Δt

(1.4) Specify initial conditions u_0, $\dot{u}_0$, $\ddot{u}_0$

(1.5) $\ddot{u}_0 = \frac{p_0 - c\dot{u}_0 - ku_0}{m}$

(1.6) Calculate constants, a = $\frac{1}{\beta\Delta t}$m + $\frac{\gamma}{\beta}$c; and b = $\frac{1}{2\beta}$m + Δt($\frac{\gamma}{2\beta}$-1)c.

(1.7) Calculate modified stiffness, $\widehat{k}$ = k + $\frac{\gamma}{\beta\Delta t}$c + $\frac{1}{\beta(\Delta t)^2}$m.

2) Calculation for each time step, *i*

(2.1) $\Delta\ \widehat{p_i} = \Delta p_i$ + a$\dot{u}_i$ + b$\ddot{u}_i$

(2.2) $\Delta u_i = \frac{\Delta\widehat{p}_i}{\widehat{k}}$

(2.3) $\Delta\ \dot{u}_i = \frac{\gamma}{\beta\Delta t}\Delta u_i$ - $\frac{\gamma}{\beta}\ \dot{u}_i$ + Δt(1-$\frac{\gamma}{2\beta}$)$\ddot{u}_i$.

(2.4) $\Delta\ \ddot{u}_i = \frac{1}{\beta(\Delta t)^2}\Delta u_i$ - $\frac{1}{\beta\Delta t}\ \dot{u}_i$ - $\frac{1}{2\beta}\ddot{u}_i$

(2.5) $u_{i+1} = u_i + \Delta u_i$, $\dot{u}_{i+1} = \dot{u}_i + \Delta\dot{u}_i$ and $\ddot{u}_{i+1} = \ddot{u}_i + \Delta\ddot{u}_i$

3) Repetition for the next time step. Replace *i* by *i* + 1 and implement steps 2.1 to 2.5 for the next time step.

4.6 Response of Uncontrolled Shear Building

In this section, four different types of earthquake excitations are used to find the response of uncontrolled shear building. Earthquake excitation considered are, El Centro, Loma Prieta, Kobe, and Northridge, where first two excitation are strong motion type while later two excitation are pulse type motion. In order to obtain response quantity equation of motion given by Equation4.8 is solved using Newmark-Beta method discussed in Section 4.5 through writing code in MATLAB. Response quantities like displacement, acceleration, inter storey drift and velocity is extracted for a shear building.

4.7 Result and Discussions

Table 4.1 shows the maximum response quantity obtained for uncontrolled building under El Centro, Kobe, loma prieta and North Ridge earthquake excitation.

Table 4.1: Response Quantity for Uncontrolled Structure under different EQ excitation

		El Cectro	Kobe	Loma Prieta	North Ridge
Maximum Displacement (mm)	1^{st}	0.010	0.031	0.029	0.044
	2^{nd}	0.018	0.055	0.051	0.078
	roof	0.023	0.066	0.062	0.095
Maximum Velocity (m/sec)	1^{st}	0.197	0.523	0.553	0.770
	2^{nd}	0.327	0.864	1.007	1.508
	roof	0.386	0.999	1.251	1.904
Maximum (m/sec^2) Acceleration	1^{st}	5.658	11.040	10.279	16.318
	2^{nd}	6.335	17.430	16.791	25.418
	roof	8.391	21.328	21.119	31.330
Maximum Inter Storey Drift (mm)	1^{st}	0.010	0.031	0.029	0.044
	2^{nd}	0.008	0.024	0.022	0.034
	roof	0.004	0.012	0.012	0.017

The number of storeys increases the maximum displacement, maximum velocity and maxi-mum acceleration are at top storey of the building and the inter storey drift is minimum at top storey and maximum at ground storey. Time history plot of response quantities like, displacement, acceleration and velocity is obtained. The time history plot of displacement, velocity and acceleration for top storey of the building is shown in Figure4.11. From the Figure4.11, it seen that maximum displacement is 23 mm, maximum velocity is 38.6 cm/sec and maximum acceleration is 838 cm/s^2. It is also observed that, response quantities shows in-creased response when frequency of earthquake excitation increases.

Similarly, response quantity like displacement, velocity, and acceleration are also obtained for uncontrolled building under Kobe, Loma Prieta, and Northridge earthquake excitations which was shown in Table4.1. Table4.1 shows that maximum displacement, maximum velocity, and maximum acceleration increases with storey numbers, however inter storey drift is maximum at lowest storey and decreases with storey numbers.Figure 4.12 to 4.14 shows time history plot of displacement, velocity and acceleration for top storey of the building for Kobe, Loma prieta and Northridge earthquake. It is seen that maximum displacement is 95 mm, maximum velocity is 190.4 cm/sec and maximum acceleration is 3133.0cm/s^2. It is also observed that, response quantities shows increased response when frequency of earthquake excitation increases.

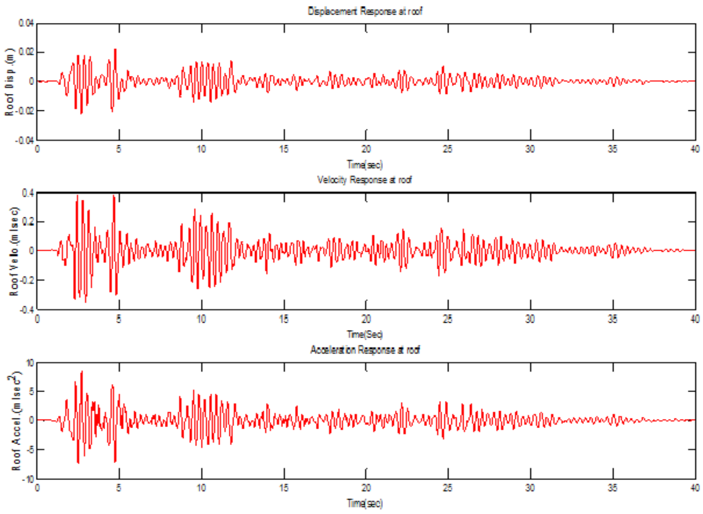

Figure 4.11: Uncontrolled Building response at roof under El centro EQ excitation

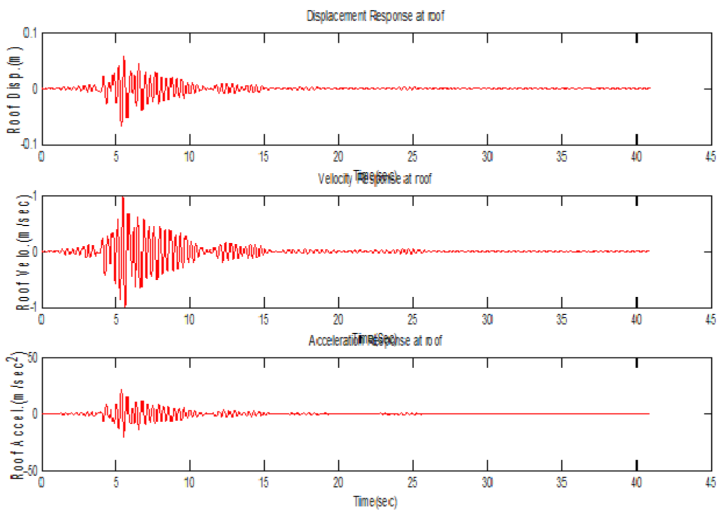

Figure 4.12: Uncontrolled Building response at roof under Kobe EQ excitation

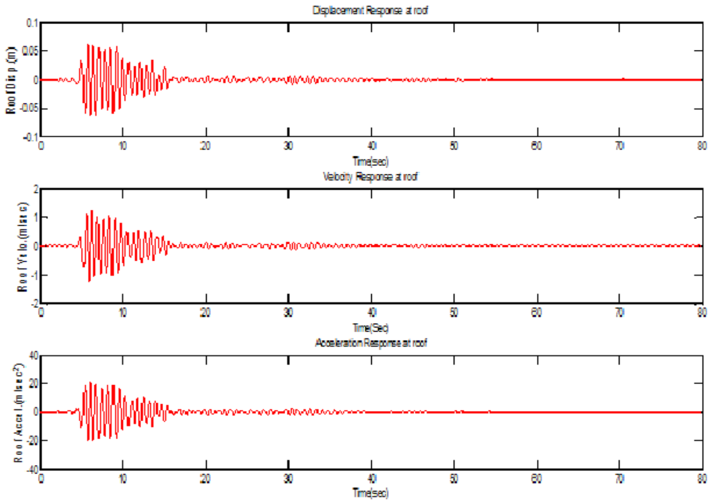

Figure 4.13: Uncontrolled Building response at roof under Loma Prieta EQ excitation

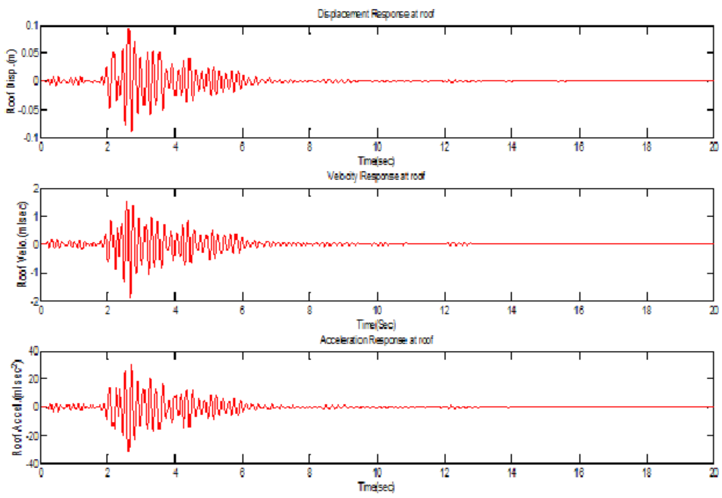

Figure 4.14: Uncontrolled Building response at roof under Northridge EQ excitation

4.8 Summary

The chapter deals with the dynamic response of uncontrolled shear building. Equation of motion for uncontrolled and controlled building with passive devices like viscous, viscoelastic and metallic damper are derived. Using Newmark-Beta method response quantities of build-ing are find out like maximum displacement, maximum velocity, maximum acceleration and inter storey drift under the four different earthquake excitations.

Chapter 5

Response of Building Using Viscous Damper

5.1 General

This chapter deals with the response characteristics of building using viscous damper at various storeys. The response of the structure is finding out by numerical method named "Newmark Beta" for both controlled and uncontrolled structures. The steps of the Newmark beta method is discussed in chapter 4.

This type of passive device is considered as the supplemental devices of choice to reduce the structural response. Only damping of the structure is increased by this type of device. The value of the resisting force in viscous fluid devices is linear. The structure is without any brick wall as shown in fig. 4.1 and the structural damping is 5% considered. For parametric study a shear building has been considered as given in Section 4.2, which was converted into a lump mass model. From this lump mass model mass matrix, stiffness matrix and damping matrix is determined, which is given in Appendix-A. For response of controlled structure, a viscous damper is connected as diagonally to the structure as shown in Figure 5.1.

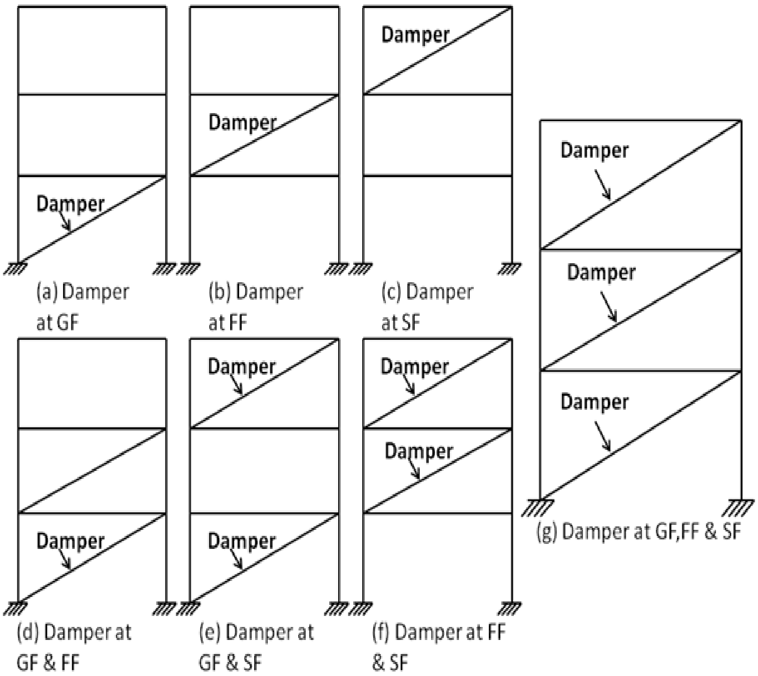

Figure 5.1: Shear Building with passive devices at different location

5.2 Parametric Study

To understand the optimal location of passive devices especially for viscous damper the different response quantities like displacement, velocity, acceleration and drift are calculated with the help of numerical method Newmark Beta as discussed in chapter 4. The response quantities is calculated for the different value of damping coefficient (C_d) which is varied from 10,20,30,40,50,60,70,80,90 and 100 kN*sec/cm. With the help of damping coefficient value, the damping ratio is calculated for passive devices. The equation of motion for all condition of damper location which is GF, FF, SF, GF FF, GF SF, FF SF and GF FF SF are derived in chapter 4. The equation of motion is solved by using the Newmark Beta method for four different types of earthquake excitation through MATLAB.

5.3 Results and Discussion

This section presents the results obtained through direct integration method Newmark Beta of three storey R.C. frame building with velocity dependent energy dissipation device (Viscous damper). The response of R.C frame building in the form of relative displacement, relative velocity, absolute acceleration, and damper force are obtained. Efficiency of these damping systems is investigated for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge. The damper is placed at various location of the structure like GF, FF, SF, GF FF, GF SF, FF SF and GF FF SF. The response of the uncontrolled structure was discussed in Section 4.6. The response of the structure for all the condition of the structure is calculated by the ration of response of controlled structure to controlled structure. There are various ways of assessing seismic response, but computation of top storey response is a reasonable measure of the overall effect of seismic response. The reduction in the top storey velocity, acceleration, and damping force at first storey of the building are also investigated for four types of earthquake excitations.

5.3.1 Comparison of Displacement Response

The ratio of displacement response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge are given in Table 5.1 to 5.4 respectively with varies damping coefficient and also damper location. The graphical representations of comparison of displacement ratio for four types of earthquake excitation for different value of 'C_d' are shown in figure 5.2. From results, it is evident that, the most suitable location for damper GF FF SF. Then, for two damper provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF FF and SF. The optimal location is found out by finding the best response reduction in all four earthquake with varies in value of C_d.

Table 5.1: Displacement response for Elcentro Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.826	0.870	0.957	0.783	0.826	0.870	0.739
20	0.739	0.783	0.913	0.652	0.696	0.783	0.609
30	0.652	0.739	0.913	0.565	0.609	0.696	0.565
40	0.609	0.696	0.870	0.565	0.565	0.652	0.522
50	0.565	0.652	0.870	0.522	0.565	0.609	0.522
60	0.565	0.652	0.826	0.522	0.522	0.565	0.478
70	0.522	0.609	0.865	0.478	0.522	0.522	0.478
80	0.522	0.609	0.826	0.478	0.478	0.522	0.435
90	0.522	0.609	0.826	0.435	0.478	0.522	0.435
100	0.522	0.609	0.826	0.435	0.478	0.522	0.391

Table 5.2: Displacement response for Kobe Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.818	0.879	0.970	0.742	0.803	0.848	0.727
20	0.697	0.788	0.939	0.606	0.667	0.742	0.591
30	0.621	0.727	0.909	0.515	0.591	0.667	0.500
40	0.561	0.667	0.879	0.455	0.530	0.606	0.439
50	0.515	0.621	0.864	0.409	0.470	0.561	0.379
60	0.485	0.591	0.848	0.364	0.439	0.515	0.348
70	0.455	0.561	0.833	0.333	0.409	0.485	0.318
80	0.439	0.530	0.833	0.303	0.379	0.470	0.288
90	0.424	0.515	0.818	0.303	0.364	0.439	0.273
100	0.409	0.500	0.818	0.288	0.348	0.409	0.273

Table 5.3: Displacement response for Lomaprieta Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.855	0.903	0.968	0.806	0.839	0.887	0.790
20	0.774	0.839	0.935	0.694	0.742	0.806	0.661
30	0.710	0.790	0.903	0.597	0.661	0.742	0.565
40	0.645	0.758	0.887	0.516	0.597	0.694	0.484
50	0.597	0.710	0.871	0.452	0.548	0.645	0.419
60	0.565	0.677	0.855	0.403	0.500	0.613	0.371
70	0.532	0.661	0.855	0.371	0.468	0.581	0.339
80	0.500	0.645	0.839	0.323	0.435	0.548	0.290
90	0.468	0.629	0.839	0.306	0.403	0.532	0.258
100	0.452	0.629	0.839	0.274	0.371	0.516	0.242

Table 5.4: Displacement response for Northridge Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.853	0.916	0.979	0.758	0.821	0.884	0.737
20	0.716	0.832	0.958	0.579	0.663	0.779	0.547
30	0.621	0.758	0.947	0.463	0.568	0.684	0.432
40	0.558	0.705	0.926	0.389	0.495	0.632	0.368
50	0.516	0.674	0.926	0.337	0.432	0.589	0.316
60	0.474	0.653	0.916	0.316	0.389	0.558	0.284
70	0.442	0.642	0.905	0.295	0.358	0.537	0.189
80	0.421	0.642	0.905	0.284	0.347	0.526	0.242
90	0.411	0.632	0.905	0.263	0.337	0.505	0.221
100	0.400	0.632	0.905	0.253	0.337	0.495	0.200

