APPLICATION OF DAMPERS IN HIGH-RISE BUILDINGS

By

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DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

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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.

Krushin N. Karavadia

Certificate

This is to certify that the Major Project entitled "Application of Dampers in High-Rise Buildings" submitted by Mr. Krushin N. Karavadia, 08MCL007, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering of Nirma University, 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

In order to control the vibration response of high rise buildings during seismic events, energy absorbing passive damping devices is most commonly used for energy absorption. Basically a passive damper requires no external energy to function. It dissipates the energy by its own characteristics. Today there are number of types of manufactured dampers available in the market, which uses a variety of materials and designs to obtain various levels of stiffness and damping. Some of this includes viscous, viscoelastic, friction, yielding, tuned liquid and tuned mass dampers. These dampers can be installed in new buildings as well as existing building for retrofit purposes. Effective damping systems can results in higher levels of safety and comfort.

The present study is an attempt to understand the behavior of various passive damping systems with principle of working, design issues and their practical applications. The effectiveness of passive dampers installed is studied from two case studies which include their applications in La Gardenia Housing Complex situated at Gurgaon and Taipei 101 Tower which is situated at Taiwan.

The seismic mitigation of high-rise buildings using embedded dampers is investigated. Two types of damping mechanisms, i.e. friction and tuned mass dampers are investigated. For design of tuned mass damper mathematical model is illustrated. In addition, a general discussion on the selection criteria and design guidelines are also included in this study. A various methods of analysis of structure embedded with passive dampers are presented. A procedure for computer modeling of tuned mass damper and friction damper is discussed in detail.

Finite element models were employed in the analysis using the computer program SAP2000 (Structural Analysis Program) version 14.1. A Fast Nonlinear modal time history Analysis and response spectrum analysis were carried out to obtain the

damped and undamped responses of the structure. For understanding the behavior of damping systems under various seismic excitations, four different characteristics of earthquake are selected. The earthquake events used in this study are El Centro, Kobe, Northridge and Loma Prieta time histories and response spectra. It has been applied as acceleration time-histories at the base of the structure in the horizontal plane. Concrete material properties were chosen to represent the model, as many high-rise buildings are constructed by using reinforced concrete.

A strategy for protecting building from earthquakes is to limit the tip deflection and tip acceleration, which provides an overall assessment of the seismic response of the structure. Several medium and high-rise building structures with embedded friction dampers and tuned mass damper were studied subjected to different earthquake loadings. The change in response of building due to dynamic force is studied for controlled and uncontrolled buildings. The efficiency of energy dissipating dampers for vibration control of structure was investigated for five different placements of dampers and to study the influence of location on the seismic response.

The results of analysis is compiled in form of displacement, acceleration, storey drift, time period, frequency, energy dissipation capacity of dampers and compared with uncontrolled structures. Comparison between tuned mass damper and friction damper is also carried out to demonstrate the feasibility of the technique for seismic mitigation of the structure for medium and high-rise buildings. Results also provide the information which can be used for optimal damper placement for reducing the maximum seismic response.

This study has investigated the use of friction damper and tuned mass damper to mitigate the seismic input energy within medium and high-rise structures. These damping devices were embedded in a variety of different placement (one at a time). The appropriate damper for reducing the seismic response of structure is finding out for medium and high-rise structure. Parametric study is carried out by varying location and type of damper.

Results from this study indicates that the performance of friction damper and tuned mass damper are dependent on the characteristics of earthquake ground motion and both are quite effective in reducing the structural dynamic response in the form of acceleration, displacement and storey drift. Friction dampers are most effective when placed close to regions of maximum storey drift, while the best performance of tuned mass damper was achieved when placed at top storey.

Tuned mass damper gives the significant performance with increasing the height of structure, while friction damper gives incredible performance for medium-rise structures. So for high-rise structure tuned mass damper is preferred and medium-rise structure friction damper should be used. It is possible to achieve seismic mitigation, under all earthquake excitations, for all the structures considered in this study, by using appropriate damper types suitably located within the structure.

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With great pleasure I express my sincere gratitude towards my guide **Prof. G. N. Patel**, Professor, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad for his valuable guidance and supervision. From the very beginning of the major project, his valuable discussions and continuous reviews and suggestions helped me a lot during this study.

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Abbreviation Notation and Nomenclature

A_h .... Design horizontal seismic coefficient

BMF .... Bending Moment resisting Frame

C .... Absorber damping

C_f .... Force coefficient

C_j .... Damping constant

C_{opt} .... Optimum damping constant of TMD

DBE .... Design Basis Earthquake

F .... Design wind load

FD .... Friction Damper

FDF .... Friction-Damped Frames

F_m .... Force applied to main mass

F_{max} .... Frictional force

FNA .... Fast Nonlinear time history Analysis

f_{opt} .... Optimum frequency ratio of TMD

H .... Height of building

h_i .... Height measured from the base of the building to floor

I .... Importance factor

k_{opt} .... Optimum stiffness ratio of TMD

K_T .... Stiffness constant

k_1 .... Risk coefficient

k_2 .... Terrain, Height and structure size factor

k_3 .... Topography factor

M .... Main mass of structure

m .... Mass of tuned mass damper

MCE .... Maximum Considerable Earthquake

N .... Normal force

n .... Number of storey

$P_o sin\omega t$ .... Force acting on main mass

P_z .... Design wind pressure

Q_i .... Lateral force at i storey

R .... Response reduction factor

S_a/g .... Average response acceleration coefficient

SWF .... Frames with Shear Walls

T .... Time period of building

Ta .... Approximate fundamental period

T_g .... Time period of ground motion

TMD .... Tuned Mass Damper

V_B .... Design seismic base shear

V_b .... Basic wind speed

V_i .... Storey shear at i storey

V_z .... Design wind speed

W_i .... Seismic weight of i storey

Y .... Displacement of main mass

$\dot{Y}$ .... Velocity of main mass

$\ddot{Y}$ .... Acceleration of main mass

Z .... Zone factor

z .... Displacement of auxiliary mass

$\dot{z}$ .... Acceleration of auxiliary mass

μ .... Mass ratio

ξ .... Damping ratio

$\xi_{d_{opt}}$ .... Optimum damping ratio of TMD

Ω .... Natural frequency of TMD

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Chapter 1

Introduction

1.1 General

As the urban population increases, construction of high-rise building gives a viable solution to the problems associated with urban population. Advances in material science and construction technology are making it possible to construct high-rise buildings. The high-rise buildings are able to carry gravity loads within its elastic limit but they are susceptible to carry dynamic (lateral) load such as wind load, wave load and earthquake load. High-rise buildings are susceptible to high drifts and consequently high acceleration under dynamic loading.

During an earthquake large amount of energy is to be released and it is imparted to structures in the form of dynamic load. Dynamic load produces vibration in the structures, which causes damage or catastrophic failure of the structures. To reduce unwanted vibration in a structure some mechanism is required to absorb or to dissipate the energy. The conventional approach for seismic design has been based upon providing a combination of strength and ductility to resist loading. Conventional design practice permits inelastic action in buildings and providing significant energy dissipation when the buildings are subjected to dynamic load.

As a response to the shortcoming in conventional seismic design a number of innovative approaches have been developed. Modern structural protective systems can be divided into three major groups: Seismic Isolation , Passive Energy Dissipation and Semi-active and Active systems as shown in Table 1.1 [1].

Table 1.1: Modern Structural Protective Systems

Seismic Isolation	*Passive Energy Dissipation*	*Semi-active and Active Control*
Elastomeric Bearings Lead Rubber Bearings Sliding Friction Pendulum	Viscous Fluid Dampers Viscoelastic Dampers Friction Dampers Metallic Dampers Tuned Mass Dampers Tuned Liquid Dampers	Active Bracing Systems Active Mass Dampers Smart Materials

These systems are characterized by inelastic behavior in specially design and detailed regions of the structure. The aim of developing these systems is to protect the structural elements against drift and displacement. In recent years serious effort have been undertaken to develop the concept of energy absorbing devices. The basic role of all systems when incorporated into a structure is to absorb or consume a portion of the input energy, thereby reducing energy dissipation demand on primarily structural members and minimizing possible structural damages.

The technique of seismic isolation is now widely used in the world. It is typically placed at the foundation of structure. The isolation system partially reflects and partially absorbs some of the earthquake input energy before this energy can be transmitted to structure. In Semi-active and Active structural control systems motion of a structure is controlled or modified by means of action of a control system through some external energy supply. Semi-active systems require only nominal amounts of energy to adjust their mechanical properties then fully Active systems. They cannot add energy to the structure. In Passive Energy Dissipation systems 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 induced motion as well as earthquake induced motion.

The human comfort criteria are also important for high-rise buildings. To increase occupant comfort during severe wind or earthquake excitation, structural response should be improved. However for improving human comfort, structural response can be improved by reducing interstorey drift or by increasing energy dissipation capacities of the structure or by decreasing structural vibration.

1.2 Background

Conventional seismic design attempts to make buildings that do not collapse under strong earthquake shaking, but may sustain damage to non-structural elements and to some structural elements in the building. Hence, it may cause large interstorey drift in high-rise or flexible structures. This may lead building to non-functional use after the earthquake. In order to protect building special techniques are required to design building, such that they remain practically undamaged even in severe earthquake. For high-rise buildings the addition of energy dissipation systems has been successfully employed throughout the world. An example of successful implementation of Tuned Mass Damper is the Taipei 101 of Taipei city of Taiwan country.