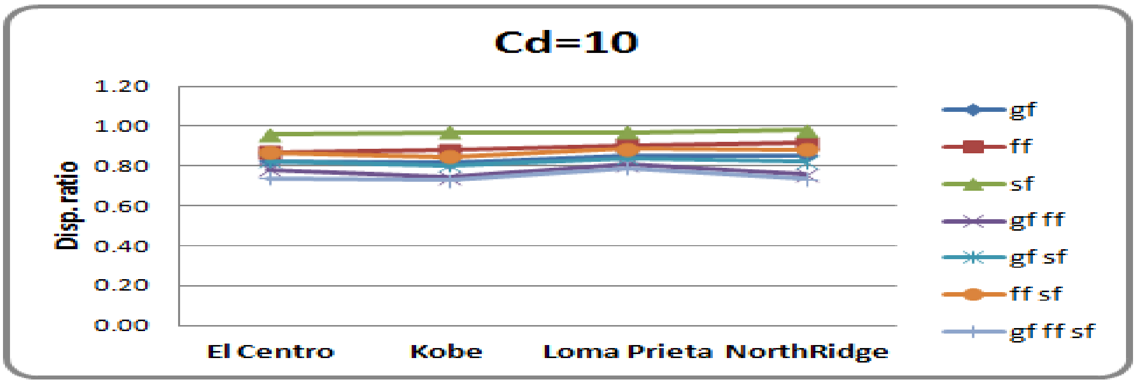

Figure 5.2: Displacement Response at c_d=10

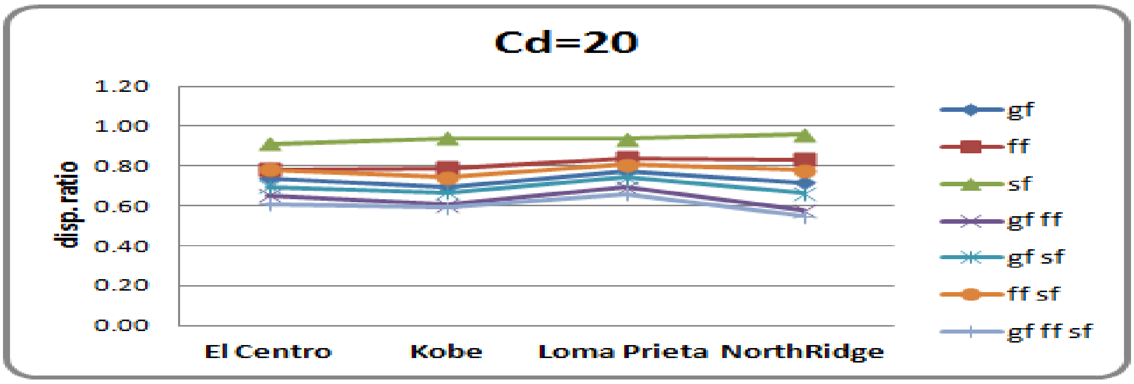

Figure 5.3: Displacement Response at c_d=20

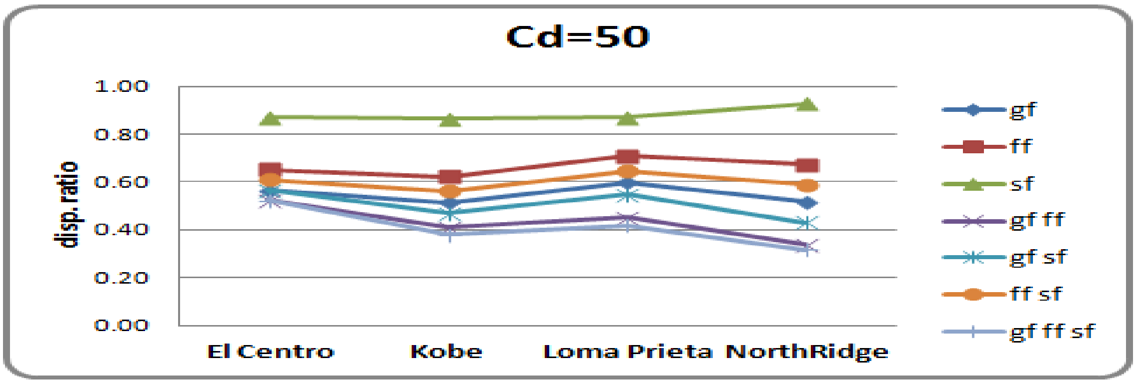

Figure 5.4: Displacement Response at c_d=50

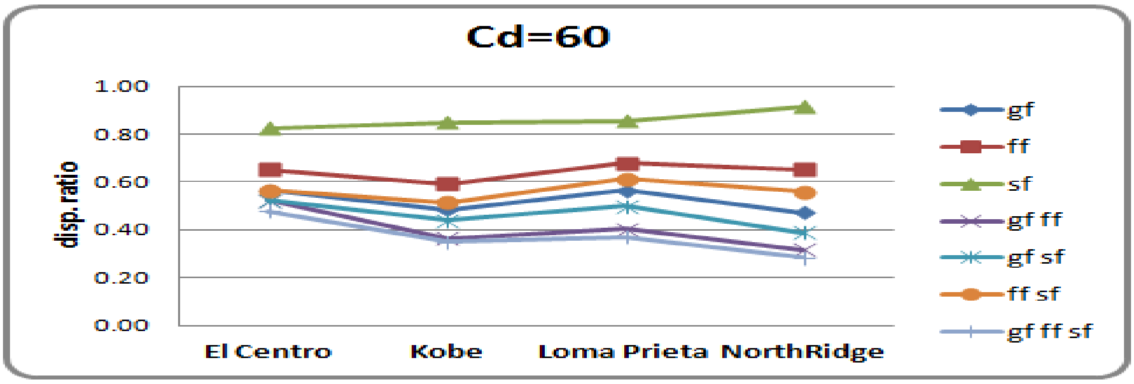

Figure 5.5: Displacement Response at c_d=60

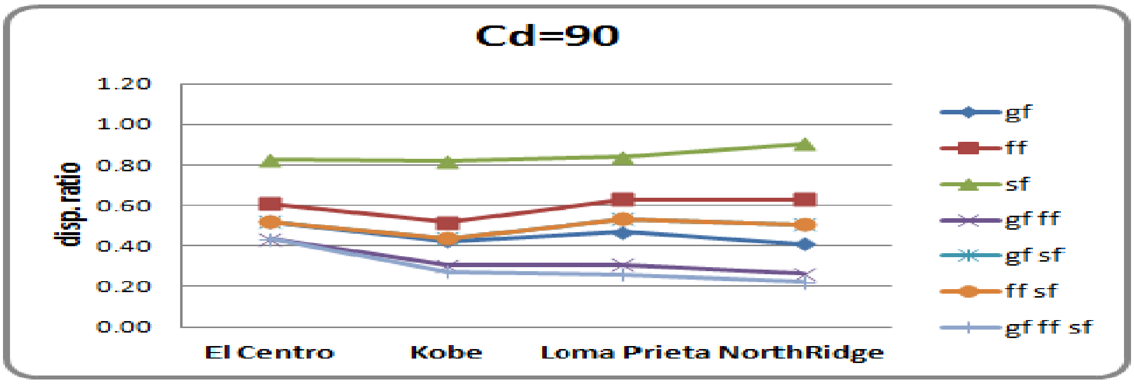

Figure 5.6: Displacement Response at c_d=90

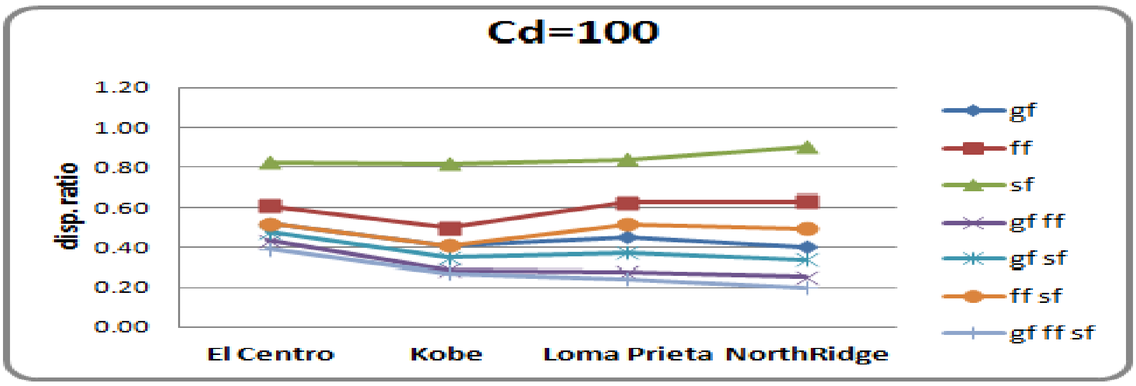

Figure 5.7: Displacement Response at c_d=100

The response of structure for different earthquake excitation with different location of damper is also varies with change in damping coefficient. For C_d is 10kNsec/cm, the suitable earthquake excitation is kobe then Northridge then Elcentro and lomaprieta earthquake. While for C_d is 20 to 100 kNsec/cm the best excitation is Northridge, kobe Lomaprieta and Elcentro earthquake.

5.3.2 Comparison of Velocity Response

The ratio of velocity response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge are given in Table 5.5 to 5.8 respectively with varies damping coefficient and also damper location. The graphical representations of comparison of displacement ratio for four types of earthquake excitation for different value of 'C_d ' are shown in figure 5.2. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure. Then, for two damper provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF, FF and SF. The optimal location is found out by finding the best response reduction in all four earthquake with varies in value of C_d .

Table 5.5: Velocity Response for Elcentro Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.894	0.931	0.992	0.824	0.877	0.920	0.807
20	0.801	0.877	0.981	0.734	0.761	0.848	0.693
30	0.772	0.831	0.968	0.680	0.716	0.784	0.625
40	0.751	0.794	0.957	0.636	0.680	0.732	0.565
50	0.733	0.785	0.947	0.593	0.651	0.709	0.510
60	0.715	0.778	0.938	0.553	0.627	0.692	0.460
70	0.702	0.771	0.932	0.523	0.604	0.676	0.420
80	0.692	0.764	0.926	0.497	0.584	0.663	0.385
90	0.681	0.759	0.922	0.475	0.566	0.651	0.353
100	0.674	0.754	0.919	0.455	0.550	0.641	0.326

Table 5.6: Velocity Response for Kobe Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.777	0.848	0.967	0.695	0.761	0.828	0.847
20	0.641	0.737	0.929	0.555	0.618	0.705	0.663
30	0.553	0.653	0.895	0.472	0.526	0.614	0.530
40	0.520	0.589	0.866	0.463	0.483	0.546	0.435
50	0.527	0.551	0.843	0.445	0.480	0.506	0.392
60	0.531	0.549	0.824	0.422	0.476	0.495	0.356
70	0.534	0.547	0.808	0.398	0.473	0.486	0.326
80	0.537	0.545	0.796	0.379	0.471	0.479	0.299
90	0.539	0.544	0.786	0.361	0.469	0.474	0.276
100	0.541	0.543	0.779	0.342	0.467	0.469	0.256

Table 5.7: Velocity Response for Lomaprieta Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.796	0.869	0.958	0.694	0.765	0.835	0.676
20	0.664	0.772	0.924	0.555	0.629	0.715	0.529
30	0.592	0.702	0.898	0.455	0.542	0.639	0.423
40	0.536	0.665	0.882	0.379	0.477	0.587	0.347
50	0.495	0.649	0.872	0.335	0.426	0.553	0.313
60	0.465	0.641	0.866	0.308	0.388	0.537	0.285
70	0.456	0.641	0.863	0.285	0.358	0.528	0.260
80	0.450	0.644	0.863	0.266	0.335	0.525	0.239
90	0.447	0.650	0.865	0.249	0.325	0.523	0.221
100	0.445	0.655	0.867	0.235	0.317	0.524	0.205

Table 5.8: Velocity Response for Northridge Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.839	0.906	0.970	0.732	0.804	0.871	0.703
20	0.699	0.824	0.942	0.548	0.640	0.758	0.528
30	0.615	0.755	0.923	0.466	0.546	0.669	0.438
40	0.551	0.725	0.910	0.399	0.488	0.619	0.367
50	0.504	0.701	0.901	0.346	0.438	0.580	0.313
60	0.474	0.682	0.896	0.306	0.403	0.547	0.270
70	0.451	0.667	0.893	0.305	0.372	0.526	0.252
80	0.431	0.655	0.892	0.303	0.345	0.516	0.243
90	0.413	0.645	0.892	0.299	0.322	0.508	0.233
100	0.398	0.636	0.893	0.294	0.320	0.501	0.223

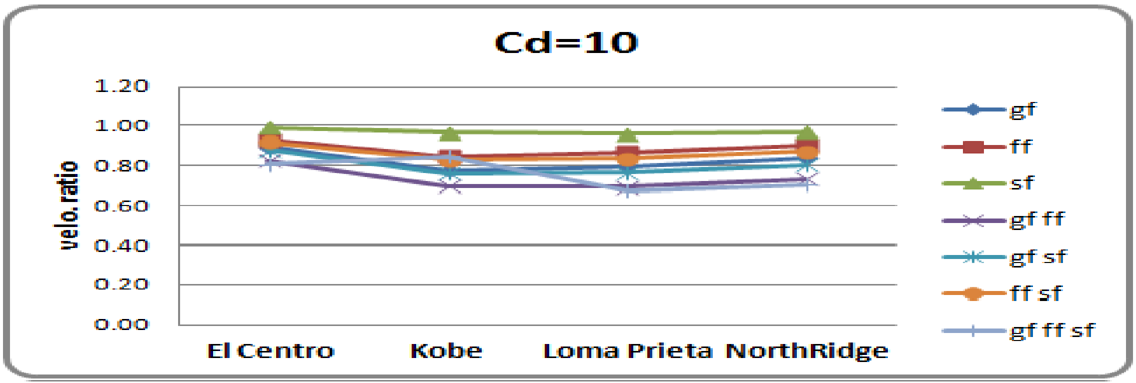

Figure 5.8: Velocity Response atc_d= 10

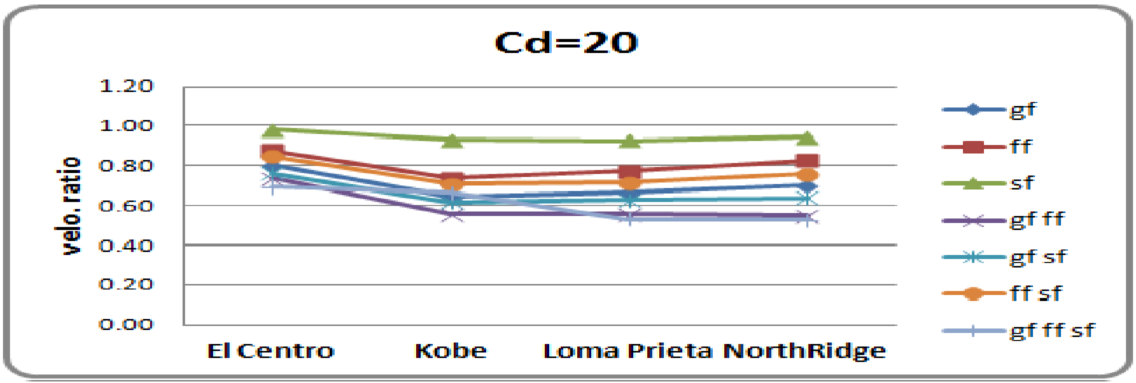

Figure 5.9: Velocity Response at $c_d = 20$

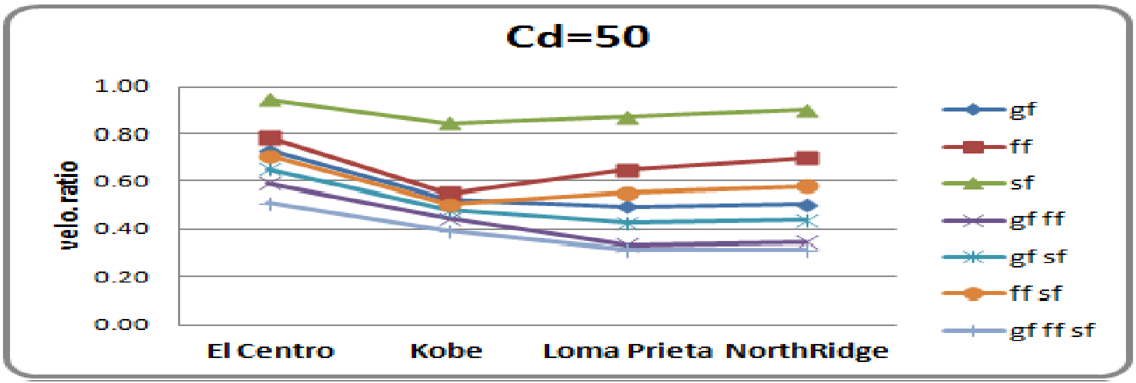

Figure 5.10: Velocity Response at $c_d = 50$

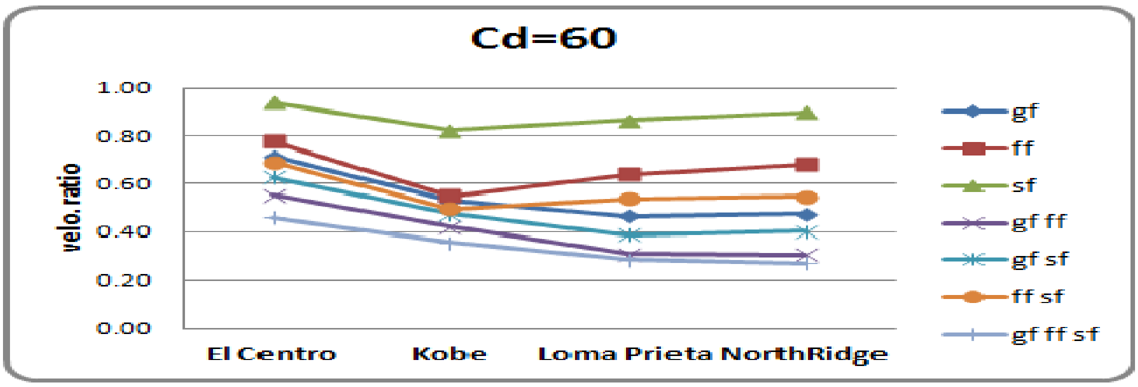

Figure 5.11: Velocity Response at $c_d = 60$

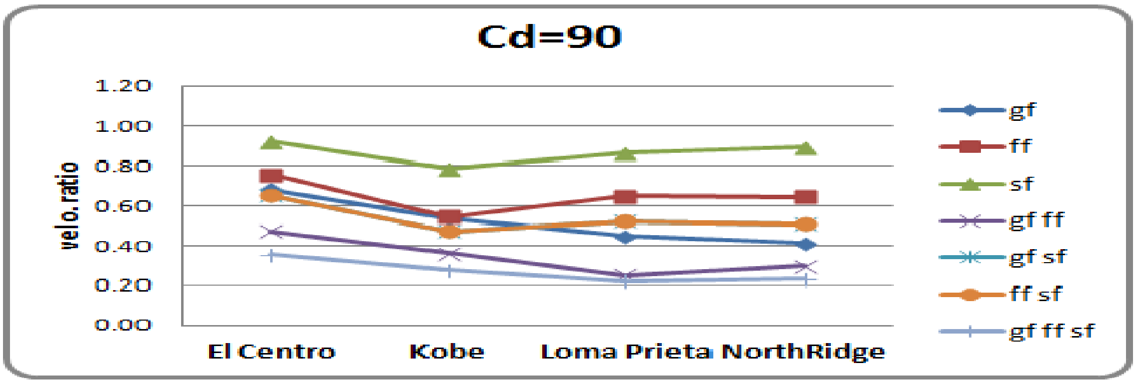

Figure 5.12: Velocity Response atc_d= 90

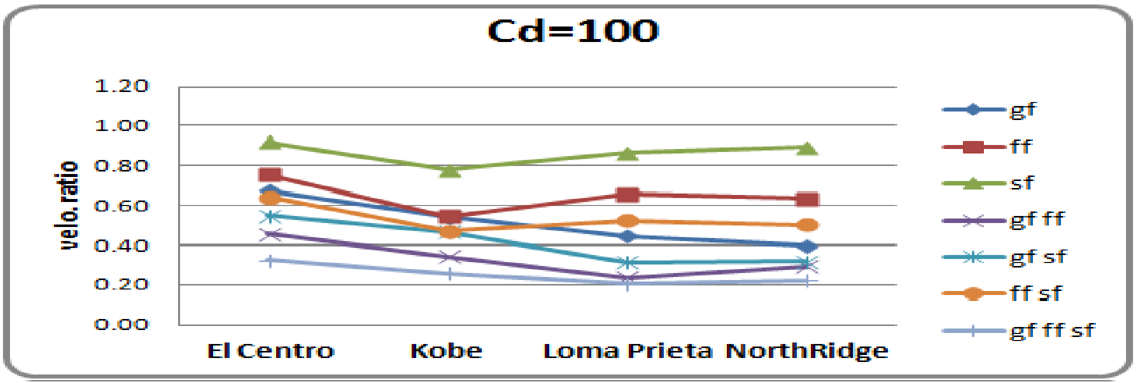

Figure 5.13: Velocity Response atc_d= 100

The velocity ratio for controlled structure to uncontrolled structure at roof level is calculated because the response is maximum for roof level.The response of structure for different earthquake excitation with different location of damper is also varies with change in damping coefficient. For C_d up to the 10 to 60 kNsec/cm the best earthquake excitation is Northridge earthquake. Then it follows by Lomaprieta earthquake, Kobe earthquake and Elcentro earthquake. While for C_d is 70 t0 100 kNsec/cm Lomaprieta earthquake is good.