Over last 25 years, significant progress has been made in developing manufactured Passive Energy Dissipation devices. Simultaneously analytical and experimental studies have been done for better understanding and characterization of effects of Passive Energy Dissipation devices on earthquake response of buildings. The principle of introduction of Passive Energy Dissipation devices in structures is to introduce damping elements to improve the structural response in the form of interstorey drift and floor accelerations. The advantages of Passive Energy Dissipation devices include the ability to eliminate or significantly reduce structural and nonstructural damage to enhance the safety of structures and to reduce seismic design forces.

1.3 Need of Dampers in High-Rise Structures

It has been studied from wind tunnel test, the designing of tall building to meet a given drift limit under code-specified equivalent static load is not enough to make occupants comfortable during wind storms. However they have only limited control over three factors, namely height, shape and mass that influence the dynamic response of buildings.

Excessive vibration of building can be reduced by three ways. Firstly, additional stiffness can be provided to reduce the vibration period of a building. Secondly, change in mass of a building can be effective in reducing excessive wind-induced excitation. Finally, aerodynamic modifications to the building shape can be effective in reducing vortex shading phenomena, caused by wind [2].

Table 1.2: Options to Prevent Wind-Induced Response of High-rise Building

Design options	*Method and Aim*	*Solutions*
Aerodynamic design	Improving aerodynamic properties to wind force coefficient	Chamfered corner openings
Structural design	Increasing building mass to reduce air/building mass ratio	Increase material costs
	Increase stiffness of natural frequency to reduce non-dimensional wind speed	Bracing walls, thick members
Damping device	Addition of materials with energy dissipative properties, increasing building damping ratio	Viscous, Viscoelastic, Friction and Metallic damper
	Adding auxiliary mass system to increase level of damping	Tuned mass and Tuned Liquid Damper

The above traditional methods (i.e. change in stiffness, mass or aerodynamic shape) can be implemented only up to a certain point, beyond which the solution become unworkable because of other design constrains such as cost, space or aesthetics. Therefore to achieve reduction in dynamic response, a practical solution is to supplement damping of the structure with an external mechanical damping system.

A big variety of damping devices can be implemented in the structure in order to increase the damping and thus decrease acceleration which causes human discomfort. In Table 1.2 various options to prevent wind-induced response of high-rise buildings are summarized.

1.4 Objective of Study

Analytical methods are now available to analyze and evaluate structures installed with passive energy dissipation devices. However, to design a structure with passive energy dissipation devices, that is to calculate the required size of a device to achieve a desired response reduction. Furthermore, to get the most out of a device, the optimal decisions about its placement location and its size are quite important. Even with nonlinear energy dissipation devices such as tuned mass damper and friction damper, the optimal placement and sizing of the devices is not a straightforward task. It is difficult to calculate the size and placement of dampers to achieve a desired performance. For such devices, the design procedure becomes very complex and remains highly iterative.

There is a need to develop systematic and quantitative approaches to popularize the use of passive energy dissipation devices in effective manner in the practice of earthquake engineering. With the currently available computing facilities and developments in the area of structural optimization, it now appears quite possible to design building structures installed with passive energy dissipation devices in an optimal manner. Also, it is quite important to know the sensitivity of an optimal design with respect to the design input and system parameter, as these parameters can vary in practice.

The main objective of present study is to formulate a general framework for the optimal design of passive energy dissipation devices for improving the response of high-rise buildings in order to achieve occupant comfort and control large drift during severe wind and earthquake. Two different types of energy dissipation systems have been selected for study, and they are tuned mass damper and friction damper. The main objectives of this study are as follows.

- To understand the fundamental concepts of passive energy dissipation systems for R.C.C buildings.
- Modeling of passive dissipation devices like as Tuned Mass Dampers and Friction Dampers.
- Response spectrum analysis of R.C.C. building with and without dampers for IS: 1893 (Part-I)-2002, Kobe, Northridge and Loma Prieta earthquake response spectra.
- Fast Non-linear Time-history analysis of structure for El Centro, Kobe, Northridge and Loma Prieta earthquakes acceleration time-history.
- Effect of positioning of damper devices in regular framed and wall-framed R.C.C structure.
- Comparison of controlled structure and uncontrolled structure and its various parameters.

1.5 Scope of Work

To achieve above objectives the scope of work is as follows;

- Study of various literatures for exploring the need of Energy dissipation devices, implementation and working of such devices.
- Study of various damping devices and their principle with practical application.
- Detailed study and design of Tuned Mass Damper and Friction Damper.
- A study of uncontrolled and controlled 3-D R.C.C buildings of 5-storey, 15-storey and 25-storey using SAP2000 (V 14.1).
- The response spectrum analysis of 3-D buildings as per IS: 1893-2002 part-I, with and without dampers.
- The Fast Non-linear time history Analysis (FNA) of 3-D buildings as per FEMA-273 and FEMA-274 guidelines.
- Compilation of results related to Displacement, Drift, Time Period, Frequency, Force carried by dampers, Energy dissipated by dampers and time history of acceleration and displacement.
- Parametric study of various damping systems based on alignment and location.

1.6 Organization of Major Project

The contents of major project are divided into various chapters as follow;

Chapter 1, *Introduction*, presents the introduction and background of the major project work. It also includes objectives of study and scope of work. The need of dampers in high-rise building is also explained in this chapter.

Chapter 2, *Literature Survey*, describes the previous research work related to the topics. This chapter provides the understanding the effects of damper in the building.

chapter 3, *Passive Damping Techniques and Case study*, deals with introduction of various passive damping techniques. The principle of working, design issues and practical applications of various dampers are discussed. The case study of Taipei 101 for tuned mass damper and another case study of La Gardenia Housing towers for friction damper is also presented in this chapter.

chapter 4, *Analysis of Passive Dampers*, presents the needs of supplemental damping in the structures. This also includes the method of analysis of structure incorporated with passive dampers. The concept and procedure for computer modeling for SAP2000 software and mathematical modeling of tuned mass damper and friction damper with design criteria is presented.

chapter 5, *Analysis of 5-Storey Structure*, deliberates the results evaluated from Response spectrum and Fast Nonlinear time history Analysis for uncontrolled and controlled structure. Effectiveness of tuned mass damper and friction damper was evaluated for various earthquakes.

chapter 6, *Analysis of 15-Storey Structure*, discusses the comparison the results of Fast Nonlinear time history analysis and Response spectrum analysis for uncontrolled and controlled structure for various earthquakes. Comparison between tuned mass damper and friction damper was done for finding out the efficiency of suitable damper for 15-storey wall frame structure.

chapter 7, *Analysis of 25-Storey Structure*, presents the results for uncontrolled and controlled structure and also comparison between two damping systems was done for appropriate selecting of damping system for 25-storey structure.

chapter 8, *Parametric Study*, describes the comparison of damping systems for medium and high-rise structures. Parametric study based on location of damper was done for choosing the optimal placement of damper.

chapter 9, *Summary and Conclusion*, gives summary and conclusion of the present work and also discuss about future scope of work for further study on this topic.

1.7 Methodology

The methodology of the major project in the form of flow chart is illustrated in the Figure 1.1.

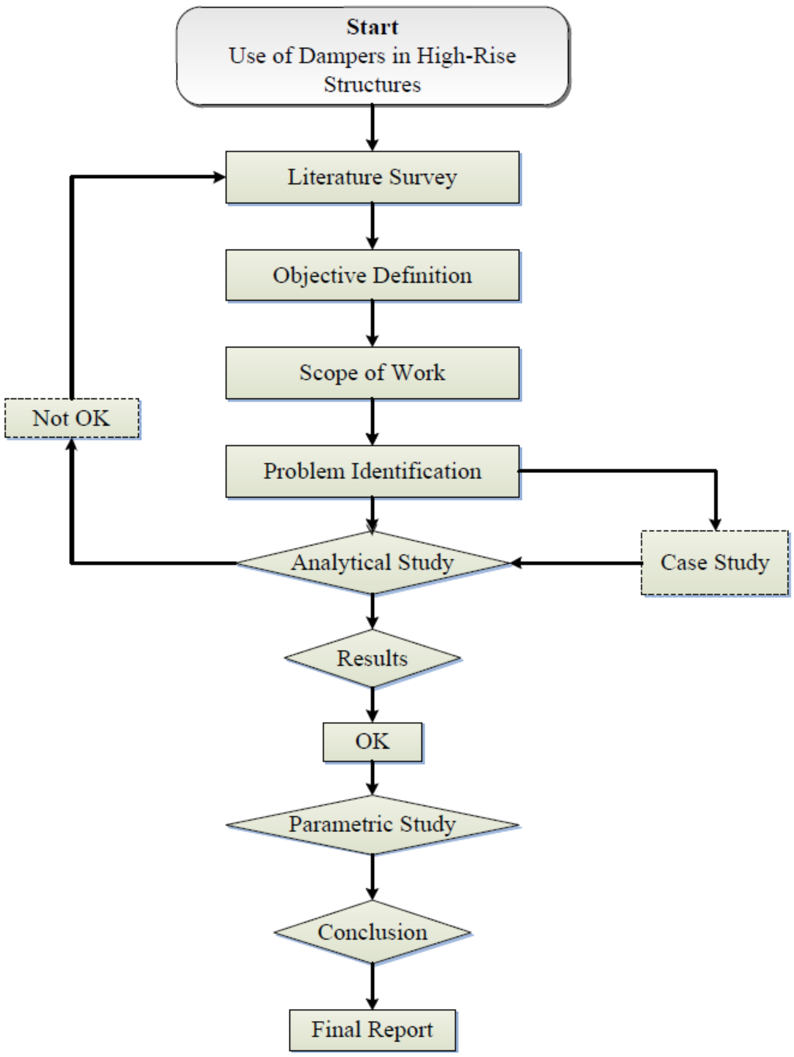


Figure 1.1: Methodology

Chapter 2

Literature Survey

2.1 General

In recent years the application of passive energy dissipation systems have seen successfully installed for rehabilitation as well as new construction of high-rise buildings for mitigating the effects of environmental disturbances such as wind and earthquake. An extensive research is going on this topic for understanding of behavior of various passive energy dissipation systems. Literature survey is carried out in the form of various journal papers, books, guidelines etc. to familiar with the amount of work done in this area throughout the world. It also helps in deciding the scope of work.

2.2 Literature Review

Various literatures have been referred for basic understanding, modeling and analysis of structure with passive energy dissipation systems. Some of the important papers are summarized below.

A.S. Sajith et al.[3] presented the efficient design of Tuned Mass Damper for controlling structural response of flexible structures subjected to ground motion. The Tuned Mass Damper is defined as a auxiliary spring and mass. Analytical and experimental studies are conducted on small scale models for finding the effectiveness of response of TMD. The following equation of motion is used in governing a system. The mathematical model of TMD are shown in Figure 2.1.

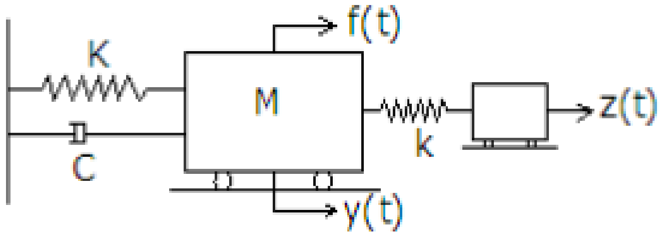


Figure 2.1: Addition of Auxiliary Spring Mass System

$$M\ddot{y} + C\dot{y} + K\left(y - z\right) = Fe^{i\omega t} \tag{2.1}$$

$$m\ddot{z} + k\left(z - y\right) = 0 \tag{2.2}$$

A G+2 storey models have prepared and excited with harmonic loading on a uni-axial shake table. For experimental setup plates are used to represent the building floor and flats represent the columns. For analytical study MATLAB program has been prepared for mathematical modeling of structure attached with TMD. By introducing a TMD in buildings, the response of building subjected to ground motion is reduced in the range of 80-90%. In the design of TMD in real structure aesthetics of the structure should be taken into account.

Carlos Y. L. et al.[4] discuss the use of friction dampers for seismic retrofit of the structure. A four storey existing steel frame is investigated for retrofit purpose. For reducing the seismic forces and movement in the structure friction damper is introduced for upgrading the structure. For reducing the seismic response twenty four 250kip friction dampers are placed at 1^{st} floor and 2^{nd} floor. 3-D models have created using ETABS to analyze the building's as-built and retrofitted conditions. The performance based analysis done as per FEMA-351 and FEMA-356 guidelines for steel moment resisting frame. A comparison was made for structure with braced frame, braced frame with friction dampers and existing moment resisting frame. From comparison, it is investigated that, friction damper retrofit scheme was significantly improve the structural performance of the building in terms of 50% and 20% reduction in storey displacement and storey shear respectively.

Chien Liang Lee et al.[5] proposed an optimum design theory for the TMD for undamped SDOF structure to control structural vibration. The Power Spectral Density (PSD) is used for considering environmental disturbances. A numerical method for determining the optimal design parameters for the TMD was proposed in a systematic fashion such that the numerical solution converges monotonically towards the exact solutions as number of iteration increase. The exact solution for the optimal design parameters of a single tuned mass damper for a SDOF system was presented by,

$$f_{opt} = \sqrt{\frac{1 + \mu/2}{1 + \mu}} \tag{2.3}$$

$$\xi_{opt} = \sqrt{\frac{\mu + (1 + 3\mu/4)}{2\,(1 + \mu)\,(2 + \mu)}} \tag{2.4}$$

Where 'f_{opt}' is optimal design frequency and 'ξ_{opt}' is optimal design damping ratio.

The optimal design parameters of TMD in terms of damping coefficient and spring constant are investigated by minimizing a performance index and increasing number of iteration. A ten-story shear building with single TMD at top is analyzed for El Centro earthquake for verification of the proposed numerical method. The top displacement and acceleration of structure with and without TMD has compared for ten-story building. The feasibility of proposed optimal design theory is verified by using a single DOF structure with single TMD, MDOF structure with single TMD and MDOF structure with multiple TMD.

K.C.S. Kwok and B. Samali[6] discuss a brief description of theory of TMD and its design and practical applications. The proposed one-degree of freedom system for TMD is shown in Figure 2.2. Various parameters such as efficiency, compactness, weight, capital cost, operating cost, maintenance & safety is studied of some well-known tall buildings. Parametric studies were conducted for wind tunnel testing and shake table model tests. For passive system, addition of 3 to 4% of critical damping, wind-induced response of building is reduced to 40 to 50%. For active system, addition of 10% critical damping, wind-induced response of building is reduced up to 2/3.

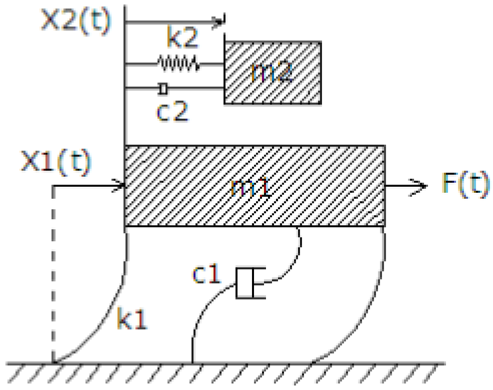


Figure 2.2: Single-degree-of-freedom System with TMD

Kiran K. Shetty et al.[7] discusses an effect of TMD to control the response of plane frame structure. The response of four-story plane frame structure having 3-DOF (i.e. translation about X & Y and rotation about Z-axis) at each node with multiple tuned mass dampers on its top has been studied. The harmonic ground motion was considered for study of TMD on plane frame structure. Tuned mass damper is modeled using two-nodded element having one DOF in X-direction at each node. The parameters of TMD has obtained by following equations,

$$K_T = \frac{\mu m_s}{\sum_{j=1}^{n} 1/\omega_j^2} \tag{2.5}$$

$$C_j = 2m_j \xi_T \omega_j \tag{2.6}$$

Where 'K_T' and 'C_j' is stiffness and damping constant of TMD, 'μ' and 'ξ' is mass ratio and damping ratio respectively. The top floor displacement, acceleration, base shear and bending moment in the members due to harmonic ground excitation are studied for with and without TMD. The peak value of the top floor displacement, top floor acceleration, base shear and bending moment decreases due to MTMDs. The effectiveness of TMD depends upon the frequency characteristics of ground motion. The multiple tuned mass dampers are more effective than a single tuned mass damper.

Rahul Rana and T.T. Soong[8] present the simplified design of Tuned Mass Damper. A parametric study was carried out for understanding of characteristics of Tuned Mass Damper. A mathematical modeling of TMD with structure is shown in Figure 2.3.

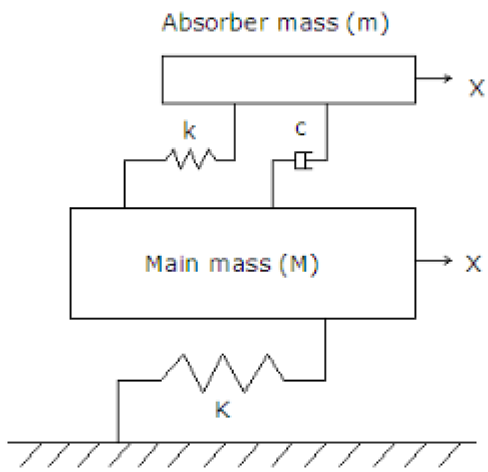


Figure 2.3: Den Hartog Damped Vibration Absorber

Following simplified formula are presented for finding optimum design parameters of TMD.

$$f_{opt} = \frac{1}{1+\mu}\left(\sqrt{\frac{2-\mu}{2}}\right) \tag{2.7}$$

$$k_{opt} = f_{opt}^2 \Omega^2 m \tag{2.8}$$

$$c_{opt} = 2\xi_{d_{opt}} f_{opt} \Omega m \tag{2.9}$$

Where 'f_{opt}', 'k_{opt}', 'c_{opt}' and 'ξ_{opt}' is optimum values of frequency, stiffness, damping and damping ratio of TMD respectively and Ω is natural frequency of main mass.

For study the effect of detuning steady state harmonic analysis and time-history analysis for El Centro earthquake is performed with varying excitation frequencies, damping and mass ratio. The relationship between designs of TMD for a SDOF structure and a certain mode of a MDOF structure is obtained to simplify design of TMD. For finding out the best suitable level of TMD and optimum TMD parameters an example is illustrated with 3-DOF and 2% building mass as taken as damper mass. The effect of detuning in TMD parameters becomes less affected with increasing structural damping and mass ratio. By increasing the structural damping TMD response will decrease. Modal contamination problem was occurred for 3-DOF structure, because of second and third mode of TMD deteriorating the first mode response.