5.3.3 Comparison of Acceleration Response

The ratio of Acceleration response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and

Northridge are given in Table 5.9 to 5.12 respectively with varies damping coefficient and also damper location. The graphical representations of comparison of displacement ratio for four types of earthquake excitation for different value of 'C_d ' are shown in figure 5.2. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure. Then, for two damper provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF, FF and SF. The optimal location is found out by finding the best response reduction in all four earthquake with varies in value of C_d.

Table 5.9: Acceleration response for Elcentro Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	1.000	1.074	1.136	0.911	0.965	1.026	0.858
20	0.927	0.980	1.117	0.804	0.834	0.898	0.733
30	0.905	0.966	1.093	0.728	0.781	0.857	0.632
40	0.892	0.960	1.072	0.682	0.742	0.831	0.577
50	0.884	0.958	1.056	0.656	0.721	0.811	0.571
60	0.884	0.958	1.045	0.638	0.706	0.802	0.564
70	0.891	0.960	1.036	0.633	0.695	0.796	0.556
80	0.899	0.963	1.030	0.634	0.693	0.792	0.548
90	0.907	0.968	1.026	0.634	0.693	0.790	0.540
100	0.915	0.974	1.023	0.633	0.694	0.789	0.534

Table 5.10: Acceleration response for Kobe Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.821	0.877	0.952	0.742	0.785	0.839	0.732
20	0.716	0.792	0.914	0.623	0.668	0.734	0.630
30	0.652	0.733	0.886	0.555	0.599	0.665	0.558
40	0.611	0.690	0.865	0.512	0.554	0.617	0.505
50	0.583	0.657	0.852	0.486	0.524	0.580	0.466
60	0.564	0.609	0.842	0.466	0.503	0.552	0.437
70	0.562	0.609	0.835	0.454	0.491	0.530	0.414
80	0.573	0.591	0.830	0.446	0.483	0.512	0.396
90	0.584	0.576	0.827	0.440	0.489	0.497	0.382
100	0.596	0.564	0.825	0.435	0.498	0.485	0.371

Table 5.11: Acceleration response for Lomaprieta Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.802	0.864	0.961	0.755	0.785	0.828	0.739
20	0.741	0.792	0.925	0.663	0.709	0.757	0.636
30	0.695	0.759	0.898	0.598	0.653	0.713	0.563
40	0.661	0.734	0.880	0.551	0.612	0.679	0.510
50	0.635	0.715	0.868	0.515	0.580	0.653	0.471
60	0.615	0.723	0.861	0.489	0.557	0.633	0.441
70	0.600	0.746	0.857	0.470	0.539	0.619	0.418
80	0.617	0.766	0.855	0.455	0.525	0.632	0.400
90	0.634	0.789	0.855	0.445	0.514	0.647	0.386
100	0.649	0.809	0.856	0.436	0.505	0.660	0.374

Table 5.12: Acceleration response for Northridge Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.856	0.916	0.978	0.764	0.819	0.881	0.724
20	0.769	0.841	0.961	0.643	0.698	0.774	0.580
30	0.730	0.776	0.950	0.544	0.629	0.720	0.468
40	0.688	0.765	0.942	0.501	0.568	0.696	0.403
50	0.678	0.769	0.938	0.499	0.540	0.675	0.374
60	0.671	0.775	0.936	0.522	0.516	0.659	0.368
70	0.662	0.782	0.935	0.538	0.494	0.648	0.366
80	0.655	0.789	0.935	0.549	0.495	0.375	0.652
90	0.647	0.796	0.937	0.556	0.504	0.656	0.380
100	0.645	0.802	0.938	0.560	0.513	0.661	0.382

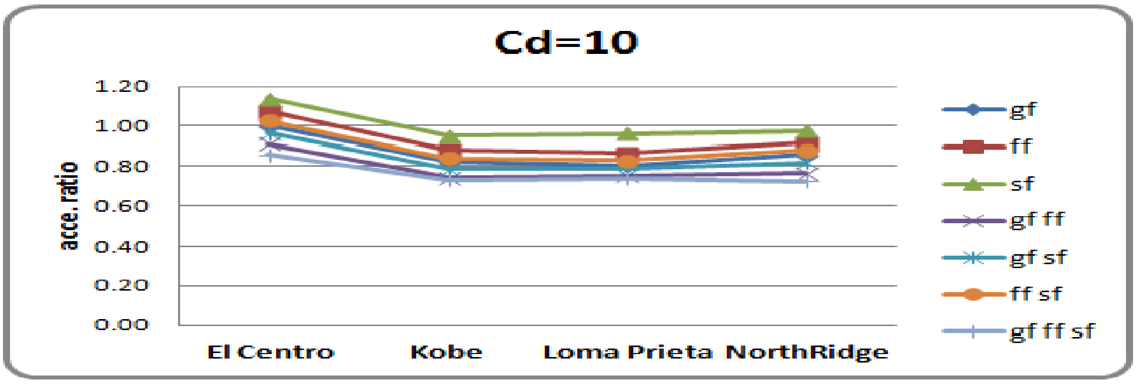

Figure 5.14: Acceleration Response atc_d= 10

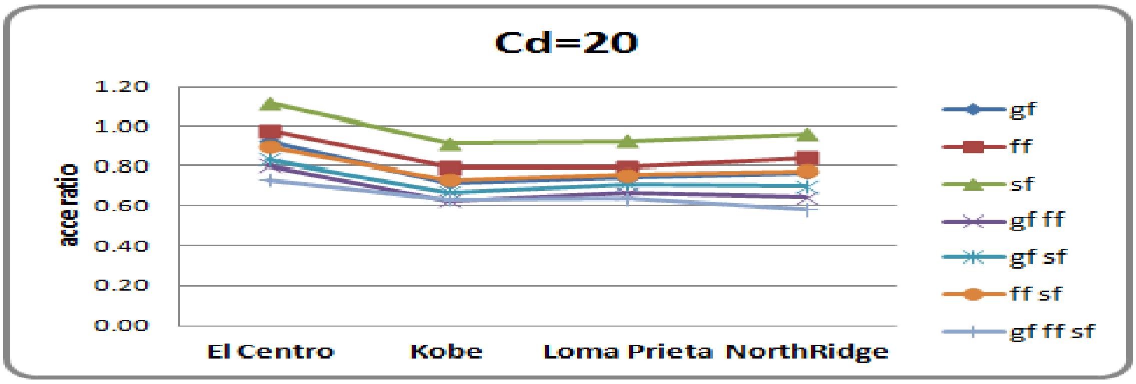

Figure 5.15: Acceleration Response atc_d= 20

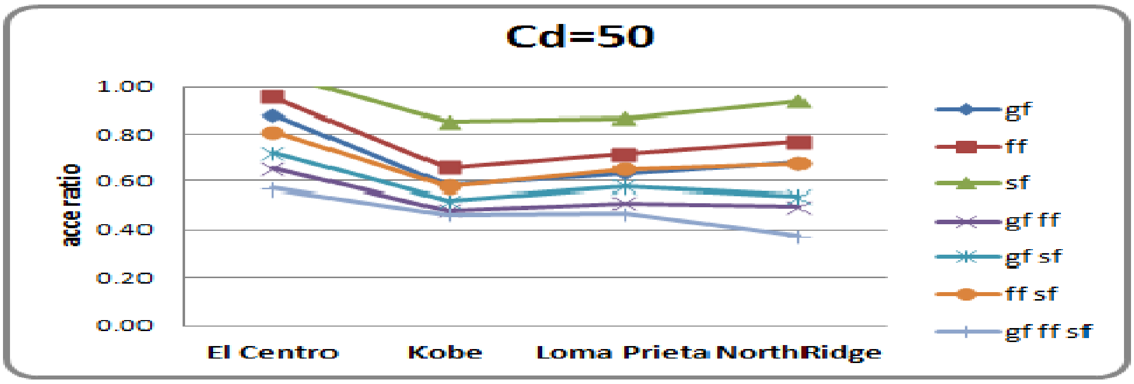

Figure 5.16: Acceleration Response atc_d= 50

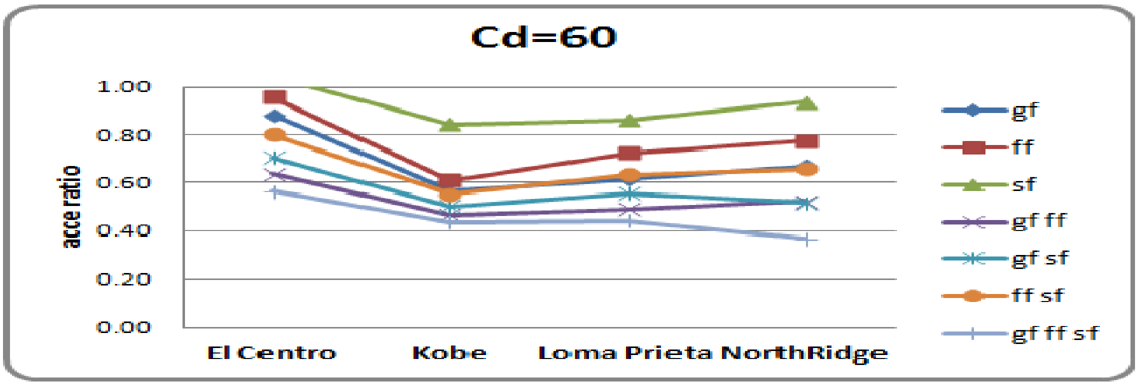

Figure 5.17: Acceleration Response atc_d= 60

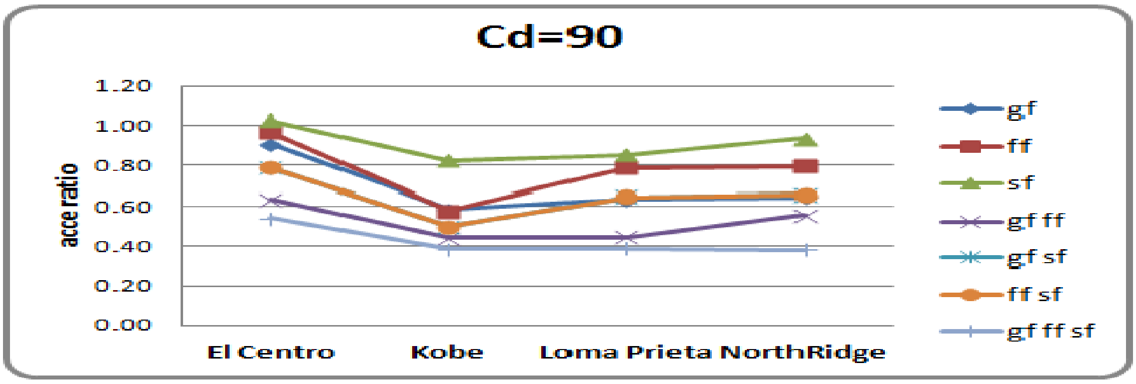

Figure 5.18: Acceleration Response atc_d= 90

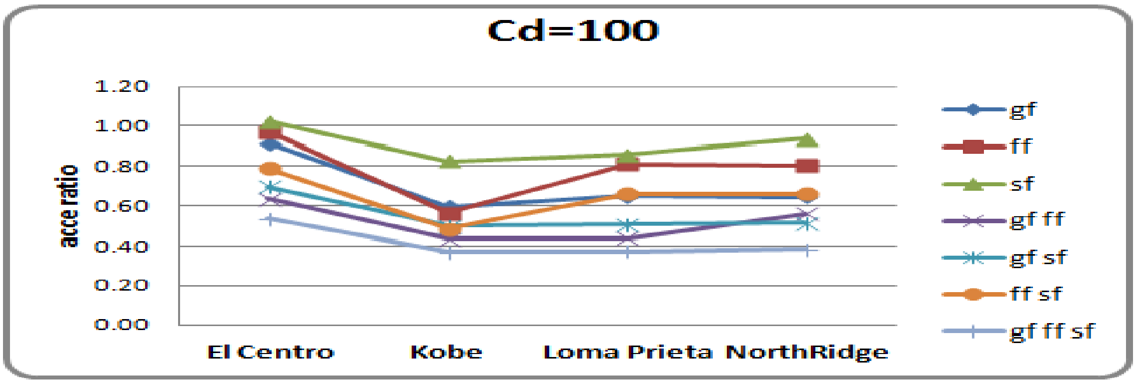

Figure 5.19: Acceleration Response atc_d= 100

The response of structure for different EQ excitation with different location of damper is also varies with change in damping coefficient. For C_d is 10kNsec/cm, the suitable EQ excitation is kobe then Northridge then Elcentro and lomaprieta EQ. While for C_d is 20 to 100 kNsec/cm the best excitation is Northridge, kobe Lomaprieta and Elcentro EQ.

5.3.4 Comparison of Inter-storey Drift response

The ratio of inter storey drift response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge are given in Table 5.13 to 5.16 respectively with varies damping coefficient and also damper lo-cation. The graphical representations of comparison of displacement ratio for four types of earthquake excitation for different value of 'C_d ' are shown in figure 5.2. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure. Then, for two damper provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF FF and SF. The optimal location is found out by finding the best response reduction in all four EQ with varies in value of C_d .

Table 5.13: Drift Response for Elcentro Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.750	0.750	1.000	0.750	0.750	1.000	0.750
20	0.750	0.750	0.750	0.750	0.500	0.750	0.500
30	0.750	0.750	0.750	0.500	0.500	0.500	0.500
40	0.750	0.750	0.500	0.750	0.500	0.500	0.500
50	0.500	0.500	0.750	0.500	0.500	0.500	0.500
60	0.750	0.750	0.500	0.500	0.500	0.500	0.250
70	0.500	0.500	0.500	0.500	0.500	0.500	0.500
80	0.500	0.750	0.250	0.500	0.250	0.250	0.250
90	0.500	0.750	0.250	0.500	0.250	0.250	0.500
100	0.750	1.000	0.250	0.500	0.500	0.500	0.250

Table 5.14: Drift Response for Kobe Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.833	0.833	0.917	0.750	0.750	0.750	0.667
20	0.667	0.750	0.833	0.583	0.583	0.667	0.583
30	0.667	0.750	0.750	0.500	0.583	0.583	0.417
40	0.583	0.667	0.667	0.417	0.500	0.500	0.417
50	0.500	0.667	0.583	0.417	0.333	0.417	0.333
60	0.500	0.667	0.500	0.333	0.333	0.333	0.333
70	0.500	0.583	0.417	0.333	0.333	0.333	0.250
80	0.500	0.583	0.417	0.250	0.250	0.333	0.250
90	0.500	0.583	0.333	0.333	0.250	0.250	0.167
100	0.500	0.583	0.333	0.333	0.250	0.167	0.250

Table 5.15: Drift Response for Lomaprieta Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.750	0.833	0.833	0.750	0.750	0.833	0.667
20	0.667	0.750	0.750	0.667	0.583	0.667	0.583
30	0.667	0.750	0.667	0.583	0.500	0.583	0.500
40	0.583	0.750	0.583	0.500	0.417	0.500	0.417
50	0.583	0.667	0.500	0.417	0.417	0.417	0.333
60	0.583	0.667	0.417	0.417	0.333	0.417	0.250
70	0.583	0.667	0.417	0.417	0.333	0.333	0.333
80	0.583	0.667	0.333	0.333	0.333	0.333	0.250
90	0.500	0.667	0.250	0.333	0.250	0.333	0.167
100	0.500	0.750	0.167	0.333	0.167	0.250	0.167

Table 5.16: Drift Response for Northridge Earthquake

C_d	GF	FF	SF	GFFF	GFSF	FFSF	GFFFSF
10	0.882	0.941	0.941	0.765	0.765	0.824	0.706
20	0.824	0.941	0.824	0.588	0.588	0.706	0.529
30	0.882	0.882	0.765	0.471	0.529	0.588	0.412
40	0.882	0.882	0.647	0.412	0.471	0.588	0.353
50	0.824	0.882	0.588	0.353	0.353	0.471	0.294
60	0.824	0.882	0.529	0.353	0.294	0.353	0.235
70	0.882	1.000	0.412	0.353	0.294	0.294	0.235
80	0.882	1.059	0.353	0.353	0.235	0.294	0.235
90	0.882	1.059	0.353	0.353	0.235	0.235	0.235
100	0.824	1.059	0.294	0.353	0.235	0.235	0.176