Ramesh Chandra et al.[9] proposed the solution for seismic control of La Gardenia Towers, Gurgaon, India. It consists of 7 towers of eighteen storeys with two levels of basements. A total of 66 pall friction dampers are placed in steel bracing in concrete frames of one towers for extract sufficient energy. For analysis of friction damped structure, building codes of U.S., Canada and some other country are used. A nonlinear time history analysis was demonstrated for showing the influence of friction dampers. Three-dimensional nonlinear time history dynamic analysis were carried out using ETABS. Friction damper is modeled as ideal elasto-plastic nonlinear element and slip load of friction damper is considered as fictitious yield force. The diagram of Cross brace friction-damper is shown in Figure 2.4.

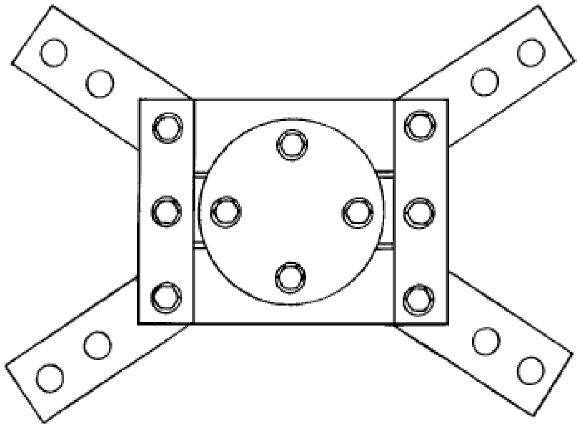


Figure 2.4: Cross Brace Friction Damper

The analysis were conducted on friction-damped frames (FDF), braced-moment frames (BMF) and frames with shear walls (SWF) for comparing the effectiveness of friction damper. By introducing the friction damper in the structure, response of structure is reduced by 63%, 62%, 40% and 27% in terms of peak amplitude, storey shear, energy dissipation and axial forces in column respectively.

Robert D. Hanson[10] discusses the broad categorization of various measures to improve the earthquake response of the structure. These measures were classified as active system, passive systems and hybrid systems. An active system is that which requires the active participation of mechanical devices whose characteristics are made to change during the building response on the basis of current response measurement. Passive system comprises of energy dissipation devices and base isolation which does not require any external output. The Hybrid systems are combination of the passive and active system, so that, safety of building is not compromised even if active system fails.

W.L. He and A.K. Agrawal[11] describe the performance of structure with passive energy damper subjected to near fault earthquakes. The performance of passive viscous and friction damper for elastic and inelastic structures with SDOF system has been investigated. A mathematical modeling for Coulomb friction damper is formulated. The dissipation of seismic energy of structure with passive viscous damper, passive friction damper and without dampers is plotted for various displacement ductility ratios. For elastic structures, friction dampers are more effective than viscous dampers for long period structure but for short period i.e $T/T_g < 1.5$ viscous dampers are more effective. For inelastic structures with $\mu = 4$, friction dampers are effective over the natural period range of $0.3 < T/T_g < 1$ and viscous dampers are more effective for $T/T_g < 1$. The acceleration of structure with friction damper is higher than structure with viscous dampers. For friction damper hysteric energy decreases rapidly with friction force is increased hence degree of damage will be reduced. Viscous damper are more effective when input energy and hysteric energy decrease and viscous energy increases.

Z. Rakicevic et al.[12] presented the effectiveness of Tuned Mass Control Systems(TMCS) based on very large experimental data which is obtained by shaking table experiments and mathematical modeling which is done using SAP2000 computer program. A five-story three-bay steel frame model of a hypothetical building has been prepared with TMCS which is installed at top of building and it has been tested on biaxial shaking table. For analytical study mathematical modeling of tested model with and without Tuned Mass Control Systems has been done using SAP2000. Tuned Mass Control Systems is modeled as a two-nodded link element using springs and dampers. For analytical study the data of stiffness of spring and damping coefficient for damper in three orthogonal directions has been taken from experimental results.

The effectiveness of Tuned Mass Control Systems was estimated by comparing experimentally obtained response time-history and analytical data obtained for different mass ratios and corresponding optimum tuning parameters as well as different location of TMCS along the height of structure. The Tuned Mass Control Systems not necessarily should be located at top of the structure, since the similar effect should be obtained by TMCS locating at lower floor. The behavior of structure can be improved by providing more than one TMCS along the height of structure. The effect of TMCS is higher for TMCS installed at top of the structure. From the experimental and analytical study, it was observed that the behavior of TMCS is very complex than it is modeled using SAP2000 computer program. It has been demonstrated that by increasing of the mass ratio the effectiveness of TMCS will be increased. The location of TMCS has influence on the effectiveness of TMCS, if the TMCS install at top of the structure the effect is higher.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes, mathematical modeling of tuned mass damper, procedure for finding optimum parameters of tuned mass damper, basic concept of analysis of damper added structure and effect of Tuned mass damper and Friction damper to reduce response of wind or earthquake induced vibration.

Chapter 3

Passive Damping Techniques and Case Study

3.1 General

The usage of passive damping techniques have many advantages in order to dissipate energy generated due to seismic excitation and strong wind. Passive damping techniques are suppressing vibration without a presence of external energy to function. The usage of passive dampers is one of the damping techniques. Passive dampers are categorized into two groups depending upon the way they damp out vibration as shown in Figure 3.1.

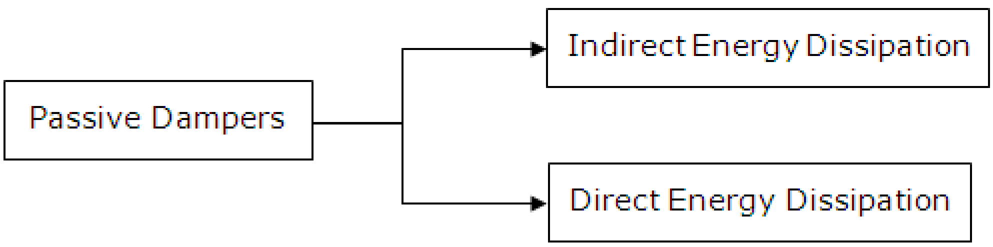


Figure 3.1: Categories of Passive Dampers

In indirect energy dissipation system energy is dissipated by secondary inertial system, for example Tuned Mass Damper and Tuned Liquid Damper. While in direct energy dissipation system energy is dissipated by shearing action of material used and flow of viscous fluid, for example Viscous damper, Viscoelastic damper, Friction damper and Metallic damper. Each type of passive dampers has their own characteristics in order to function as vibration reducer to concrete building during an earthquake.

3.2 Fluid Viscous Damper

The fluid viscous damper consists of components such as piston, cylinder, piston rod, piston head, control valve, accumulator housing and so on. At the end of fluid viscous damper, there is a spherical bearing which is a function to make sure that damper is not attached to the structure and allowing rotation in every direction.

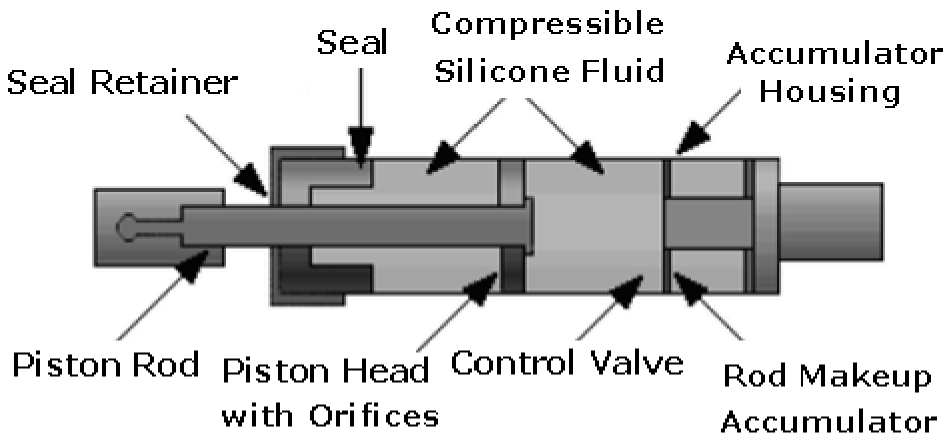


Figure 3.2: Diagram of Fluid Viscous Damper

The function of fluid viscous damper is basically to dissipate energy that is produced by lateral system of building structure. As there is movement generated by vibrations in the building, the piston is forced to move in and out of the cylinder at a high speed. This mechanism will force the fluid to flow through orifice in the piston head and produce friction as damping. The damping force will change some of the energy that generated from lateral system into heat energy. The diagram of fluid viscous damper is shown in Figure 3.2.

The fluid viscous damper can be applied into new building and existing structures. By adding fluid viscous damper, there is no change in natural period but damping will increase from 5% of critical to between 20% and 30%. The practical application of fluid viscous damper is summarized in Table 3.1.

Table 3.1: Application of Fluid Viscous Damping Devices

Sr.No.	*Structure and Location*	*Type of Damper*	*Year*	*Design Information*
1	Rich Stadium, Buffalo	12 No., 50kN, 460mm stroke	1993	Damper connect light poles to stadium wall to eliminate fatigue of anchor bolt
2	Petronas twin Towers, Malaysia	12 No., 10kN, 50mm stroke	1995	Part of mass damping systems in sky-bridge legs
3	Woodland Hotel (4-storey), USA	16 No., 450kN, 50mm stroke	1996	Seismic retrofit of 4-storey historic concrete structure

3.3 Viscoelastic Damper

It consists of steel plates and viscoelastic material that install as part of a diagonal brace. Copolymer or glassy substances are commonly used as viscoelastic material. The diagram of viscoelastic damper is presented in Figure 3.3. The viscoelastic damper dissipates the energy by shear deformation of viscoelastic material which is readily installed in the system. When structure vibrates, there will be relative motion between the steel angle and the center plate, thus energy dissipated. This type of damper is temperature sensitive so when it is used as external fitting some problem will occur.

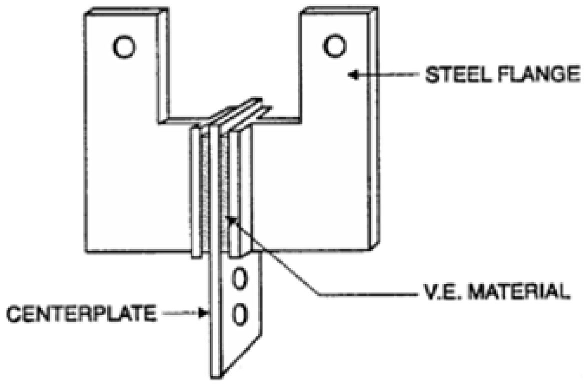


Figure 3.3: Diagram of Viscoelastic Damper

The application of viscoelastic damper is widely used on several countries; some of them are summarized in Table 3.2.

Table 3.2: Application of Viscoelastic Damping Devices

Sr.No	*Structure and Location*	*No. of Damper*	*Year*	*Design Information*
1	World Trade Center (110-storey), New York	20000 evenly distributed in 10-110 floors	1969	Damper in bottom chord of floor truss system
2	Columbia Sea First Building (73-storey), Washington	Total: 260	1982	New const., damper in parallel with diagonal bracing
3	Two Union Square Building (60-storey), Washington	Total: 16	1988	New const., damper in series with secondary column

3.4 Friction Damper

Friction damper consist of coated steel plates that being bolt together with slotted holes in them. Friction damper system is commonly used in form of ‘X’ where the damper is located at the middle of the ‘X’. The diagram of friction damper is illustrated in Figure 3.4. When the ground moves during earthquake, the members

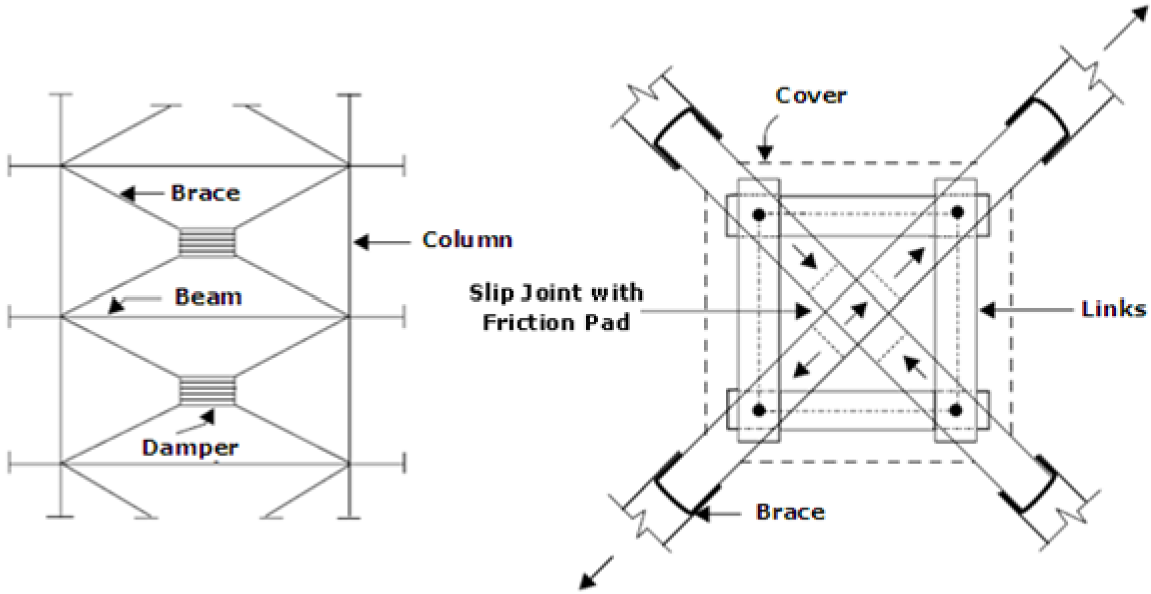


Figure 3.4: Diagram of Friction Damper

in friction damper such as steel plate will slide each other under damping force and dissipate the input energy by heat. By adding friction dampers to an existing structure the seismic load carrying capacity will be increased by means of reducing the demand of seismic resistance capacity. The practical application of friction damper in structures is summarized in Table 3.3 [13] [14].

Table 3.3: Application of Friction Damper

Sr.No	*Structure and Location*	*Type of Damper*	*Year*	*Design Information*
1	La Gardenia Towers (18-storey), Gurgaon, India	Total: 504, slip load: 375kN	1998	They are provided in steel bracing in concrete frame
2	Canadian Space Agency (4-storey), Colombia	Total: 58, slip load: 500kN	1993	They are provided in diagonal bracing
3	Federal Building (4-storey), Canada	Total: 30 slip load: 350kN	1995	They are provided for retrofit purpose

3.5 Friction Damper - La Gardenia Towers - A Case Study

This case study describes the use of friction damper in a La Gardenia Housing Complex, which is situated at Gurgaon, India. The complex is developed and owned by Unitech Limited of New Delhi.[9]

3.5.1 Structure

La gardenia housing complex consist of 7 towers of eighteen storeys with two levels of basement. The complex is spread over 11 acre of land in South city, Gurgaon and it is 8 km away from international airport, New Delhi. The complex presents a new concept of good living that borrows inspirit from Gardenia a very beautiful tropical flower. To live up to the theme La Gardenia, Unitech decided to use latest construction materials for the comfort and safety of its occupants.

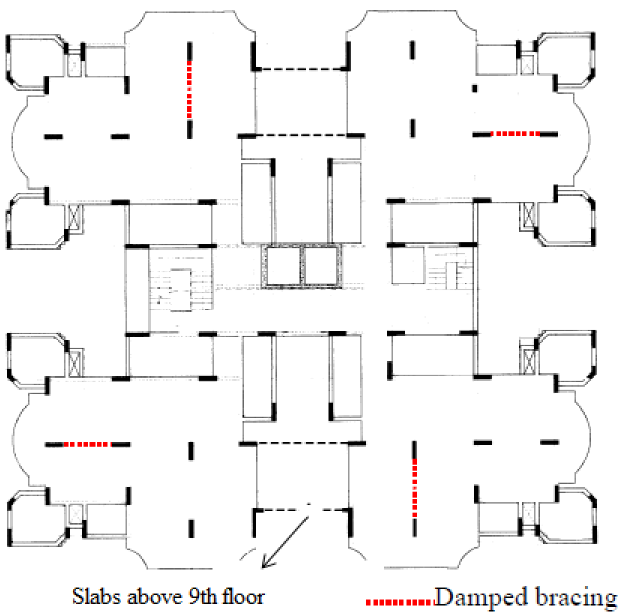


Figure 3.5: Typical Plan of La Gardenia Towers

A typical floor plan and three dimensional view of a 3-bed room apartment is shown in Figure 3.5 and Figure 3.6 respectively. The area of each apartment is about 200 m^2. There are four apartments at each floor, giving nearly a symmetrical plan. Between ground and ninth floor, the two apartments in north are connected to the two apartments in south with only at elevator lobby slab. At upper levels, two pairs of apartments are rigidly connected. The structure lacked torsional rigidity below ninth floor, when accidental eccentricity in mass and earthquake acting at an angle to the major axis are considered. The concrete shear wall was provided around central elevator from foundation to ninth floor with thick lobby slab, in combination with friction-damped bracing from ground to fourteenth floor.

Figure 3.6: Three dimensional view of La Gardenia Towers

3.5.2 Friction Damper

Pall friction dampers are simple and full proof in construction and inexpensive in cost. Basically, these consist of series of steel plates specially treated to develop most reliable friction. These plates are damped together with high strength steel bolts. Pall friction dampers are provided in steel bracing in concrete frames. The use of steel bracing eliminated the need of expensive shear walls and the use of friction dampers eliminated the need of dependence on member ductility. Friction damped bracing are located in partitions, around staircases or elevator shaft. Their use provided greater flexibility in space planning and it allows more open space for parking in the basement.

The use of state-of-art earthquake resistant design technology is the first application in India. Friction damped bracing do not carry any gravity load, these do not need to go down, through the basements to the foundation. Friction dampers are designed not to slip during service load and windstorms. The architects have exposed some friction dampers to view as they add to the aesthetic appearance. A total of 66 friction dampers were provided in one tower to extract sufficient energy to safeguard the structure and its components from damage [15].

During a major earthquake friction dampers slip at a predetermined optimum load before yielding occurs in other structural members and dissipate a major portion of the seismic energy. As per NEHRP guidelines friction dampers are designed for 130% MCE displacements and all bracing and connections are designed for 130% of damper slip load. Friction damper having 700 kN slip load is used for earthquake resistant design.