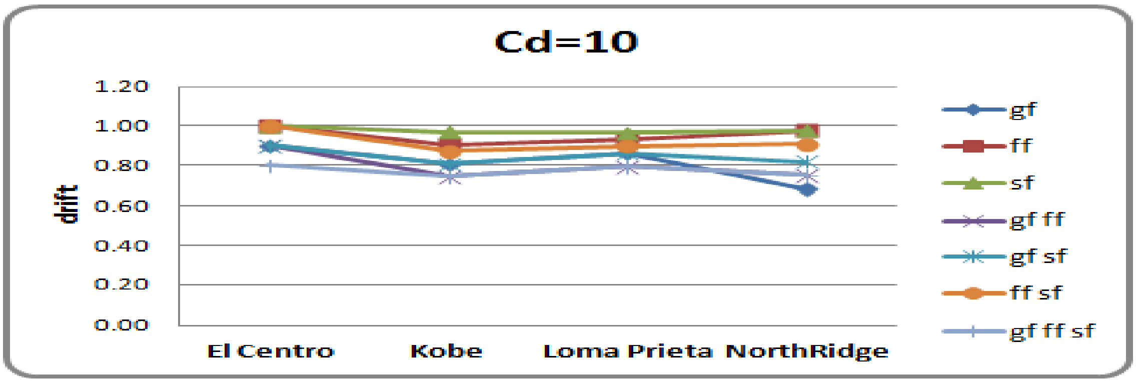

Figure 5.20: Inter-storey Drift Response atc_d= 10

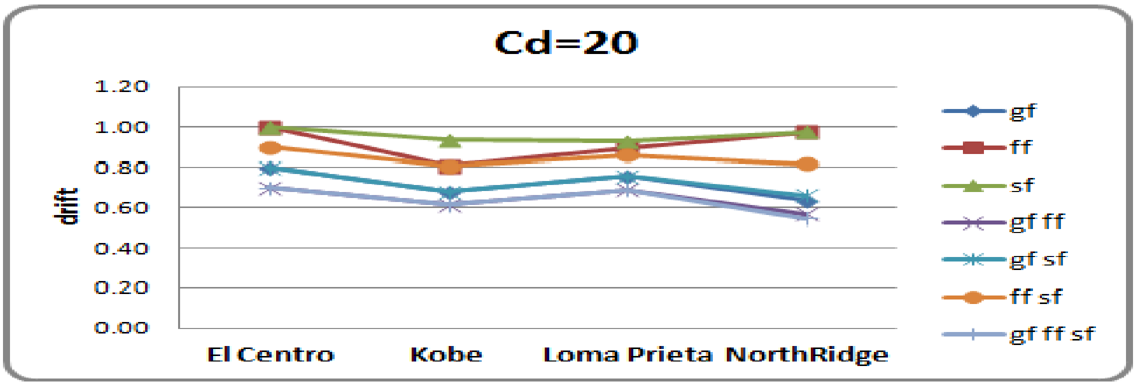

Figure 5.21: Inter-storey Drift Response ac_d= 20

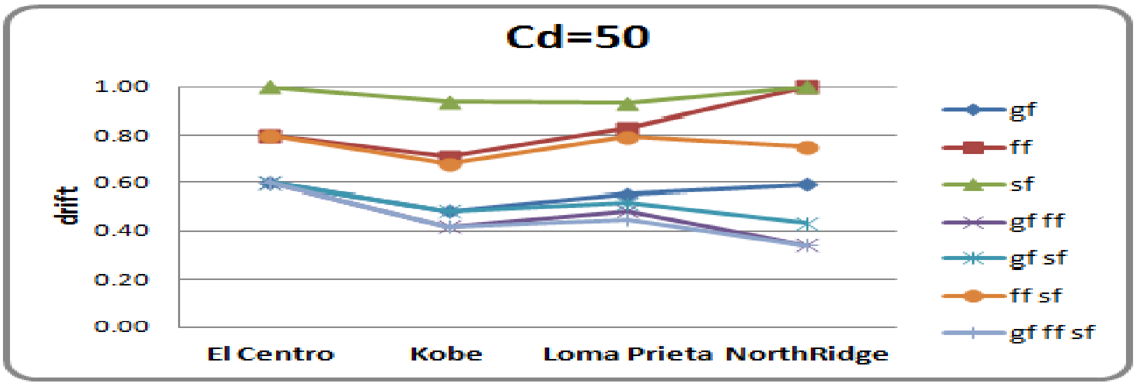

Figure 5.22: Inter-storey Drift Response atc_d= 50

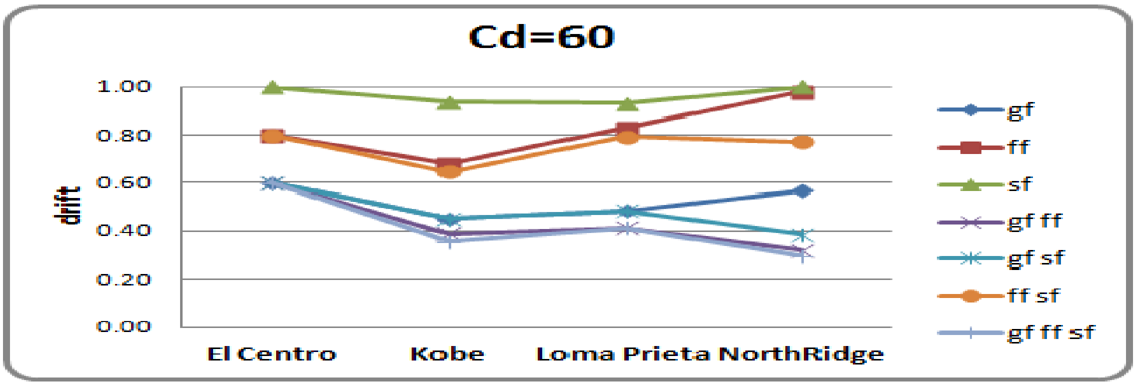

Figure 5.23: Inter-storey Drift Response at c_d= 60

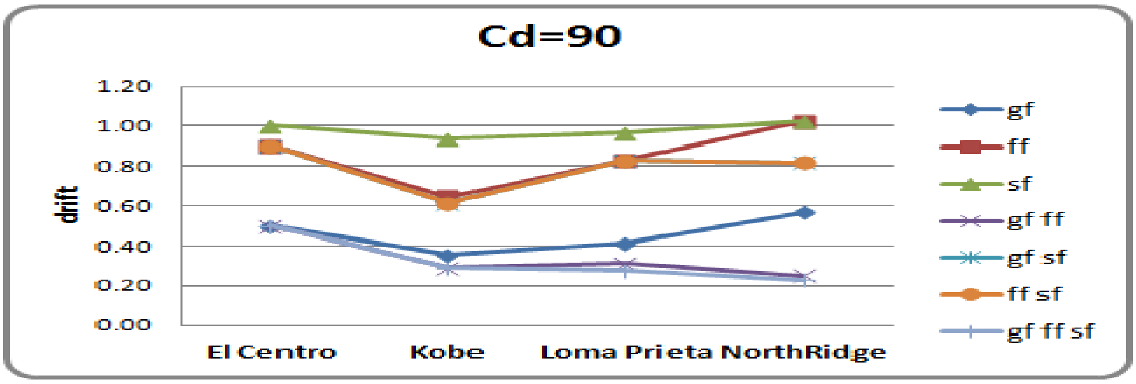

Figure 5.24: Inter-storey Drift Response at c_d= 90

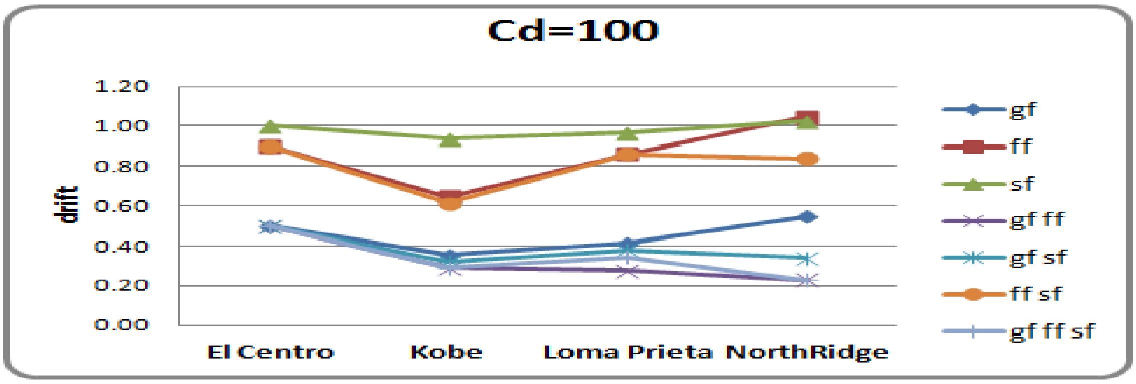

Figure 5.25: Inter-storey Drift Response at c_d= 100

The response of structure for different EQ excitation with different location of damper is also varies with change in damping coefficient. For C_d is 10kNsec/cm, the suitable EQ excitation is kobe then Northridge then Elcentro and lomaprieta . While for C_d is 20 to 100 kNsec/cm the best excitation is Northridge, kobe Lomaprieta and Elcentro EQ.

5.3.5 Comparison of Damper Force

The graphical representation shows the damper force where the damper is attached. From this graphical representation, it concludes that when damper is connected to the second floor maximum force is generate. With varies the C_d value from 10 to 100 kNsec/cm, the force is increased. When the number of damper is increased in structure, the generated force is de-creased compare to force generate by lesser number of damper in structure.. Figure shows the force v/s damping coefficient graph for different condition of damper location with different value of C_d for different earthquake excitation.

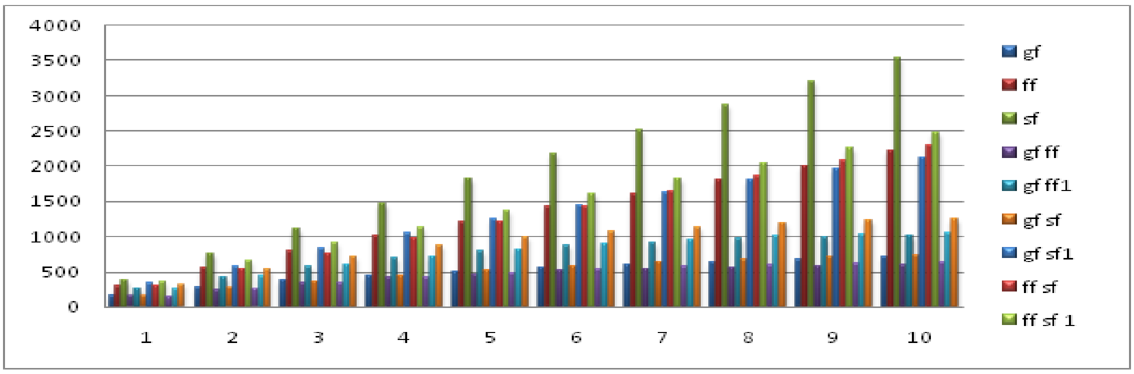

Figure 5.26: Force for Elcentro earthquake

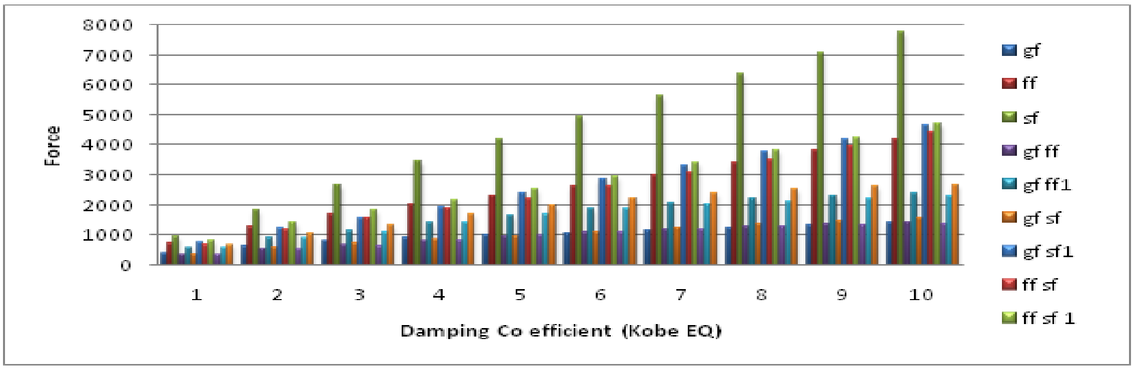

Figure 5.27: Force for Kobe earthquake

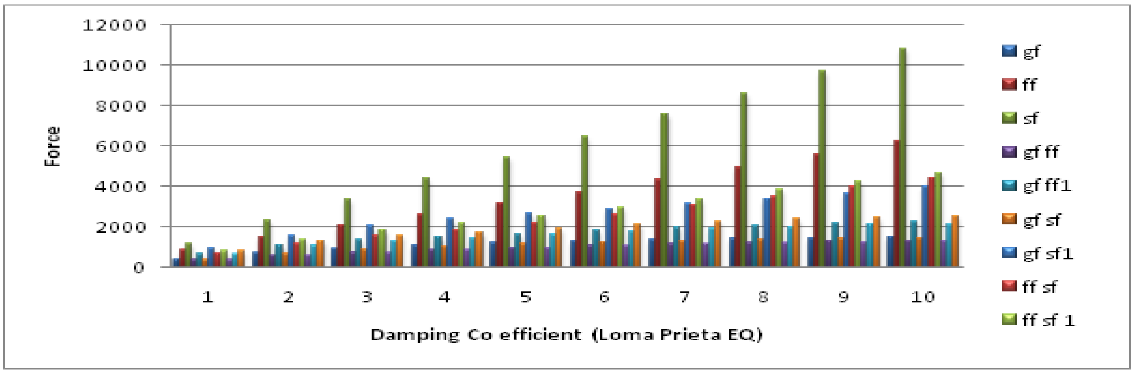

Figure 5.28: Force for Lomaprieta earthquake

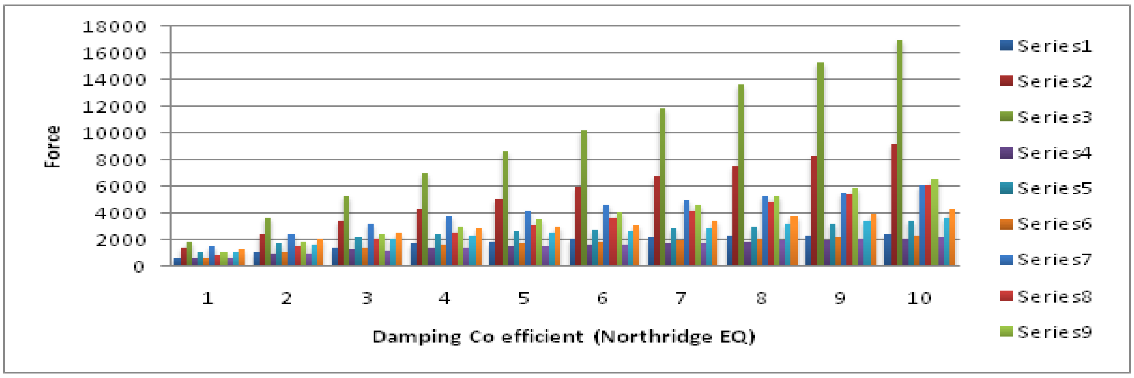

Figure 5.29: Force for Northridge earthquake

5.4 Summary

This chapter deals with the response of structure, which was found out by numerical method Newmark Beta for different value of Damping coefficient with different location of damper with four different earthquake excitation like Elcentro earthquake, Kobe earthquake, Lomaprieta Earthquake and Northridge Earthquake. From all response quantities like displacement, velocity, acceleration and drift conclude that for single damper in structure the best or suitable position for damper location is at GF. It follows by FF and SF respectively. For two damper in structure, the optimal location for damper is GF FF and which follows by GF SF and FF SF. For all different location of damper, the optimal condition is when damper is at GF FF SF.

Chapter 6

Response of Building Using Viscoelastic Damper

6.1 General

This chapter deals with the response characteristics of building using viscoelastic damper at various storeys. The response of the structure is finding out by numerical method named "Newmark Beta" for both controlled and uncontrolled structures. The steps of the Newmark beta method is discussed in chapter 4. By adding viscoelastic damper in structure, the stiffness and damping in the structure is also increased.

6.2 Parametric Study

To understand the optimal location of passive devices especially for viscoelastic damper the different response quantities like displacement, velocity, acceleration and drift are calculated with the help of numerical method Newmark Beta as discussed in chapter 4. The response quantities are calculated for the different value of damping ratio which is varied from 12, 14, 16, 18, 20, 22, 24, 26, 28 and 30. For required damping in structure, viscoelastic damper is designed as describe below and based on the design for particular value of damping ration, the modified stiffness and damping coefficient is calculated for the VE damper. The equation of motion for all condition of damper location which is GF, FF, SF, GF FF, GF SF, FF SF and GF FF SF are derived in chapter 4. The equation of motion is solved by using the Newmark Beta method for four different types of earthquake excitation through MATLAB.

6.2.1 Viscoelastic Damper Design

Following design procedure illustrate the parameters like number, size and required properties of damper for any structure to achieve target structural response.

- The required damping in general can be determined from the response spectra of the design earthquake. Prior to design it is required to decide, desired damping ratio that should be achieved to reduce prescribed response level of building. In this study, the required structural damping ratio ζ is assumed for the initial goal.

- The selection of VE damper stiffness K_d and loss factor is a trial and error procedure. This is determine from the modal strain energy method as,

$$\alpha_d K_d = \frac{2\zeta}{\eta - 2\zeta} K_s \tag{6.1}$$

  Where, ζ is the target added damping ratio; α_d is the attachment coefficient; η is the loss factor; and K_s is the storey stiffness of structure without damper.

- The thickness of VE material 't' can be determined based on the maximum allowable damper deformation to ensure that the maximum strain in the viscoelastic material is smaller than the maximum allowable value. Thickness of VE material can be determine as,

$$t = \frac{0.004 \times hs \times \cos\theta}{\gamma \times d} \tag{6.2}$$

  Where, 't' is the thickness of one layer of VE material in the damper; 'hs' is the typical storey height; ‘d’ is the maximum design damper strain; γ is the angel of inclination of VE device. In this study maximum design damper strain ‘d’ of 60

- The area of damper is determined from the following equation. Thus, damper size can be decided by assuming damper width and from the required length of damper.

$$A = \frac{k_d \times t'}{G} \tag{6.3}$$

Where, K_d is the damper stiffness; $G^{"}$ is the damper storage modulus; t is the thick-ness of one layer of VE material. The damping co-efficient $'C_d$' of viscoelastic damper can be determine from following equation,

$$C_d = \frac{G^{00} \times A}{\omega \times t} \tag{6.4}$$

Where, $G^{"}$ is the damper loss modulus; ω is the natural frequency of the structure.

- Properties of damper like Shear Modulus, Loss Factor can be decided as per the temperature for which damper is to be design. Maximum allowable strain in VE material will also change as per design temperature, to avoid the nonlinear behavior of VE material. In this study design temperature is assumed as 25^o C.

- The RC building can be analyzed now with added VE damper. We can find damping ratio achieved from the following equation.

$$\zeta = \frac{\eta}{2}(1 - \frac{{\omega_d}^2}{{\omega_{dn}}^2}) \tag{6.5}$$

Where, ω_d and ω_{dn} is the natural frequency and damped natural frequency of the system; η is the loss factor.

For parametric study, different value of required damping ratio ζ 12%, 14%, 16%, 18%, 20%, 22%, 24%, 26%, 28%, 30% are considered. VE damper is design for different value of ζ as per above discussion, and find out the VE damper parameter like damper stiffness K_d, co-efficient of damper C_d, and size of damper, which is given in Table 6.2.1 Sample calculation of VE damper is given in Appendix-B. A three storey building with VE damper at first storey level is analyzed using Newmark-Beta Method for four different types of earthquake excitations as per Table6.2.1 through MATLAB, and obtain the response quantity like relative displacement, relative velocity, absolute acceleration and damper force. Subsequent section deals with the results and discussion of VE damper equipped building with uncontrolled building.

Table 6.1: Viscoelastic Dampers Design Parameter

ζ	C^d	K^d	Damping Coefficient		
%	Nsec/m	N/m	L(m)	B(m)	t(m)
12	1732345	25855250	0.4	0.25	0.016
14	2381974	35550969	0.55	0.25	0.016
16	2815061	42014781	0.65	0.25	0.016
18	3637925	54296025	0.7	0.3	0.016
20	4417480	65930888	0.85	0.3	0.016
22	4937183	73687463	0.95	0.3	0.016
24	5976590	89200613	1.15	0.3	0.016
26	6756146	100835475	1.3	0.3	0.016
28	7795553	116348625	1.5	0.3	0.016
30	9094812	135740063	1.5	0.35	0.016

6.3 Results and Discussion

This section presents the results obtained through Newmark-Beta direct integration method of three storey R.C. shear building with Viscoelastic damper for different value of required damping ratio. The response of R.C frame building in the form of relative displacement, relative velocity, absolute acceleration, and damper force are obtained. Efficiency of these damp-ing systems is investigated for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge. The damper is placed at various location of the structure like GF, FF, SF, GF FF, GF SF, FF SF and GF FF SF. The response of the uncontrolled structure was discussed in Section 4.6. The response of the structure for all the condition of the structure is calculated by the ratio of response of controlled structure to controlled structure. There are various ways of assessing seismic response, but computation of top storey response is a reasonable measure of the overall effect of seismic response. The reduction in the top storey velocity, acceleration, and damping force at first storey of the building are also investigated for four types of earthquake excitations.

6.3.1 Comparison of Displacement Response

The ratio of displacement response of controlled building for four earthquake excitation of different peak acceleration value,namely El Centro, Kobe, Loma Prieta and Northridge are given in Table6.3.1 to 6.3.1 respectively with varies damping ratio and also damper location. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure.Then, for two damper provide in structure, the best location is GF FF, GF SF and FF SF respectively.Similarly, for one damper in structure, the best location is GF, FF and SF. The optimal location is found out by finding the best response reduction in all four earthquake with varies in value of ζ .

Table 6.2: Displacement response for Elcentro Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.727	0.783	0.913	0.609	0.652	0.739	0.565
14	0.682	0.783	0.913	0.522	0.609	0.696	0.478
16	0.682	0.739	0.913	0.478	0.565	0.696	0.435
18	0.636	0.739	0.87	0.435	0.565	0.652	0.391
20	0.636	0.696	0.87	0.391	0.522	0.609	0.348
22	0.636	0.696	0.87	0.391	0.522	0.609	0.348
24	0.591	0.696	0.87	0.348	0.478	0.609	0.304
26	0.591	0.696	0.87	0.348	0.435	0.565	0.261
28	0.591	0.652	0.87	0.304	0.435	0.565	0.261
30	0.545	0.652	0.87	0.304	0.435	0.565	0.304

Table 6.3: Displacement response for Kobe Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.652	0.788	0.939	0.515	0.606	0.712	0.485
14	0.591	0.727	0.909	0.455	0.53	0.652	0.409
16	0.545	0.697	0.909	0.424	0.5	0.606	0.379
18	0.5	0.652	0.894	0.379	0.439	0.561	0.333
20	0.47	0.621	0.879	0.348	0.409	0.515	0.303
22	0.439	0.591	0.879	0.318	0.394	0.5	0.273
24	0.409	0.561	0.864	0.303	0.364	0.455	0.242
26	0.394	0.545	0.864	0.288	0.394	0.439	0.227
28	0.379	0.515	0.848	0.258	0.333	0.409	0.197
30	0.379	0.5	0.848	0.242	0.333	0.394	0.182

Table 6.4: Displacement response for Lomaprieta Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.742	0.823	0.935	0.613	0.694	0.774	0.581
14	0.694	0.806	0.935	0.548	0.629	0.742	0.5
16	0.677	0.806	0.919	0.5	0.597	0.726	0.452
18	0.645	0.79	0.903	0.435	0.548	0.71	0.387
20	0.629	0.79	0.903	0.403	0.516	0.694	0.339
22	0.629	0.79	0.903	0.371	0.5	0.677	0.323
24	0.613	0.79	0.903	0.339	0.468	0.677	0.274
26	0.597	0.79	0.903	0.306	0.597	0.661	0.258
28	0.597	0.79	0.903	0.29	0.435	0.661	0.226
30	0.581	0.79	0.903	0.258	0.419	0.661	0.194

Table 6.5: Displacement response for Northridge Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.779	0.895	0.968	0.611	0.716	0.832	0.568
14	0.716	0.853	0.968	0.526	0.653	0.779	0.463
16	0.695	0.832	0.958	0.474	0.611	0.758	0.411
18	0.663	0.811	0.958	0.411	0.558	0.716	0.337
20	0.621	0.8	0.947	0.358	0.516	0.684	0.284
22	0.611	0.789	0.947	0.326	0.495	0.674	0.263
24	0.589	0.789	0.947	0.305	0.453	0.663	0.232
26	0.568	0.789	0.947	0.295	0.568	0.642	0.221
28	0.558	0.789	0.947	0.274	0.4	0.632	0.2
30	0.526	0.789	0.937	0.263	0.368	0.611	0.179

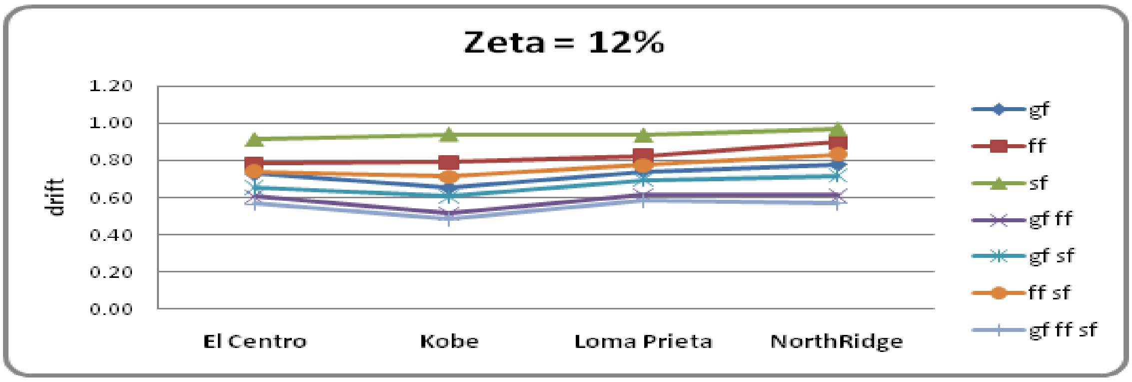

Figure 6.1: Displacement Response for ζ = 12%

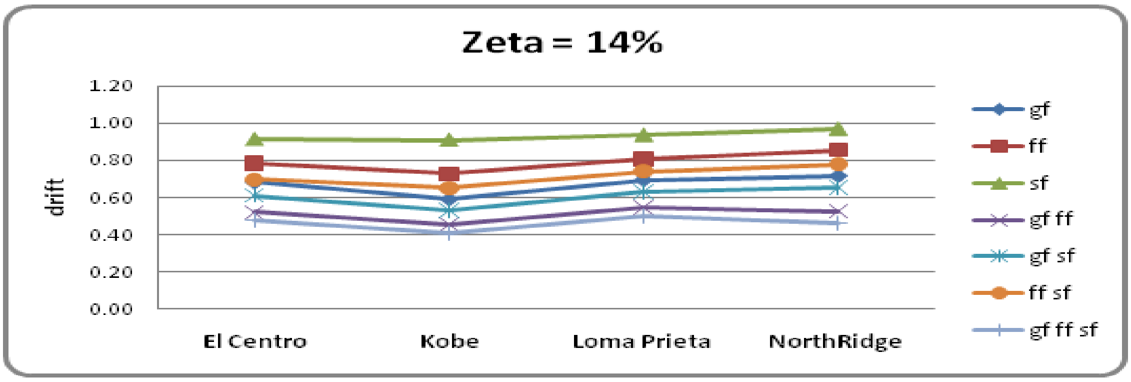

Figure 6.2: Displacement Response for ζ = 14%

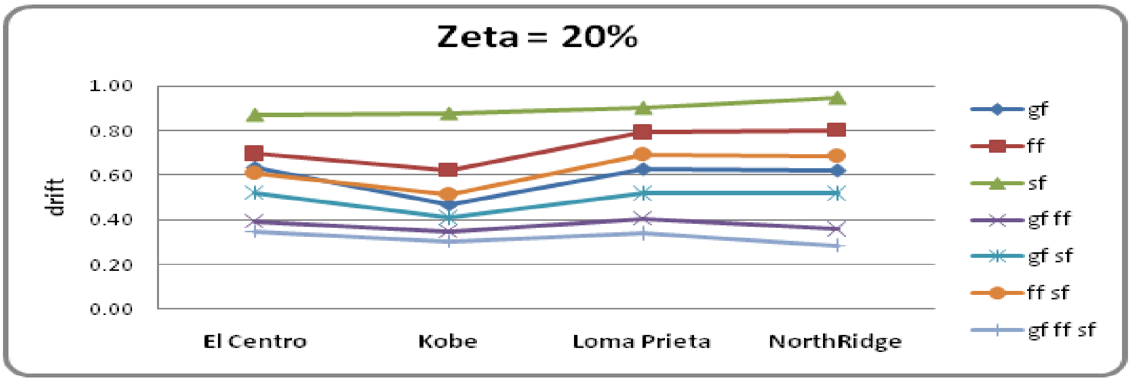

Figure 6.3: Displacement Response forζ = 20%

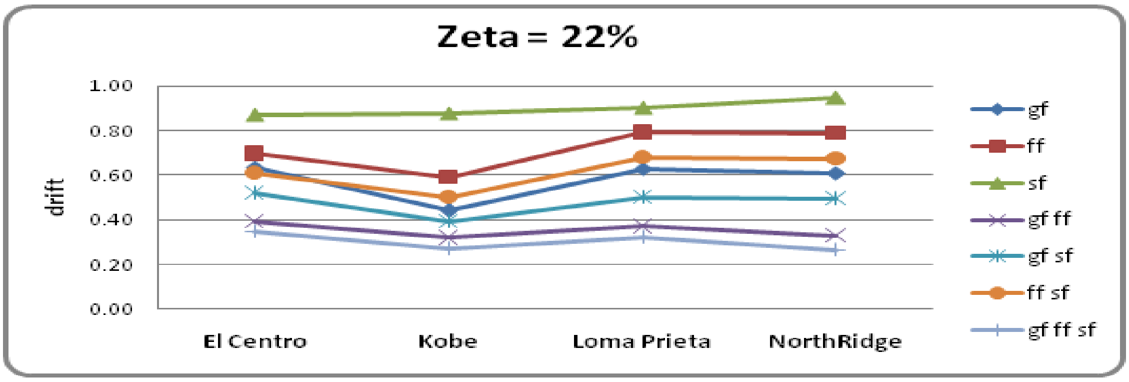

Figure 6.4: Displacement Response forζ = 22%

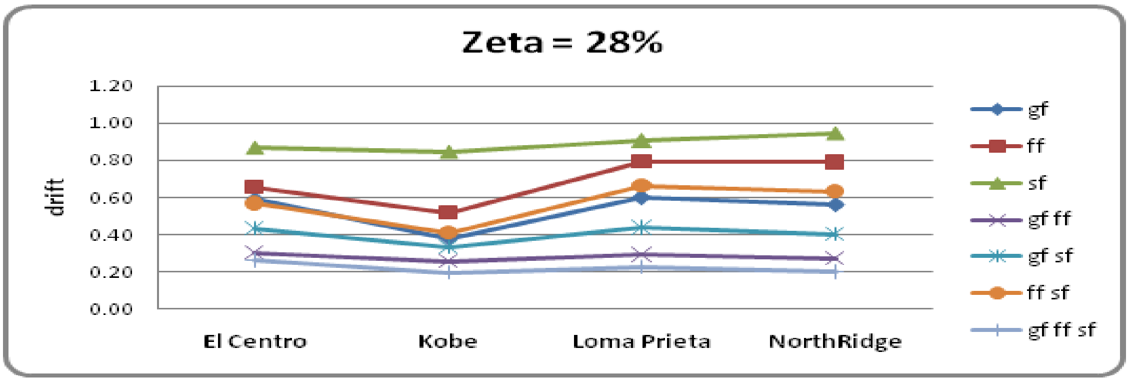

Figure 6.5: Displacement Response forζ = 28%

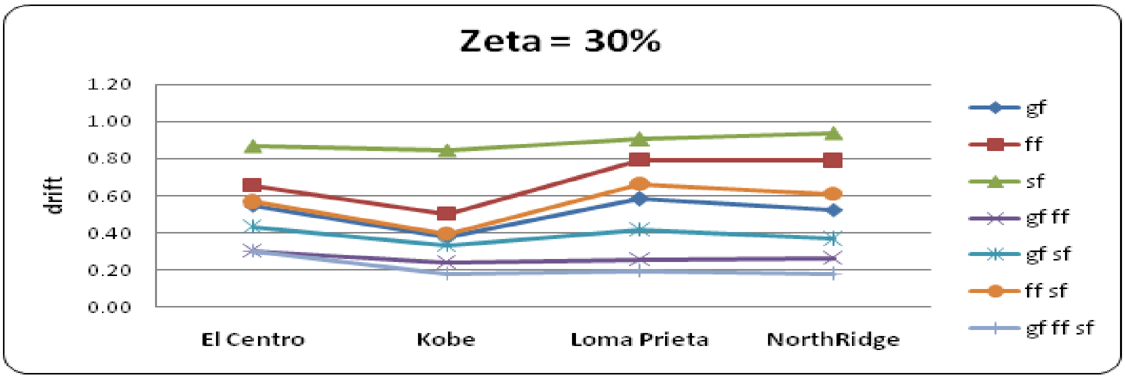

Figure 6.6: Displacement Response forζ = 30%

The response of structure for different EQ excitation with different location of damper is also varies with change in damping ratio. With increased the damping ration, the structural response is also increased so, the ration of response quantities is decreased. The best earthquake excitation for damping value from 12 to 30% Kobe earthquake is best.

6.3.2 Comparison of Velocity Response

The ratio of velocity response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge are given in Table **??** to **??** respectively with varies damping ratio and also damper location. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure. Then, for two dampers provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF, FF and SF. The optimal location is found out by finding the best response reduction in all four earthquake with varies in value of ζ.

Table 6.6: Velocity response for Elcentro Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.839	0.907	0.982	0.72	0.79	0.868	0.671
14	0.798	0.881	0.974	0.687	0.728	0.837	0.619
16	0.772	0.868	0.969	0.663	0.697	0.819	0.588
18	0.749	0.852	0.964	0.624	0.674	0.788	0.539
20	0.741	0.839	0.959	0.596	0.653	0.764	0.497
22	0.741	0.834	0.956	0.58	0.642	0.751	0.474
24	0.754	0.839	0.951	0.549	0.622	0.736	0.427
26	0.762	0.85	0.948	0.528	0.614	0.725	0.402
28	0.769	0.86	0.946	0.505	0.598	0.723	0.373
30	0.777	0.873	0.94	0.479	0.596	0.731	0.339

Table 6.7: Velocity response for Kobe Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.6	0.713	0.927	0.518	0.568	0.629	0.488
14	0.584	0.649	0.907	0.481	0.54	0.566	0.441
16	0.573	0.629	0.896	0.469	0.525	0.588	0.426
18	0.564	0.625	0.879	0.46	0.508	0.572	0.403
20	0.559	0.62	0.865	0.448	0.495	0.559	0.377
22	0.557	0.616	0.858	0.439	0.488	0.552	0.363
24	0.556	0.609	0.845	0.427	0.484	0.539	0.334
26	0.557	0.604	0.837	0.415	0.557	0.532	0.312
28	0.561	0.6	0.829	0.396	0.484	0.526	0.285
30	0.569	0.597	0.822	0.379	0.485	0.521	0.258

Table 6.8: Velocity response for Lomaprieta Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.78	0.868	0.949	0.614	0.716	0.811	0.559
14	0.739	0.847	0.94	0.533	0.657	0.776	0.469
16	0.716	0.837	0.935	0.491	0.627	0.759	0.422
18	0.683	0.824	0.928	0.425	0.579	0.731	0.352
20	0.662	0.816	0.924	0.377	0.544	0.715	0.302
22	0.649	0.814	0.922	0.352	0.524	0.704	0.275
24	0.629	0.81	0.92	0.309	0.492	0.691	0.233
26	0.619	0.808	0.92	0.285	0.619	0.682	0.209
28	0.606	0.808	0.92	0.257	0.453	0.675	0.183
30	0.596	0.809	0.92	0.231	0.432	0.666	0.158

Table 6.9: Velocity response for Northridge Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.83	0.907	0.961	0.641	0.759	0.849	0.572
14	0.773	0.898	0.952	0.53	0.68	0.82	0.463
16	0.735	0.892	0.948	0.482	0.63	0.8	0.418
18	0.667	0.881	0.943	0.409	0.561	0.764	0.347
20	0.628	0.869	0.94	0.36	0.512	0.731	0.297
22	0.605	0.861	0.938	0.336	0.483	0.711	0.273
24	0.56	0.846	0.936	0.328	0.436	0.685	0.26
26	0.53	0.835	0.935	0.327	0.53	0.674	0.249
28	0.505	0.82	0.935	0.323	0.377	0.66	0.235
30	0.491	0.818	0.935	0.316	0.359	0.642	0.216