3.5.3 Summary

A friction damper is a novel structural solution for construction of eighteen storey La Gardenia Towers. By incorporating Pall friction dampers in steel bracing, the earthquake resistance and damage control potential of structure has dramatically increased. The introduction of supplemental damping provided by the friction dampers significantly reduced the lateral inertial forces and peak amplitude by 36% with respect to Bending Moment resisting Frame (BMF). By providing friction dampers, axial forces in column is reduced by 27% and 40% seismic energy is dissipated.

The use of Pall friction dampers has provided a practical and economical solution for the seismic control of structures. As the seismic forces exerted on the structure are significantly reduced, the system offers saving in construction materials. Friction damped structure has performed satisfactorily in the event of a major earthquake with possibly reduced damage to building and its components.

3.6 Metallic Damper

Metallic damper utilize the hysteric behavior of metals in the inelastic range. The resisting force of damper depends on nonlinear stress-strain characteristics of metallic material. Triangular plate and X-shape plate dampers are common type of metallic damper. The diagram of metallic damper is shown in Figure 3.7.

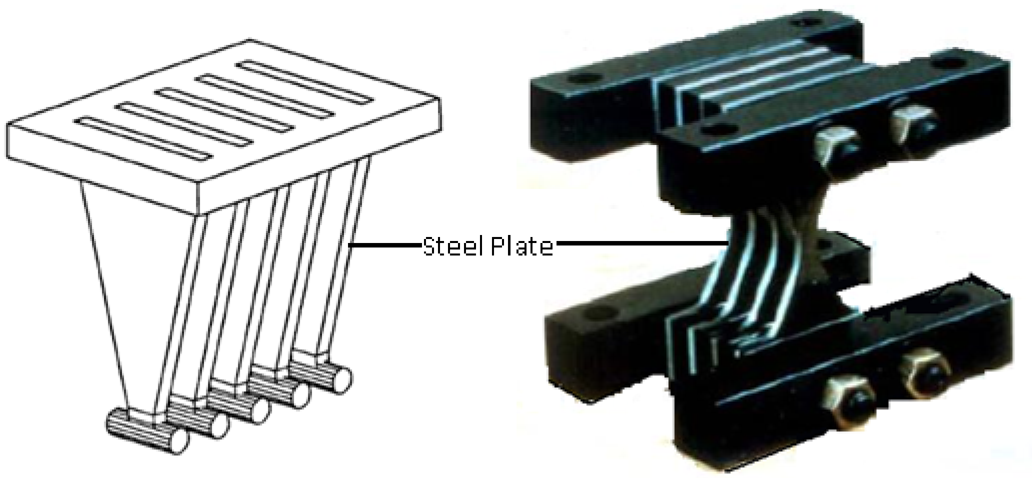


Figure 3.7: Diagram of Metallic Damper

The most desirable characteristic of these devices are their stable hysteric behavior, low-cycle fatigue property and long term reliability. Metallic dampers has installed within the frame bay between a chevron brace and the overlying beam. The damper primarily resists the horizontal force by flexural deformation of the individual plates. Beyond a certain level of force, the plates yield hence provides substantial energy dissipation. The practical application of metallic damper is summarized in Table 3.4.

Table 3.4: Application of Metallic Damper

Sr.No	*Structure and Location*	*No. of Damper*	*Year*	*Design Information*
1	Cardiology Building, Mexico	Total: 90	1990	Retrofit of Hospital: damaged in 1985 earthquake
2	Wells Fargo Bank (2-storey), California	Total:7 (ADAS type)	1992	Retrofit: damaged in 1989 Loma Prieta earthquake
3	Izazaga 38-40 (12-storey), Mexico	Total:200 (ADAS type)	1990	Retrofit: damaged in 1985,1986,1989 earthquakes

3.7 Tuned Mass Damper

A tuned mass damper is a relatively small device composed of mass, spring and a viscous damper which is installed near the top of the building. The purpose of providing a tuned mass damper is to reduce its response to dynamic loading. The natural frequency of the device is always chosen to match one of the natural frequencies of the vibrating system. The diagram of tuned mass damper is shown in Figure 3.8.

The principle of tuned mass damper is based on its natural frequency equal to one of the natural frequency of the structure. The tuned mass damper always oppose the motion of structure, hence structure vibratory motion is reduced. The advantage of tuned mass damper is that since no mechanical parts are involved hence little or no maintenance is required. The application of tuned mass damper is widely used in many tall buildings; some of them are summarized in Table 3.5.

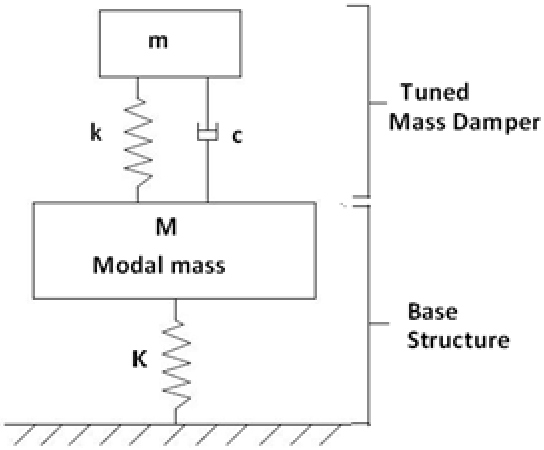


Figure 3.8: Diagram of Tuned Mass Damper

Table 3.5: Application of Tuned Mass Damper

Sr.No	*Structure and Location*	*No. of Damper*	*Year*	*Design Information*
1	John Hancock Tower (244m), Boston	2 No., 300 ton each	1977	Freq.- 0.14 Hz Damping ratio : 4%
2	Sydney Tower (305m), Australia	220 ton Pendulum type	1981	Freq.- 0.10 - 0,50 Hz Damping ratio : 2.9%
3	Higashiyama Sky Tower (134m), Japan	20 ton	1989	Freq.- 0.49 - 0.55 Hz

3.8 Tuned Mass Damper - Taipei-101 - A Case Study

This case study describes the use of tuned mass damper in one of the world's tallest building, Taipei 101. The work of tuned mass damper was carried out by one of the specialists in design and construction of damping systems for civil structures, Motioneering Inc. Guelph, Ontario Canada. The case study is based on an article written by T. Haskett, J. Robinson and J. Kottelenbery.

3.8.1 Structure

Taipei 101 is second world tallest building in the world which is situated at Hsin-Yi District, Taipei, Taiwan. The building consists of 101 floors with the height of 509m. Taipei 101 is a high-rise building that utilized steel super column and reinforced concrete until level 62. The building consists of commercials; offices and hotel space that cover the area of 412400 m^2. It is stiff structures which involve a total of 95000 ton of high strength steel and 70 MPa high strength concrete. The natural vibration period is 6.8 seconds. The height of Taipei is 508m i.e. from base to top of pinnacle.

Figure 3.9: Taipei 101 Tower

Figure 3.10: Spherical Tuned Mass Damper in Taipei 101

3.8.2 Tuned Mass Damper

Damper system was installed in Taipei 101 at two locations; one was installed in the building itself while the other one was installed at the pinnacle. The application of tuned mass damper in building was the first damper constructed by considering the architectural aspect. The building mass damper was exposed for occupant's observation and hanging up started from 87^{th} floor to 92^{nd} floor. Basically tuned mass dampers were installed to resist vibration induced by typhoon and earthquake, which is commonly occurred in Taiwan. The application of tuned mass damper in Taipei 101 will help to protect main structure of the building and avoid metal fatigue and failure.

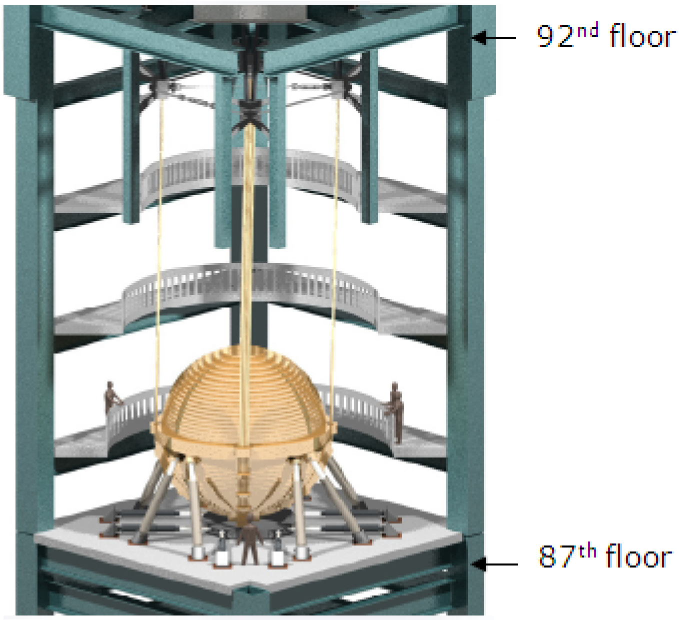


Figure 3.11: Installation of Tuned Mass Damper

Building tuned mass damper consist of 730 ton steel, which is 0.26% of total mass of building. The damper was made from stack of steel plate and was connected to the piston, which drives oil through small holes in order to dissipate vibrations. It was suspended by high-strength cable and it is supported by secondary systems called “snubber bearing”. It is a one kind of visco-elastic damping devices which function to engage TMD when amplitudes exceed 1m. The surface of damper was painted in gold. The pinnacle tuned mass damper mounted on a pinnacle. Pinnacle is a slender structure hence during strong winds there would be a number of vibration modes which may cause Vortex Induced Oscillation. It also occurs because of the structural discontinuity between the diameter of pinnacle and width of building [16].