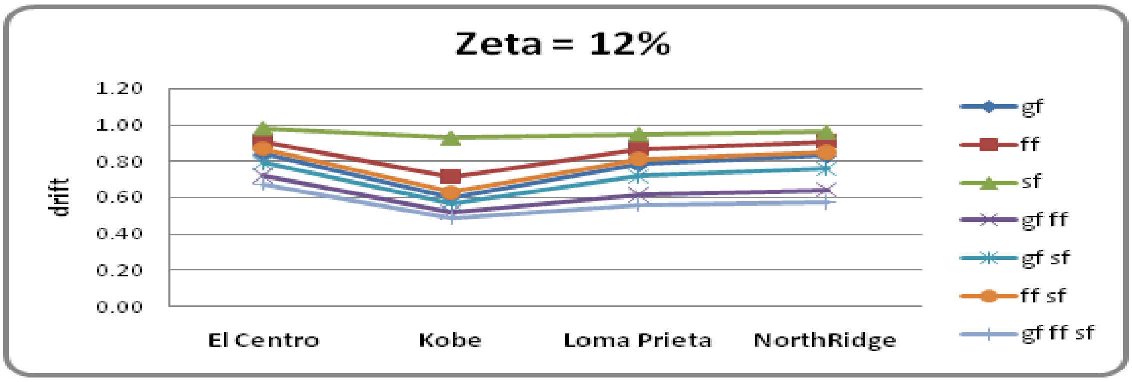

Figure 6.7: Velocity Response forζ = 12%

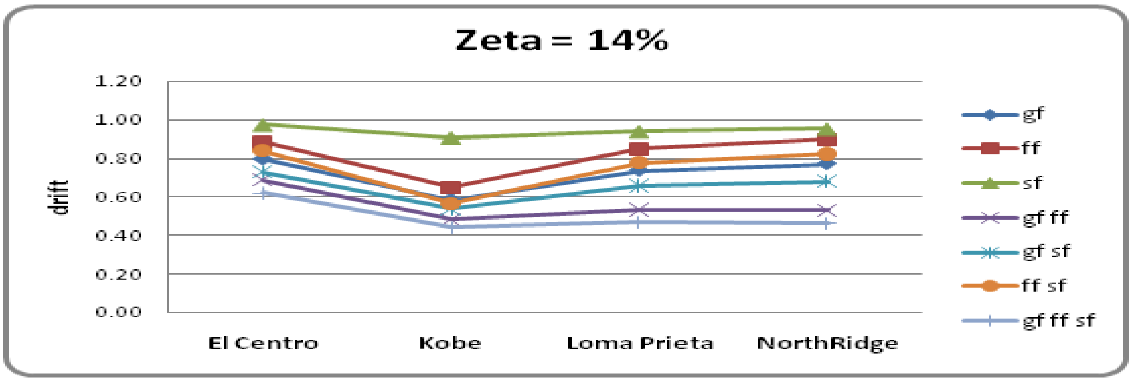

Figure 6.8: Velocity Response for $\zeta = 14\%$

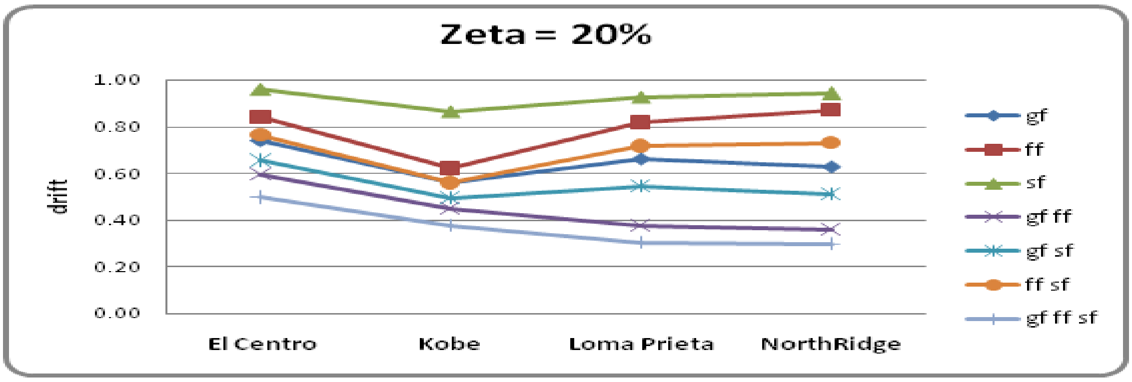

Figure 6.9: Velocity Response for $\zeta = 20\%$

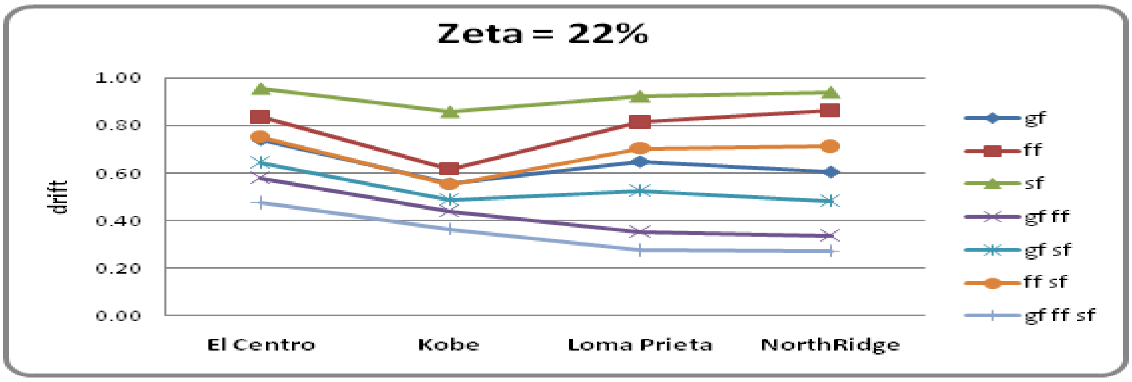

Figure 6.10: Velocity Response for $\zeta = 22\%$

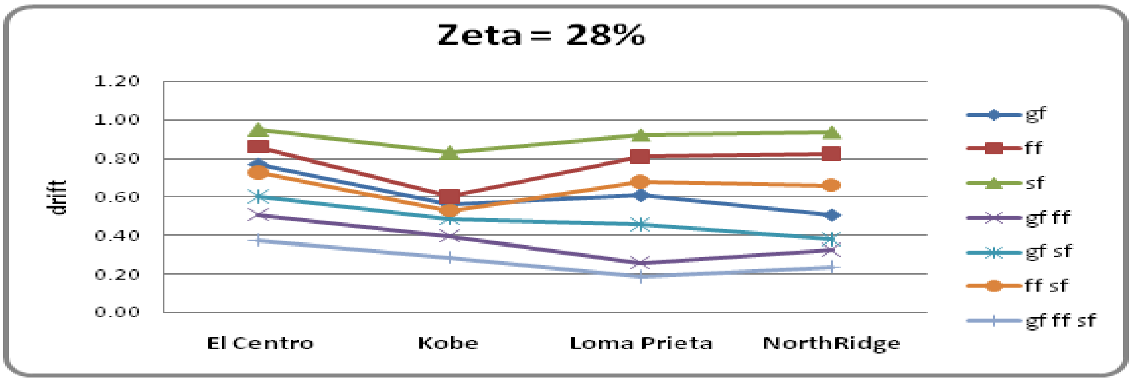

Figure 6.11: Velocity Response forζ = 28%

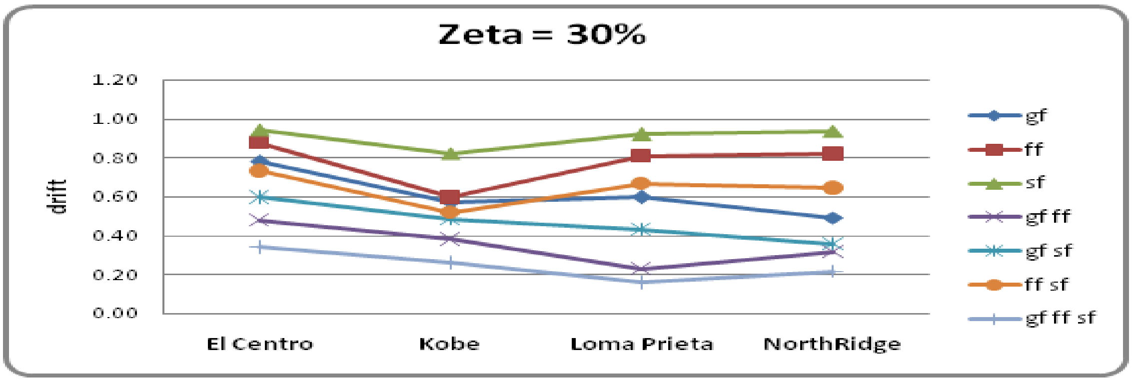

Figure 6.12: Velocity Response forζ = 30%

The response of structure for different EQ excitation with different location of damper is also varies with change in damping ratio. With increased the damping ration, the structural response is also increased so, the ration of response quantities is decreased. The best earthquake excitation for damping value from 12% to 30 % northridge earthquake is best.

6.3.3 Comparison of Acceleration Response

The ratio of Acceleration response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge are given in Table 6.3.3 to 6.3.3 respectively with varies damping coefficient and also damper lo-cation. The graphical representations of comparison of acceleration ratio for four types of earthquake excitation for different value of ζ is shown in figure. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure. Then, for two dampers provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF, FF and SF.

Table 6.10: Acceleration response for Elcentro Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.771	0.805	0.849	0.712	0.693	0.737	0.628
14	0.772	0.796	0.841	0.69	0.673	0.699	0.586
16	0.772	0.791	0.835	0.683	0.665	0.689	0.563
18	0.772	0.791	0.827	0.666	0.657	0.827	0.524
20	0.772	0.797	0.821	0.663	0.652	0.678	0.495
22	0.773	0.801	0.818	0.658	0.649	0.607	0.478
24	0.781	0.807	0.813	0.652	0.646	0.678	0.449
26	0.788	0.812	0.81	0.654	0.626	0.678	0.431
28	0.796	0.818	0.808	0.653	0.65	0.683	0.413
30	0.806	0.824	0.809	0.649	0.652	0.683	0.407

Table 6.11: Acceleration response for Kobe Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.714	0.813	0.925	0.607	0.659	0.745	0.566
14	0.659	0.768	0.912	0.564	0.603	0.692	0.52
16	0.631	0.742	0.905	0.547	0.575	0.664	0.5
18	0.592	0.705	0.895	0.533	0.54	0.62	0.475
20	0.573	0.676	0.888	0.538	0.518	0.587	0.46
22	0.578	0.66	0.884	0.544	0.519	0.569	0.451
24	0.59	0.632	0.878	0.545	0.53	0.54	0.437
26	0.599	0.614	0.875	0.552	0.599	0.523	0.429
28	0.611	0.595	0.871	0.554	0.554	0.518	0.418
30	0.625	0.58	0.867	0.552	0.574	0.519	0.407

Table 6.12: Acceleration response for Lomaprieta Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.836	0.894	0.946	0.713	0.778	0.844	0.649
14	0.828	0.897	0.936	0.666	0.752	0.832	0.59
16	0.826	0.903	0.93	0.641	0.739	0.828	0.564
18	0.829	0.916	0.923	0.602	0.719	0.826	0.527
20	0.834	0.928	0.919	0.573	0.708	0.827	0.5
22	0.838	0.938	0.916	0.557	0.702	0.83	0.486
24	0.847	0.955	0.913	0.53	0.693	0.836	0.462
26	0.854	0.969	0.911	0.514	0.854	0.84	0.448
28	0.863	0.985	0.912	0.502	0.683	0.848	0.433
30	0.873	1.003	0.915	0.493	0.678	0.873	0.418

Table 6.13: Acceleration response for Northridge Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.929	0.974	0.986	0.764	0.831	0.904	0.698
14	0.916	0.967	0.983	0.715	0.78	0.872	0.628
16	0.9	0.972	0.982	0.684	0.764	0.864	0.58
18	0.872	0.996	0.98	0.622	0.741	0.858	0.522
20	0.875	1.013	0.98	0.618	0.714	0.849	0.482
22	0.873	1.021	0.979	0.616	0.695	0.841	0.471
24	0.864	1.033	0.979	0.644	0.659	0.823	0.465
26	0.855	1.039	0.98	0.663	0.855	0.821	0.467
28	0.841	1.045	0.98	0.68	0.636	0.825	0.481
30	0.824	1.049	0.981	0.692	0.618	0.828	0.489

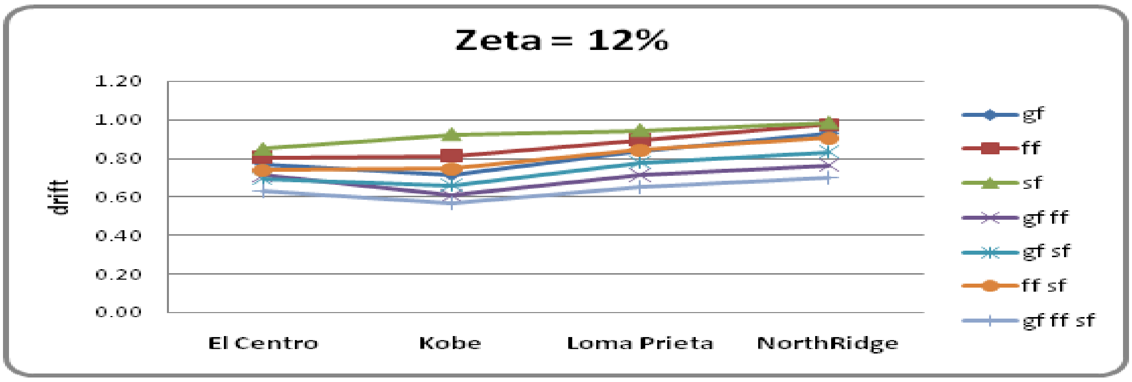

Figure 6.13: Acceleration Response forζ = 12%

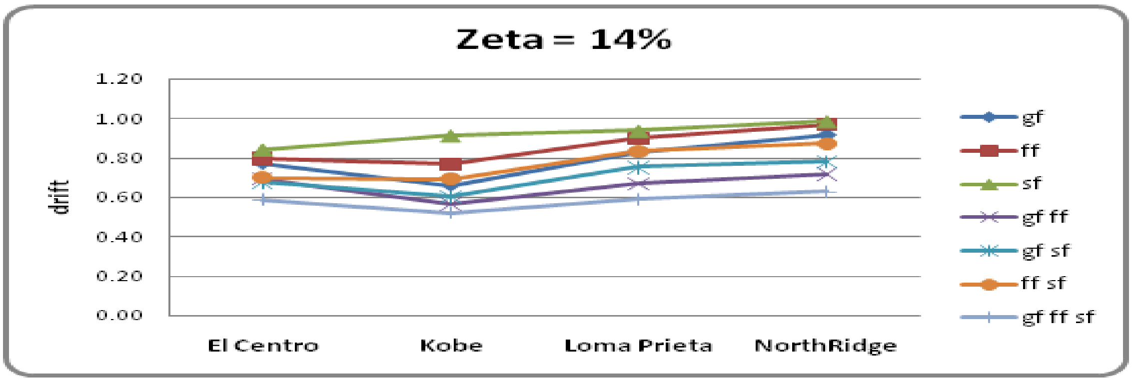

Figure 6.14: Acceleration Response forζ = 14%

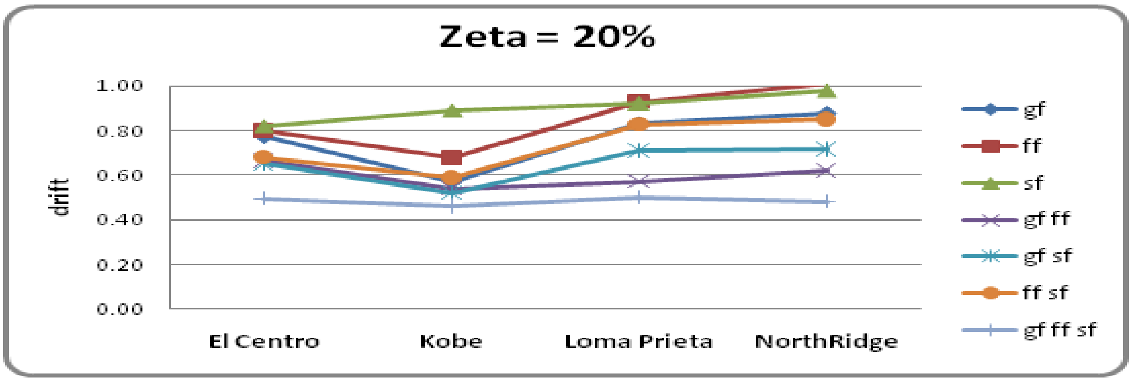

Figure 6.15: Acceleration Response forζ = 20%

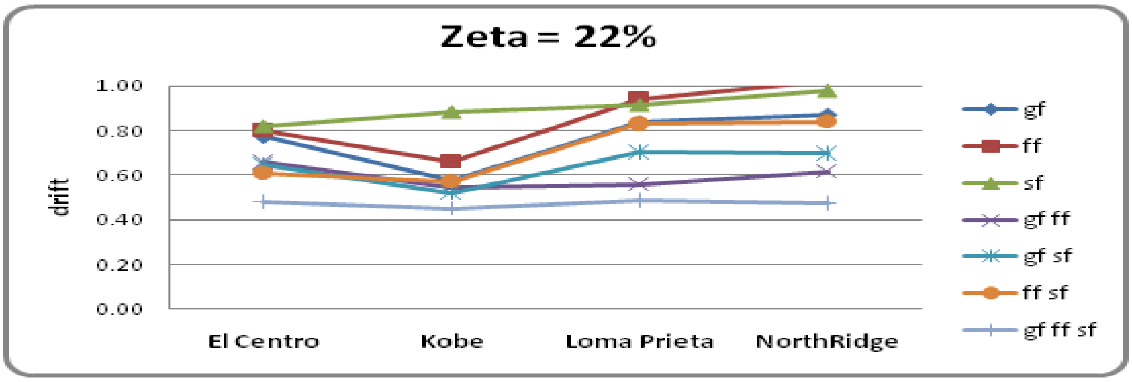

Figure 6.16: Acceleration Response forζ = 22%

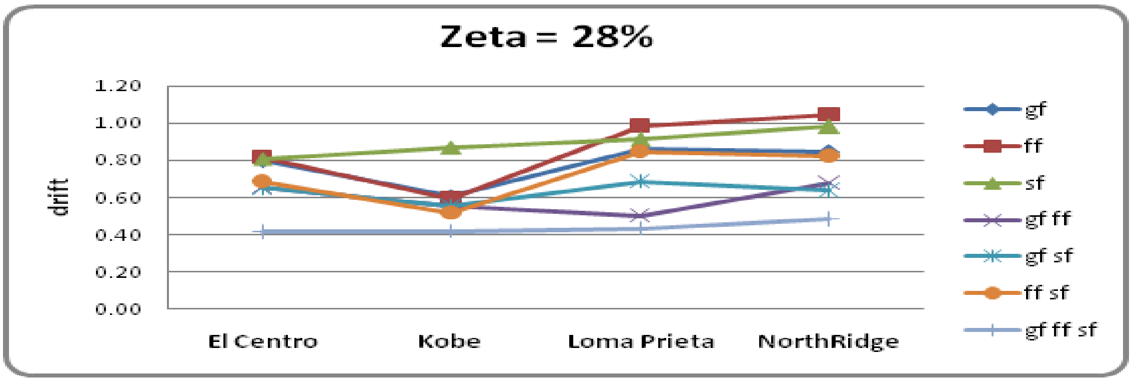

Figure 6.17: Acceleration Response forζ = 28%

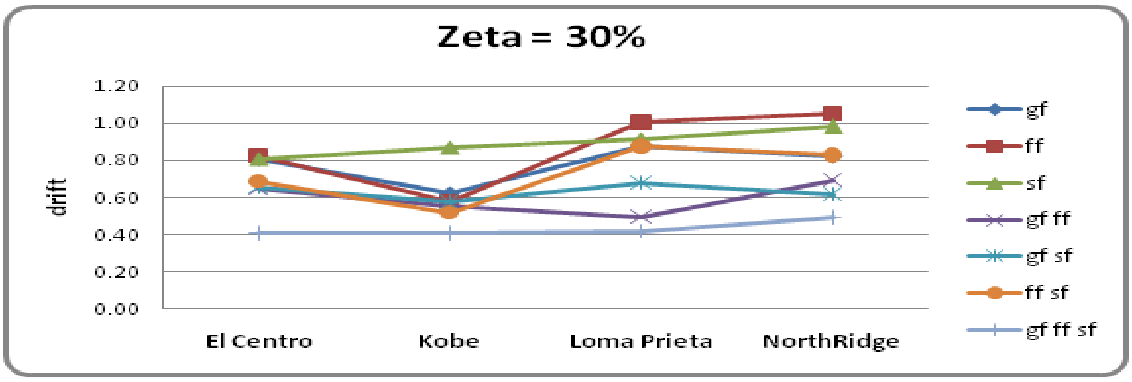

Figure 6.18: Acceleration Response forζ = 30%

The response of structure for different EQ excitation with different location of damper is also varies with change in damping ratio. With increased the damping ration, the structural response is also increased so, the ration of response quantities is decreased. The best earthquake excitation for damping value from 12% to 30 % Kobe earthquake is best.

6.3.4 Comparison of Inter-storey Drift Response

The ratio of inter storey drift response of controlled building for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Loma Prieta and Northridge are given in Table 6.3.4 to 6.3.4 respectively with varies damping ratio and also damper location. From results, it is evident that, when damper is provided at GF FF SF it is most suitable location for structure. Then, for two dampers provide in structure, the best location is GF FF, GF SF and FF SF respectively. Similarly, for one damper in structure, the best location is GF FF and SF.

Table 6.14: Drift response for Elcentro Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.700	1.000	1.000	0.600	0.700	1.000	0.600
14	0.600	1.000	1.000	0.500	0.600	0.900	0.500
16	0.600	1.000	1.000	0.500	0.600	0.900	0.500
18	0.600	1.000	1.000	0.400	0.600	0.900	0.400
20	0.600	1.000	1.000	0.400	0.600	0.900	0.400
22	0.600	1.000	1.000	0.400	0.600	0.900	0.400
24	0.600	1.000	1.000	0.400	0.600	0.900	0.400
26	0.600	1.000	1.000	0.300	0.600	0.900	0.300
28	0.600	1.000	1.000	0.300	0.600	1.000	0.300
30	0.600	1.000	1.000	0.300	0.600	1.000	0.300

Table 6.15: Drift response for Kobe Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.581	0.871	0.968	0.516	0.548	0.839	0.484
14	0.516	0.839	0.968	0.452	0.484	0.774	0.419
16	0.484	0.806	0.968	0.387	0.452	0.774	0.387
18	0.452	0.774	0.968	0.355	0.419	0.742	0.355
20	0.419	0.742	0.968	0.323	0.419	0.71	0.323
22	0.419	0.742	0.968	0.29	0.419	0.677	0.29
24	0.387	0.71	0.968	0.258	0.387	0.677	0.258
26	0.387	0.71	0.968	0.258	0.387	0.645	0.226
28	0.387	0.677	0.968	0.226	0.387	0.613	0.226
30	0.387	0.677	0.968	0.194	0.419	0.613	0.194

Table 6.16: Drift response for Lomaprieta Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.655	0.897	0.966	0.586	0.655	0.897	0.586
14	0.586	0.897	0.966	0.517	0.586	0.897	0.517
16	0.586	0.897	0.966	0.483	0.552	0.897	0.483
18	0.586	0.931	1	0.414	0.552	0.931	0.414
20	0.586	0.931	1	0.379	0.552	0.966	0.379
22	0.621	0.966	1	0.345	0.552	0.966	0.345
24	0.621	1	1	0.31	0.552	1	0.31
26	0.621	1	1.034	0.276	0.621	1.034	0.276
28	0.621	1.034	1.034	0.241	0.552	1.034	0.241
30	0.621	1.069	1.034	0.207	0.552	1.069	0.207

Table 6.17: Drift response for Northridge Earthquake

ζ	gf	ff	sf	gfff	gfsf	ffsf	gfffsf
12	0.682	0.977	0.977	0.568	0.659	0.977	0.568
14	0.636	0.977	0.977	0.477	0.614	0.955	0.477
16	0.614	1	1	0.432	0.614	0.955	0.409
18	0.591	1	1	0.364	0.568	0.932	0.341
20	0.591	1	1	0.318	0.545	0.955	0.295
22	0.568	0.977	0.977	0.295	0.545	0.955	0.273
24	0.568	1	1	0.25	0.523	0.977	0.25
26	0.568	1.023	1.023	0.227	0.568	0.977	0.227
28	0.568	1.023	1.023	0.227	0.5	1	0.205
30	0.545	1.045	1.045	0.227	0.477	1.023	0.182

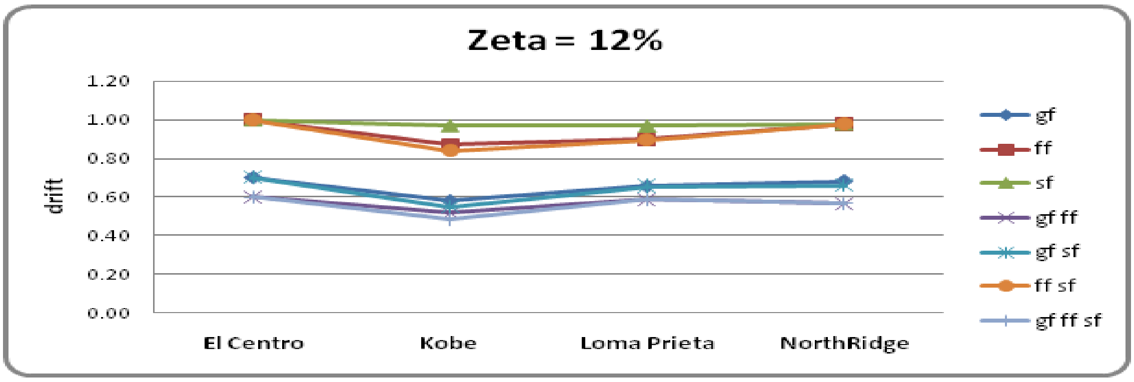

Figure 6.19: Inter-storey Drift Response for$\zeta = 12\%$

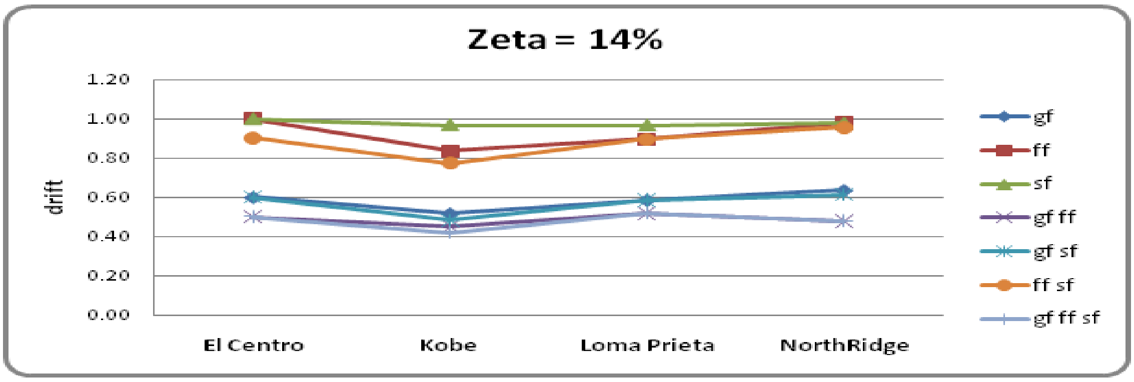

Figure 6.20: Inter-storey Drift Response for$\zeta = 14\%$

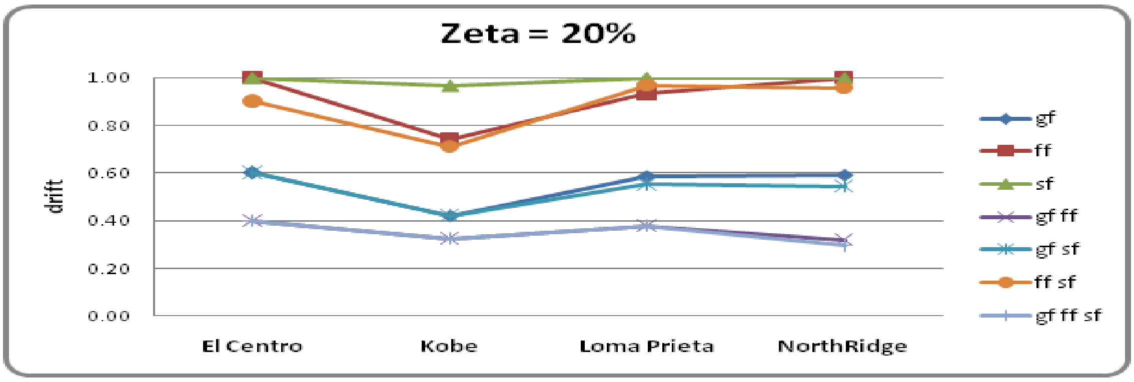

Figure 6.21: Inter-storey Drift Response for$\zeta = 20\%$

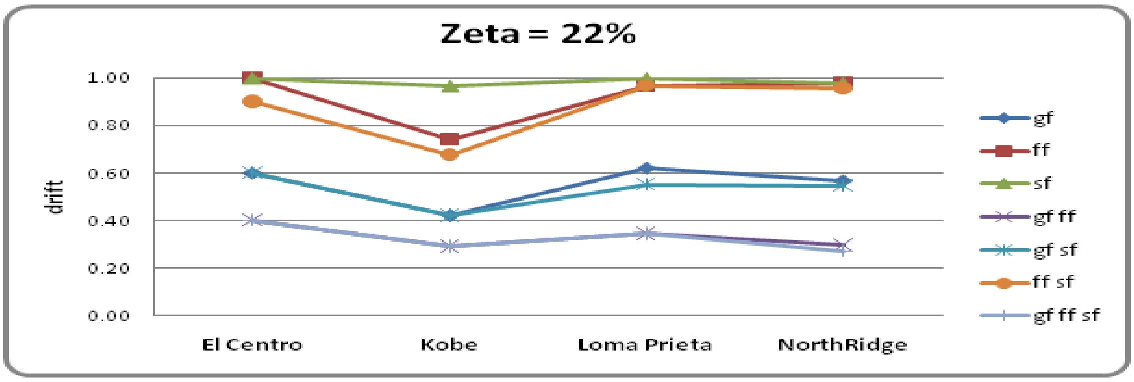

Figure 6.22: Inter-storey Drift Response for$\zeta = 22\%$

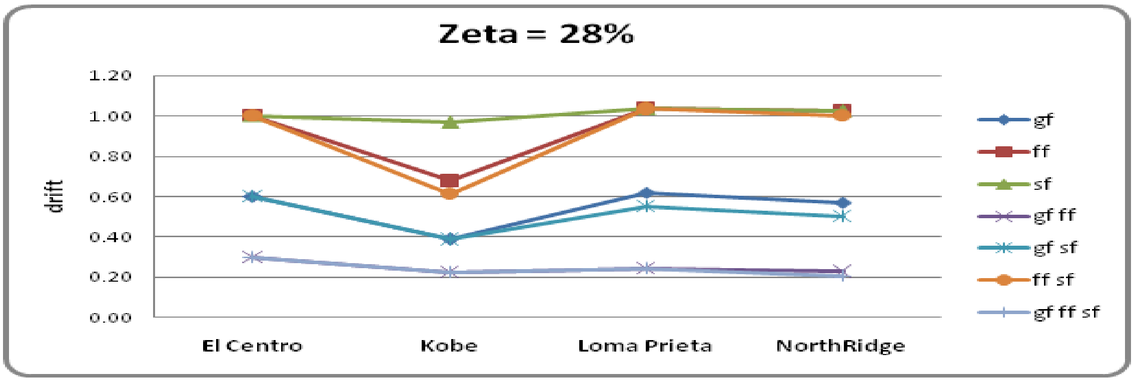

Figure 6.23: Inter-storey Drift Response for$\zeta = 28\%$

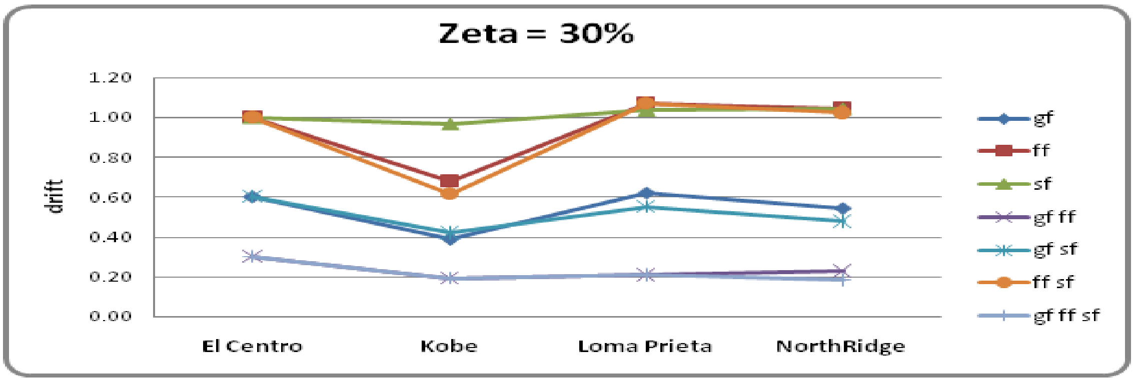

Figure 6.24: Inter-storey Drift Response forζ = 30%

The response of structure for different EQ excitation with different location of damper is also varies with change in damping ratio. With increased the damping ration, the structural response is also increased so, the ration of response quantities is decreased. The best earthquake excitation for damping value from 12 % to 20 % Kobe earthquake is best and for ζ 22% to 30% Northridge earthquake is best.

6.3.5 Comparison of Damper Force

The graphical representation shows the damper force where the damper is attached. From this graphical representation, it concludes that when damper is connected to the second floor maximum force is generate. With varies the ζ value from 12% to 30 %, the force is increased. When the number of damper is increased in structure, the generated force is decreased compare to force generate by lesser number of damper in structure.Figure shows the force v/s damping ratio graph for different condition of damper location with different value of ζ for different earthquake excitation.

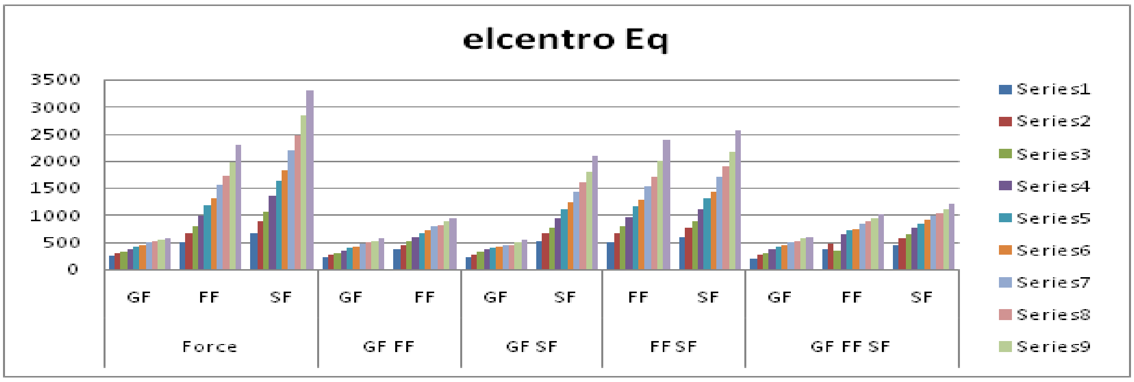

Figure 6.25: Comparison of Damper Force for Elcentro Earthquake

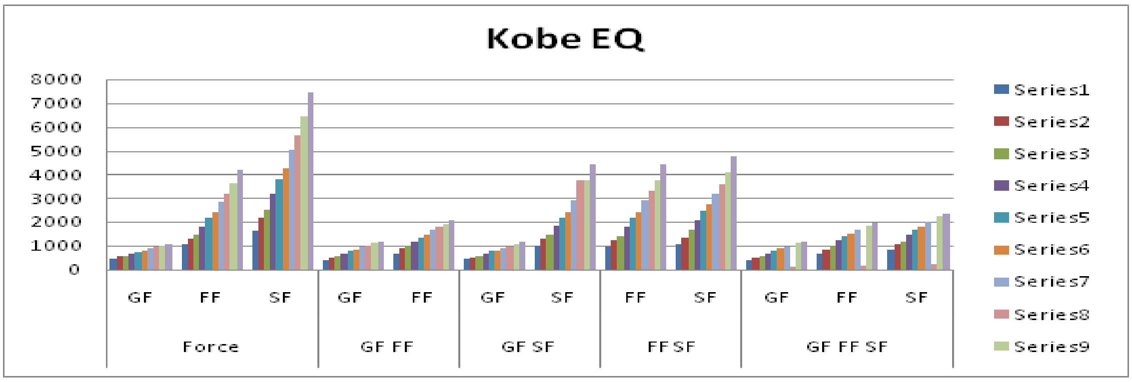

Figure 6.26: Comparison of Damper Force for Kobe Earthquake

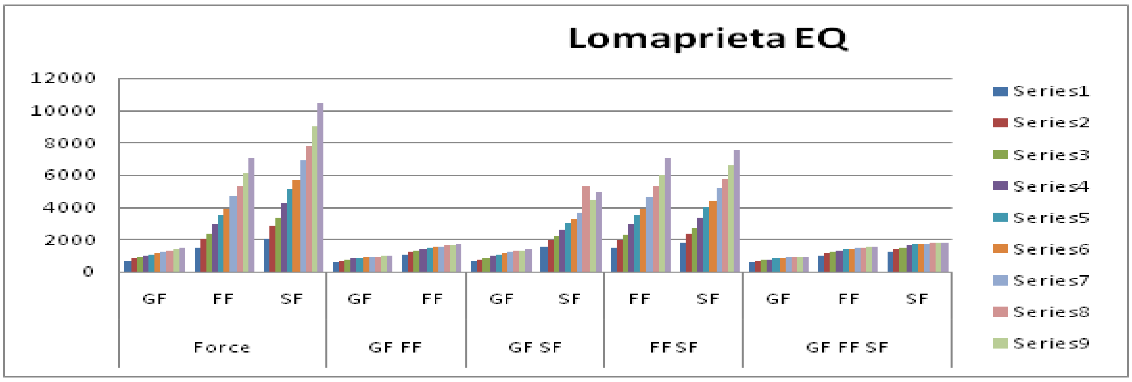

Figure 6.27: Comparison of Damper Force for Lomaprieta Earthquake

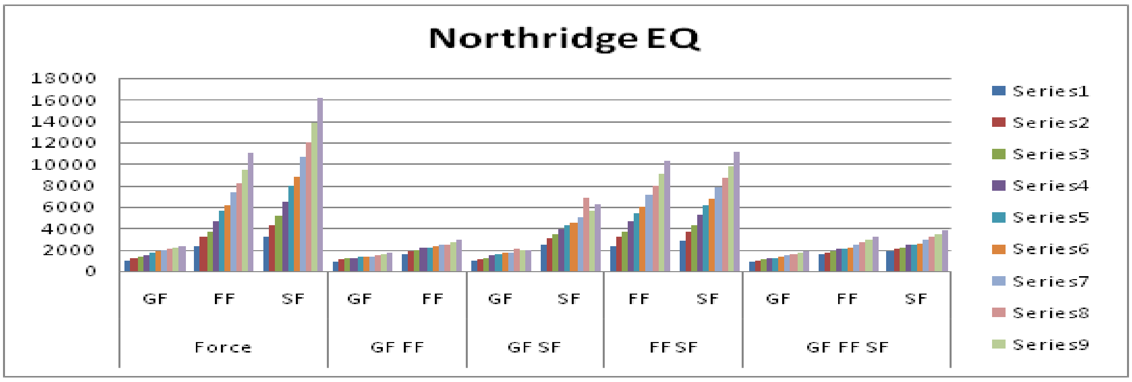

Figure 6.28: Comparison of Damper Force for Northridge Earthquake

6.4 Summary

This chapter deals with the response of structure, which was found out by numerical method Newmark Beta for different value of Damping ratio with different location of damper with four different earthquake excitation like Elcentro earthquake, Kobe earthquake, Lomaprieta Earthquake and Northridge Earthquake. From all response quantities like displacement, velocity, acceleration and drift conclude that for single damper in structure the best or suitable position for damper location is at GF. It follows by FF and SF respectively. For two damper in structure, the optimal location for damper is GF FF and which follows by GF SF and FF SF. For all different location of damper, the optimal condition is when damper is at GF FF SF.

Chapter 7

Summary & Conclusion

7.1 Summary

During various earthquake excitations, it is necessary to the structural remains in stable condition or in working condition for this the special techniques are required rather conventional seismic design. For structures subjected to strong earthquake motions, the inherent damping in the structure is not sufficient to mitigate the structural response, therefore extra damping is require in the form of energy dissipating systems. Three basic technologies are used to protect buildings from damaging earthquake effects. These are Base isolation, Passive energy Dissipation Devices and Active Control devices. In passive energy dissipation systems the motion of structure is controlled by adding devices to structure in the form of stiffness, mass and damping. In this work, the main focuses were to found out the optimal location of the passive energy dissipation devices like viscous damper and viscoelastic damper. To understand the behaviour of viscous and viscoelastic damper, characterization of this dampers have been carried out under the sinusoidal and different earthquake excitations namely, El Centro, Kobe, Loma Prieta and Northridge excitations.
A three storey shear building has been considered. This building is converted to lump mass model, and mass matrix and stiffness matrix are derived. A Rayleigh's damping

is assumed and damping matrix is obtained. Equations of motion for multi degree of freedom system subjected to earthquake excitations are derived with vary the damper location. Also, equation of motion for shear building with passive devices like viscous damper and viscoelastic damper are derived. These equations of motions are solving using numerical method like Newmark beta for uncontrolled and controlled building under the different earthquake excitations through MATLAB. Response quantities like maximum displacement, maximum velocity, maximum acceleration and maximum inter storey drift has been obtained for uncontrolled and controlled building. These response quantities of uncontrolled building have been compared with the controlled building.

7.2 Conclusion

The main aim of the work was to understand the optimal placement for the passive control devices namely, viscous damper and viscoelastic damper. The mathematical model and behaviour of viscous damper and viscoelastic damper are studied. From this mathematical model the characterization of viscous damper and viscoelastic damper for both sinusoidal and random earthquake excitations. Three storey shear building analysis has been done using time stepping numerical method Newmark-Beta for uncontrolled and the building equipped with passive energy dissipation devices, and extract the response quantities like maximum storey displacement, velocity, acceleration and damper force for four earthquake excitations through MATLAB.

Numerical results of three storey shear building equipped with viscous damper clearly indicate that the maximum roof displacement, maximum roof velocity, maximum roof acceleration and maximum inter storey drift are significantly reduces as co-efficient of damper 'C_d' increases, under four different types of earthquake excitations namely, El centro, Kobe, Northridge and Lomaprieta earthquake for the possible condition of damper with varies it's nos. Similarly, viscoelastic damper are effective in reducing all response quantities of building as required damping ratio increases under the earthquake excitations. This result indicates that amount of damping and stiffness directly influences the responses by reducing it.

- The best optimal location for both damper to the structure is damper was provided at all the stories GF FF and SF.
- The best optimal location for viscous damper is GF & FF, then GF & SF and FF & SF, respectively when two dampers were attached to the structure.
- The best optimal location for viscous damper is GF, then FF and SF, respectively when one damper was attached to the structure.

From all the results of different passive damper added three storey shear building, it is concluded that all are good enough to reduce all response quantities. It can be also concluded that viscous damper are more effective under the Northridge type of earthquake excitations, however viscoelastic damper are suitable for Loma Prieta type of earthquake excitations.