3.8.3 Summary

A tuned mass damper is a state-of-art solution which mitigates the structures motion due to excessive wind induced vibrations. The engineers designed Taipei 101 in an ingenious way, combining old and modern technological achievements in the field of structural, wind and mechanical engineering i.e. Tuned mass dampers, aerodynamic optimization through wind tunnel testing, super column with high performance concrete, double deck elevators etc. In Taipei 101, the acceleration of top of tower is reduced to 35% (7.9 $\times 10^{-6} m/sec^2$ to 5 $\times 10^{-6} m/sec^2$) which indicates that the motion of structure has been reduced by installing TMD and the TMD proved to be a tremendous engineering achievement.

Instead of having a huge 730 ton steel ball which occupies valuable rentable space of four floors, one could design a system in which multiple tuned mass dampers will be hidden into the walls. They will be attached to the structural frame in a way that result in the similar reduction of the accelerations could be achieved as with a single tuned mass damper. In this way the constructibility and maintainability is increased and also if one of the dampers fails to operate, there can be some response from rest.

3.9 Tuned Liquid Damper

Tuned liquid damper is operated by attaching one or more multiple full liquid tanks to the structures. The basic principle of tuned liquid damper depends on the sloshing wave that develop at the free surface of fluid to dissipate a portion of the dynamic energy. The diagram of tuned liquid damper is presented in Figure 3.12.

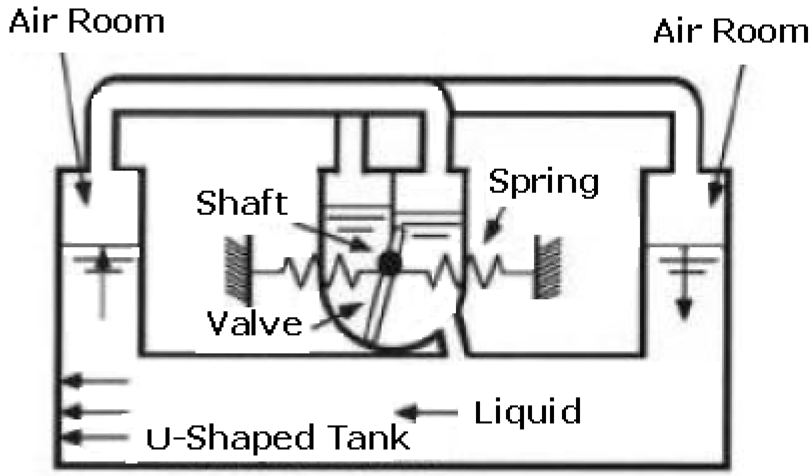


Figure 3.12: Diagram of Tuned Liquid Damper

Table 3.6: Application of Tuned Liquid Damper

Sr.No	*Structure and Location*	*No. of Damper*	*Year*	*Frequency*
1	Nagasaki Airport Tower (42m), Japan	25 No., 1 ton, Circular Sloshing Type	1987	1.07Hz
2	Gold Tower (136m), Japan	16 No., 9.6 ton, Rectangular Unidirectional type	1988	0.42Hz
3	TYG Building (159m), Japan	720 No., 18.2 ton, Double donut type	1992	0.53Hz

Tuned liquid damper is design for both shallow and deep water configuration. It is functioned by using the amplitude of fluid motion and wave braking pattern as additional damping. Energy has dissipated through viscous action of liquid configuration. The natural frequency of the system can be adjusted by the depth of liquid and the dimension of the container.The one of the simplest example of tuned liquid damper is a water tanks on the building without affecting functional use of water supply tanks. The application of tuned mass damper is widely used in many tall buildings; some of them are summarized in Table 3.6.

3.10 Summary

In this chapter, the main part is to describe the background of the structures involved and the types of damper used. The principle of working, design issues and practical application of various passive damping techniques are discussed in this chapter. The characteristics of the structures and the properties of the passive dampers are identified based on the researches that have been done. A case study of Taipei 101 and La Gardenia Towers are discussed for understanding of reducing earthquake and wind or typhoon induced vibration. Through the case study, the effectiveness of the installed damper will be evaluated according to the results that have already been identified.

Chapter 4

Analysis of Passive Dampers

4.1 General

Damping is a property of a structural system by which energy of the earthquake ground motion will be dissipated. Structural damping depends upon internal friction and absorption of energy by structural and non-structural element. Two types of damping in a structure is possible, i.e. inherent (material) damping and supplement damping.

The inherent damping depends upon type of material and property of material. In the supplement damping techniques, dampers or other energy absorbing devices are added to a structure with a purpose of increasing damping and reduce its response to dynamic loading. It is a technique to dissipate the input energy while keeping the structure undamaged.

4.2 Analysis of Tuned Mass Damper (TMD)

Tuned mass damper is amongst the oldest structural vibration control devices in existence. The concept of tuned mass damper was developed by Frahm in 1909. The Frahm tuned mass damper is shown in Figure 4.1. At present, a number of newer tall buildings in world are equipped with various types of tuned mass damper such as Taipei 101 at Taiwan, Petronas tower at Malaysia, John Hancock Tower at Boston and CN Tower at Canada etc.

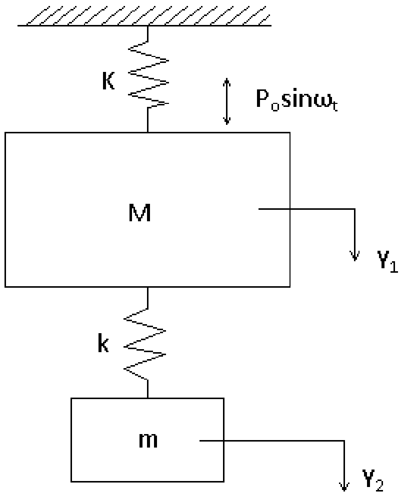


Figure 4.1: 1^{st} Frahm Tuned Mass Damper

A tuned mass damper is relatively small device, it composed of mass, spring and damping device which is installed near the top of a building with the purpose of reducing its response to dynamic loading. The natural frequency of the device is always chosen to match one of the natural frequencies of a vibratory system. The mass of tuned mass damper must be placed on smooth surface to minimize friction forces and allow its free movement.

4.2.1 Principles

From the laws of physics, F = ma and m=F/a. This means that when an external force is applied to a system, there has to be acceleration. In order to make the occupant comfort, tuned mass damper is placed in structure where the horizontal deflections due to wind or earthquake force felt the maximum. Therefore tuned mass damper is always placed near to top floor for effective response control of building. When the building begins to oscillate or sway, the tuned mass damper into motion by means of the spring. At this time building is forced right and the tuned mass damper moves simultaneously forces it to the left. The schematic representation of movement of tuned mass damper and building is shown in Figure 4.2.

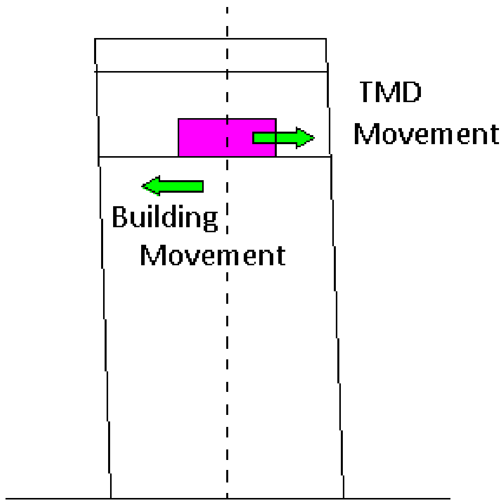


Figure 4.2: Movement of Tuned Mass Damper in structure

In order to increase the effectiveness of tuned mass damper to reduce maximum dynamic response of the main system, the frequency and amplitude of the tuned mass damper and structure should nearly match. So that every time lateral force push the building and tuned mass damper creates an equal and opposite push to the building for keeping the horizontal displacement at top zero. If their frequencies were significantly different, tuned mass damper would create push that is less than the push from the lateral force hence building motion would still be uncomfortable for the occupants. Similarly if their amplitude is significantly different, the push of tuned mass damper and building are not equal and opposite hence the building would still experience too much motion. The effectiveness of tuned mass damper is dependent on the mass ratio, frequency ratio and damping ratio.

4.2.2 Advantages and Disadvantages

Tuned mass dampers seem to be an effective way to add damping to a structure and to control its response to dynamic loads. Their impact on the design of the structure is minimum, since a structure with this type of device does not require special design procedures. They are easy to design and construct. Its construction requires only putting together mass, spring and dashpot at localized points of the structure, without the need of sophisticated hardware. Other advantages are as follow.

- They do not depend on an external power source for their operation and they require low maintenance.
- They do not interfere with the principal vertical and horizontal load paths of the structure.
- They can respond to small level of excitation, and their properties can be adjusted in the field.
- They can be considered in new design as well as in rehabilitation work and it is cost effective also.
- A single unit can be effective in reducing vibrations induced by earthquake or wind.

The **disadvantages** of tuned mass damper are as follow.