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Appendix A

Calculation of Eigenvalue and Eigenvector

As discussed earlier, three storey shear building is shown in Figure 4.1 is converted in to a Lump mass model, which is given in Figure 4.2. Calculation of Eigenvalues, Eigenvectors, Mass Matrix [M], Stiffness Matrix [K], Damping Matrix [D] of this lump mass model are found out as follows,

Building Configuration

Number of Stories	3 No.
Floor height (c/c)	3 m
Imposed load	3 kN/m^2
Percentage of Imposed Load	0.75 kN/m^2
Characteristics Strength of Concrete, f_{ck}	25 N/mm^2
Characteristics Strength of Steel, f_y	415 N/mm^2
No. of Bays In X-Direction	3 No.
No. of Bays In Y-Direction	3 No.
Bay Width In X-Direction	4 m
Bay Width In Y-Direction	4 m

Column size,	(0.3 x 0.3) m
Beam size,	(0.23 x 0.3) m
Depth of slab	0.12 m
Specific weight of R.C.C	25 kN/m^3
Specific weight of infill	0
Inherent Damping Ratio for Concrete Structure	5%

Lump Mass Calculation

At Roof Level		At Typical Storey	
Weight of Infill	0	Weight of Infill	0
Weight of Columns	54 kN	Weight of Columns	108 kN
Weight of Beams	165.6 kN	Weight of Beams	165.6 kN
Weight of Slab	432 kN	Weight of Slab	432 kN
Imposed Load	0 (IS 1893:2002)]	Imposed Load	108 kN
Total Roof Load	651.6 kN	Total Floor Load	1627.2 kN

Total Seismic Weight of Building, W = 2278.8 kN

Calculation of Eigenvalues and Eigenvectors

Mass Matrix of lumped mass model of building, M

$$[M]= \begin{vmatrix} M_1 & 0 & 0 \\ 0 & M_2 & 0 \\ 0 & 0 & M_3 \end{vmatrix} \text{Kg}$$

$$[M]= \begin{vmatrix} 82935.78 & 0 & 0 \\ 0 & 82935.78 & 0 \\ 0 & 0 & 66422.02 \end{vmatrix} \text{Kg}$$

Stiffness Matrix of lumped mass model of building, K

Column stiffness in X and Y direction, k=12EI/l^3

Total lateral stiffness of each story = No of columns in a story × k = 120000000 N/m

$$[\mathrm{K}]= \begin{vmatrix} K_1+K_2 & -K_2 & 0 \\ -K_2 & K_2+K_3 & -K_3 \\ 0 & -K_3 & K_3 \end{vmatrix}$$

$$[\mathrm{K}]= \begin{vmatrix} 240000000 & -120000000 & 0 \\ -120000000 & 240000000 & -120000000 \\ 0 & -120000000 & 120000000 \end{vmatrix} \mathrm{N/m}$$

For the above stiffness and mass matrices, eigenvalue and eigenvector are worked out using MATLAB as follows,

$$[K] \times [M]^{-1} = \begin{vmatrix} 2893.81 & -1446.9027 & 0 \\ -1446.9 & 2893.80531 & -1806.6298 \\ 0 & -1446.9027 & 1806.62983 \end{vmatrix}$$

Eigenvalues or natural frequencies of various modes are,

$$[\omega^2] = \begin{vmatrix} 320.82 & 0 & 0 \\ 0 & 2438.17 & 0 \\ 0 & 0 & 4835.25 \end{vmatrix}$$

ω_1=17.92 rad/sec, ω_2=49.38 rad/sec, ω_3=69.54 rad/sec,

The eigenvector (mode shapes) and natural periods corresponding to each natural frequency are,

$$[\phi] = [\phi_1\ \phi_2\ \phi_2] = \begin{vmatrix} 0.0012 & -0.0022 & 0.0027 \\ 0.0009 & -0.0029 & -0.0018 \\ 0.0012 & 0.0015 & -0.0032 \end{vmatrix}$$

$$T = \begin{vmatrix} 0.351 & 0 & 0 \\ 0 & 0.127 & 0 \\ 0 & 0 & 0.351 \end{vmatrix} \text{ sec}$$

Evaluate the Rayleigh Damping Matrix

By considering first mass-proportional damping and stiffness-proportional damping, C = $a_0 M + a_1 K$

Where, C is the rayleigh damping matrix; a_0 and a_1 are the co-efficient; M and K are the mass and stiffness matrix of building respectively. The co-efficient a_0 and a_1 can be determine from specified damping ratios ξ_i and ξ_j for the i th and j th modes, respectively. If all modes are to have the same damping ratio ξ, which is reasonable based on experiment data, therefore

$$a_0 = \frac{\xi \omega_i \omega_j}{\omega_i + \omega_j}$$

$$a_1 = \frac{2\xi}{\omega_i + \omega_j}$$

Where, ξ is the inherent damping ratio of the structure, ω_i and ω_j are the i th and j th natural frequency of of the building. Therefore, damping matrix of three storey building as per rayleigh's damping 'C' is,

$$C = \begin{vmatrix} 464401.54 & -177600 & 0 \\ -177600 & 464401.54 & -177600 \\ 0 & -177600 & 265057.87 \end{vmatrix} \text{ N Sec/m}$$

Appendix B

Design of Viscoelastic Damper

- The design of Viscoelastic damper is an iterative process. Many iteration were performed to achieve the exact area of Viscoelastic material pad. The design is carried out according to R. D. Hanson and T. T. Soong [20], which recommends Kelvin Model for analysis. To support the iterative calculations Microsoft Excel Sheet was used.
- The procedure for design was given in chapter 6 Section 6.2.1 with various equations. Calculation usually comprise of estimating additional stiffness and damping provided by the damper which were calculated by Equations 6.1, 6.3 and 6.4 explained in Chapter 6.
- Prior to design it is required to decide, desired damping ratio that should be achieved to reduce prescribed response level of building. In this study, the required structural damping ratio 'ζ' is assumed for the initial goal.
- Here sample calculation of design of damper is carried out for required damping ratio 'ζ' is equal to 20 %.

Data Taken:

Fundamental Frequency of the Building, ω	17.91 rad/sec
Inherent Damping Ratio of Building	5%
Storey Drift Ratio	0.40%
Operating Temperature, T	25^oC
Storey Height, h	3 m
Required Damping Ratio, ζ	20%
Angel Between Bracing Member and Floor, θ	36.86
Target Added damping Ratio, ς	15%
Assumed Loss Factor, η	1.2

1) From modifying modal strain energy method,

$$\alpha_d K_d = \frac{2\varsigma}{(\eta - 2\varsigma)} K_s$$

where, α_d is the attachment co-efficient is equal to $cos^2\theta$ for diagonally attached damper, and K_s = 120000000 N/m is the typical storey stiffness of the building. Therefore damper stiffness K_d is equal to 62490235.90 N/m

2) In this study of VE damper design, Maximum design damper deformation is $0.004 \times h \times cos\theta = 0.0096$ m.

3) If the maximum design damper strain of 60 % is allowed, then the damper thickness t is 0.0096/0.6 = 0.016 m.

4) Simplified relationship for shear storage and shear loss modulus is given by soong and dargush [11], from this relationship shear storage modulus, $G^{'} = 2068420$ N/m^2 and shear loss modulus, $G^{''} = 2482104$ N/m^2 are determined.

5) Area of viscoelastic damper is calculated using Equation **??**. Therefore, area

of viscoelastic damper $A = 0.48338527\ m^2$ for one layer.

6) If two VE layers are used per damper, then the selected dimension of each damper pad are, $A = 0.241692\ m^2$, Length of damper pad $L = 0.85$ m, Width of damper $B = 0.3$ m, and thickness of damper $t = 0.016$ m.

7) From area of VE damper, final stiffness of damper $K_d = 65930887.5$ N/m and co-efficient of damper $C_d = 4417479.899$ N sec/m are calculated using Equation **??** and Equation **??** respectively.

8) Similarly, VE damper design is carries out for different value of required damping ratio 'ζ', and damper stiffness K_d, co-efficient of damper C_d, and size of damper are find out, which is given in Table **??**.

Appendix C

MATLAB Code

A) MATLAB Code for Response of Viscous Damper Subjected to Sinusoidal Input (for varying value of amplitude)

```
% For linear viscous damper
w=6.28; % Frequency is constant in rad/sec
cd=160; % Damping co efficient in N*S/mm
for a = 20:5:40 %Amplitudes are varying(in mm)
t=0:0.002:2; %Time in Sec
x=a*sin(w*t); %Displacement in mm
x1=a*w*cos(w*t); % Velocity in mm/sec
f=(cd*x1); % Force in Damper in N
subplot(2,2,1:2)
plot(t,f,'g'); % Plot of Force Vs Time
grid on
xlabel('Time(sec)')
ylabel('Force(N)')
hold on
subplot(2,2,3)
plot(x,f,'g'); % Plot of Force Vs Displacement
```

```
grid on
xlabel('Displacement(mm)')
ylabel('Force(N)')
hold on
subplot(2,2,4)
plot(x1,f,'g'); % Plot of Force Vs Velocity
grid on
xlabel('Velocity(mm/sec)')
ylabel('Force(N)')
hold on
end
```

B) MATLAB Code for Response of Viscous Damper Subjected to Earthquake Excitations

```
cd=160000 ; % Damping coefficient in NS/m
t=0:0.01:40; %Time in Sec
fid1 = fopen('.txt file of El Centro Displacement Data');
x=fscanf(fid1,'%g'); %Displacement in cm
x=[0 ; x];
x=x.*0.01; %in m
fid2 = fopen('.txt file of El Centro Velocity Data');
x1=fscanf(fid2,'%g'); % Velocity in cm/sec
x1=[0 ; x1];
x1=x1.*0.01; % in m/sec
f=(cd*x1); % Force in Damper in N
subplot(2,2,1:2)
plot(t,f,'k');% Plot of Force Vs Time
title('Response of Viscous Damper');
```

```
grid on
xlabel('Time(sec)')
ylabel('Force(N)')
hold on
subplot(2,2,3)
plot(x,f,'k'); % Plot of Force Vs Displacement
grid on
xlabel('Displacement(m)')
ylabel('Force(N)')
hold on
subplot(2,2,4)
plot(x1,f,'k'); % Plot of Force Vs Velocity
grid on
xlabel('Velocity(m/sec)')
ylabel('Force(N)')
hold on
```

C) MATLAB Code for Response of Viscoelastic Damper Subjected to Sinusoidal Motion (for varying value of amplitude)

```
w=6.28; % Frequency is constant in rad/sec
kd=486.85; % Stiffness co efficient for VE Damper in N/mm
cd=93.093; % Damping co efficient in N*S/mm
for a=0.75:0.25:2; %Amplitudes are varying(in mm)
t=0:0.001:3; %Time in Sec
x=a*sin(w*t); %Displacement in mm
x1=a*w*cos(w*t); % Velocity in mm/sec
f=(kd*x)+(cd*x1); % Force in Damper in N
subplot(2,2,1:2)
```

```
plot(t,f); % Plot of Force Vs Time
grid on
xlabel('Time(sec)')
ylabel('Force(N)')
hold on
subplot(2,2,3)
plot(x,f); % Plot of Force Vs Displacement
grid on
xlabel('Displacement(mm)')
ylabel('Force(N)')
hold on
subplot(2,2,4)
plot(x1,f); % Plot of Force Vs Velocity
grid on
xlabel('Velocity(mm/sec)')
ylabel('Force(N)')
hold on
end
```

D) MATLAB Code for Response of Viscoelastic Damper Subjected to Earthquake Excitations

```
kd=468850; % Stiffness co efficient for VE Damper in N/m
cd=93093; % Damping co efficient in N*S/m
t=0:0.01:40; %Time in Sec
fid1 = fopen('.txt file of earthquake displacement data ');
x=fscanf(fid1,'%g'); %Displacement in cm
x=[0 ; x];
x=x.*0.01; %in m
```

```
fid2 = fopen('.txt file of earthquake velocity data ');
x1=fscanf(fid2,'%g'); % Velocity in cm/sec
x1=[0 ; x1];
x1=x1.*0.01; % in m/sec
f=(kd*x)+(cd*x1);% Force in Damper in N
x1max = max(abs(f))
subplot(2,2,1:2)
plot(t,f,'k'); % Plot of Force Vs Time
grid on
xlabel('Time(sec)')
ylabel('Force(N)')
hold on
subplot(2,2,3)
plot(x,f,'k'); % Plot of Force Vs Displacement
grid on
xlabel('Displacement(m)')
ylabel('Force(N)')
hold on
subplot(2,2,4)
plot(x1,f,'k'); % Plot of Force Vs Velocity
grid on
xlabel('Velocity(m/sec)')
ylabel('Force(N)')
hold on
```

E) MATLAB Code for Seismic Response of Uncontrolled Building to Find out Maximum Roof Displacement, Velocity and Acceleration using Newmark-Beta Method (El Centro EQ Excitation)

```
%Uncontrol Seismic Response of Three storey Building using newmark's method (El
centro)
clc;
close all
%mass matrix
m=[82935.78 0 0;0 82935.78 0;0 0 66422.02];
disp('mass matrix')
m
[ns,ms]=size(m);
fid=fopen('.txt file of Elcentro earthquake excitation acceleration Data,'r','r');
di=fscanf(fid,'%g');
di=di.*9.81%in m/sec^2
di=[0 ; di]
for i=1:ns
f(:,i)=-di*m(i,i);
end
%damping matrix in N sec/m
c=[465677.0273 -178334.295 0;-178334.295 465677.0273 -178334.295;0 -178334.295 265637.5122];
%stiffness matrix in N/m
k=[240000000 -120000000 0;-120000000 240000000 -120000000;0 -120000000 120000000];
kim=inv(m)*k;
[evec,ev]=eig(kim);
for i=1:ns
omega(i)=sqrt(ev(i,i));
end
disp('natural frequency')
omega
beta=1/4;
gamma=0.5;
```

```
%specify increment in time
dt=0.01;
%specify intial displacement
u0=[0 0 0];
v0=[0 0 0];
for i=1:ns
a0=inv(m)*(f(1,:)'-c*v0'-k*u0');
end
kba=k+(gamma/(beta*dt))*c+(1/(beta*dt*dt))*m;
kin=inv(kba);
aa=(1/(beta*dt))*m+(gamma/beta)*c;
bb=(1/(2*beta))*m+dt*(gamma/(2*beta)-1)*c;
u(1,:)=u0;
v(1,:)=v0;
a(1,:)=a0;
for i=2:4097
df(i,:)=f(i,:)-f(i-1,:)+v(i-1,:)*aa'+a(i-1,:)*bb';
du(i,:)=df(i,:)*kin;
dv(i,:)=(gamma/(beta*dt))*du(i,:)-(gamma/beta)*v(i-1,:)+dt*(1- gamma/(2*beta))*a(i-
1,:);
da(i,:) = (1/(beta*dt^2))*du(i,:)-(1/(beta*dt))*v(i-1,:)-(1/(2*beta))*a(i-1,:);
u(i,:)=u(i-1,:)+du(i,:);
v(i,:)=v(i-1,:)+dv(i,:);
a(i,:)=a(i-1,:)+da(i,:);
end
tt=linspace(0,40,4097);
unat1 = a(:,1) + di;%total acceleration
unat2 = a(:,2) + di;%total acceleration
unat3 = a(:,3) + di;%total acceleration
```

```
uncdi1=u(:,1);
uncdi2=u(:,2);
uncdi3=u(:,3);
uncve1=v(:,1);
uncve2=v(:,2);
uncve3=v(:,3);
uncac1=unat1;
uncac2=unat2;
uncac3=unat3;
%roof level
subplot(3,1,1)
plot(tt,u(:,3),'r');
um=max(abs(u(:,3)))
xlabel('Time(sec)');
ylabel('Roof Disp.(m)');
title('Displacement Response at roof');
subplot(3,1,2)
plot(tt,v(:,3),'r');
vm=max(abs(v(:,3)))
xlabel('Time(Sec)');
ylabel('Roof Velo.(m/sec)');
title('Velocity Response at roof');
subplot(3,1,3)
plot(tt,a(:,3),'r');
am=max(abs(a(:,3)))
xlabel('Time(sec)');
ylabel('Roof Accel.(m/sec^2)');
title('Acceleration Response at roof');
```

F) MATLAB Code for Seismic Response of Controlled Building to Find out Maximum Roof Displacement, Velocity and Acceleration using Newmark-Beta Method when damper at GF (El Centro EQ Excitation)

%dynamic analysis using direct integration method (Newmark Method)

clc;

clear all;

close all;

%mass matrix

m=[82935.78 0 0;0 82935.78 0;0 0 66422.02]

$[ns, ms] = size(m)$

fid=fopen('.txt file for earthquake excitation data','r');

di=fscanf(fid,'%g');

$di = di. * 9.81\% inm/sec^2$

$di = [0; di]$

for i=1:ns

f(:,i)=-di*m(i,i);

end

%damping matrix in N sec/m

$cs = [464401.54 - 1776000; -177600464401.54 - 177600; 0 - 177600265057.87]$

%stiffness matrix in N/m

\k=$[240000000{-}1200000000; -120000000240000000{-}120000000; 0{-}120000000120000000]$

%column vector of ones

$l = [111]$

%matrix determined by the placement of Viscous dampers in the structure

$b = [100; 000; 000]$

%damping matrix due to viscous damper in N sec/m

cd=5000000

c=cs+(b*cd)

```
format long;
kim=inv(m)*k
[evec, ev] = eig(kim)
for i=1:ns
omega(i)=sqrt(ev(i,i));
end
disp('natural frequency')
omega
% specify integration parameter for average acceleration method
beta=1/4;
gamma=0.5;
% specify integration parameter for linear acceleration method
% beta=1/6; gamma=0.5;
%specify intial displacement
u0 = [000];
v0 = [000];
for i=1:ns
a0=inv(m)*(f(1,:)*l'-c*v0'-k*u0')
end
%specify increment in time
dt=0.01;
kba=k+(gamma/(beta*dt))*c+(1/(beta*dt*dt))*m;
kin=inv(kba);
aa=(1/(beta*dt))*m+(gamma/beta)*c
bb=(1/(2*beta))*m+dt*(gamma/(2*beta)-1)*c
u(1,:)=u0
v(1,:)=v0
a(1,:)=a0
for i=2:4001
```

```
df(i,:)=f(i,:)-f(i-1,:)+v(i-1,:)*aa'+a(i-1,:)*bb';
du(i,:)=df(i,:)*kin;
dv(i,:)=(gamma/(beta*dt))*du(i,:)-(gamma/beta)*v(i-1,:)+dt*(1-gamma/(2*beta))*a(i-1,:);
da(i,:)=(1/(beta*dt^2))*du(i,:)-(1/(beta*dt))*v(i-1,:)-(1/(2*beta))*a(i-1,:);
u(i,:)=u(i-1,:)+du(i,:); %Relative disp.
v(i,:)=v(i-1,:)+dv(i,:); %Relativr velo.
a(i,:)=a(i-1,:)+da(i,:); %Relative Accele.
end
tt=linspace(0,40,4001);
%find total acceleration
at3=a(:,3)+di
at2=a(:,2)+di
at1=a(:,1)+di
%roof level
subplot(3,1,1)
plot(tt,u(:,3),'k');
um=max(abs(u(:,3)))
xlabel('Time(sec)');
ylabel('Roof Disp.(m)');
title('Displacement Response at roof');
subplot(3,1,2)
plot(tt,v(:,3),'k');
vm=max(abs(v(:,3)))
xlabel('Time(Sec)');
ylabel('Roof Velo.(m/sec)');
title('Velocity Response at roof');
subplot(3,1,3)
plot(tt,at3,'k');
```

```
am=max(abs(at3))
xlabel('Time(sec)');
ylabel('Roof Accel.(m/sec^2)');
title('Acceleration Response at roof');
```

Similarly several other program are developed and are not discussed here.