- A large mass is needed for their effectiveness or a large space is needed for their installation.
- The amount of travel of a tuned mass damper is by design large and therefore room is needed to accommodate for such mass travel.
- Their effectiveness depends on the accuracy of their tuning. Since the natural frequencies of a structure cannot be predicted with great accuracy, tuned mass

damper require field adjustment at the time of their installation and periodic adjustments thereafter.

- A tuned mass damper is only effective to control the response of a structure in one of its modes.
- The effectiveness of a tuned mass damper is constrained by the maximum weight, it is difficult to practically place on top of the structure.

4.2.3 Mathematical Modeling

The mathematical model of tuned mass damper attached with structure is shown in Figure 4.3 [8]. The equation of motion of a SDOF structure and tuned mass damper mechanism are as per eq. 4.1 and eq. 4.2.

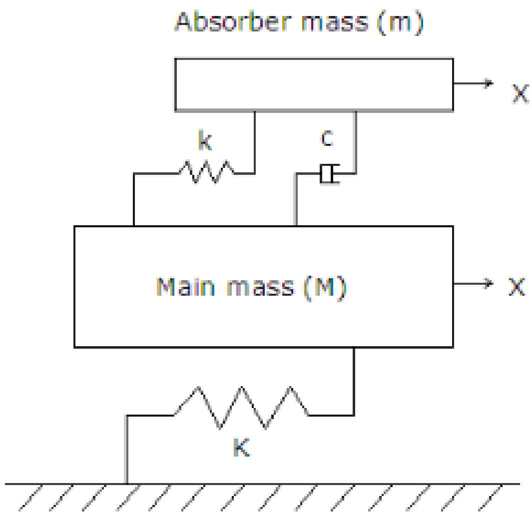


Figure 4.3: Mathematical Model of Tuned Mass Damper

$$M\ddot{X}_{(t)} + KX_{(t)} - \left[C\left\{\dot{x}_{(t)} - \ddot{X}_{(t)}\right\} + k\left\{x_{(t)} - X\left(t\right)\right\}\right] = P_{(t)} \tag{4.1}$$

$$m\ddot{x}_{(t)} + C\left\{\dot{x}_{(t)} - \dot{X}_{(t)} + k\left(x_{(t)} - X_{(t)}\right)\right\} = p_{(t)} \tag{4.2}$$

The expression for calculating optimum parameters for structure subjected to a harmonic excitation is as follow.

$$OptimumFrequency, f_{opt} = \frac{1}{1+\mu}\left(\sqrt{\frac{2-\mu}{2}}\right) \tag{4.3}$$

$$massratio, \mu = \frac{m}{M} \tag{4.4}$$

Damping ratio of TMD,

$$\xi_{opt} = \sqrt{\frac{3\mu}{8\left(1+\mu\right)}}\left(\sqrt{\frac{2}{2-\mu}}\right) \tag{4.5}$$

Stiffness of TMD can be evaluated from following expression

$$Frequencyratio, f_{opt} = \frac{\omega_{damper}}{\omega_{structure}} = \frac{\omega_{damper}}{\Omega} \tag{4.6}$$

$$\Omega = \frac{\sqrt{k_{opt}/m}}{f_{opt}} \tag{4.7}$$

$$k_{opt} = f_{opt}^{2}\Omega^{2}m \tag{4.8}$$

Damping ratio,

$$\xi_{opt} = \frac{C_{opt}}{C_c} = \frac{C_{opt}}{2m\omega_{\,dam}} \tag{4.9}$$

$$C_{opt} = 2m\omega_{dam}\xi_{opt} \tag{4.10}$$

$$C_{opt} = 2mf_{opt}\Omega\xi_{opt} \tag{4.11}$$

4.2.4 Procedure for Modeling of TMD in SAP2000

The tuned mass damper modeled as two plates which are shell element and two jointed non-linear link elements and linear spring elements. It consists of non-linear link damper elements for viscous damper and linear spring elements for the springs. The procedure of modeling of tuned mass damper is as follow [5].

Step-1 From **Define** menu commands select **Link/Support properties** to display the Non-linear properties dialog box.

Step-2 Click the **Add New Property** button to display the Link/Support type Data dialog box.

In this dialog box:

- Select **Damper** from the type drop-down box.
- Type the mass and weight property.
- Click the **U1** direction check box and also check box of Non-linear.
- Click the **Modify/Show for U1** button to display the Non-linear directional properties dialog box.
- In this diagonal box, type the values of Stiffness, Damping coefficient and damping Exponent for Nonlinear analysis Cases only.
- Click the **OK** button to return to the Non-linear Property Data dialog box.
- Same as check the **U2** and **U3** Direction check box and type the same values properties of damper as type in **U1** direction.
- Accept the rest of the default values.
- Click the **OK** button to return to the Non-linear Property dialog box.

Step-3 Now, again click the **Add New Property** button to display the Link/Support type Data dialog box.

In this dialog box:

- Select **Linear** from the type drop-down box.
- Type the mass and weight property.
- Click the U1, U2 and U3 direction check box.

- Click the **Modify/Show for all** button to display the directional properties dialog box.

- Type the stiffness values for **U1, U2** and **U3** direction.

- Click the **OK** button three times to exit all dialog boxes.

Step-4 From the **Draw** menu, select **Draw 2-Joint Link**.
In this dialog box select damper and draw a damper and spring element in geometry as per requirement.

4.3 Analysis of Friction Damper (FD)

The application of friction phenomena to structural engineering leads to the development of friction dissipaters, which are devices, intended to dissipate considerable amount of energy caused by earthquake loads and other dynamic excitations. Friction forces can be effectively used to dissipate energy and to mitigate damage to structures during seismic events. The irrecoverable work of friction damper is done by the tangential force required to slide a solid body across the surface of another one. The contacting surfaces are generally intended to remain dry during operation.

4.3.1 Principles

The scientific study of dry friction has a long history dating to the work of Da Vinchi, Amonntons and Coulombs. Basically, these consist of series of steel plates, which are specially treated to develop very reliable friction. These plates are clamped together and allowed to slip at a predetermined load. Model of friction damper is presented in Figure 4.4 [17].

The basic theory relies on the following hypotheses, which were initially referred from physical experimental involving planner sliding of rectilinear blocks.

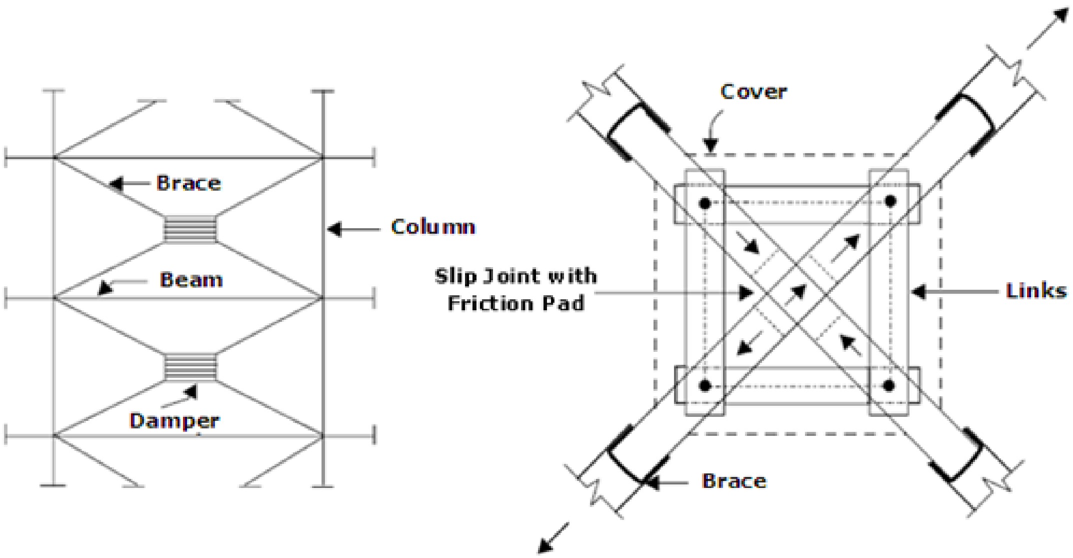


Figure 4.4: X-Braced Friction Damper

1. The maximum friction force that can be developed is independent of the apparent contact area, as long as the normal force keeps constants.

2. The maximum friction force that can be developed is proportional to the total normal force acting across the interface, provided that the pressure is uniformly distributed.

3. The maximum static friction force is greater than the kinematic friction force.

4. For the case of sliding with low relative velocity, the friction force is independent of that velocity.

5. Ordinary changes in temperature do not significantly affect the coefficient of friction.

As a result of these assumptions, at the instant of impending slippage of during sliding itself,

$$F = \mu N \tag{4.12}$$

Where 'F' and 'N' represent the frictional and normal forces, respectively and 'μ' is the coefficient of friction. The coefficient of friction is defined as the ratio of the absolute value of the maximum static frictional force to the magnitude of the normal force between the two surfaces. The value of 'μ' at any instant depends not only upon the selection of sliding materials, but also on the present condition of two sliding interface. In mathematical form,

$$\mu = Fmax/N \tag{4.13}$$

This equation expressed the Coulomb law of dry friction. Since it is frequently observed that the coefficient of friction is somewhat higher when slippage is imminent than during sliding, separate static and dynamic coefficients are often introduced. In either case, the frictional force 'F' acts tangentially within the interfacial plane in the direction opposing the motion or impeding it.

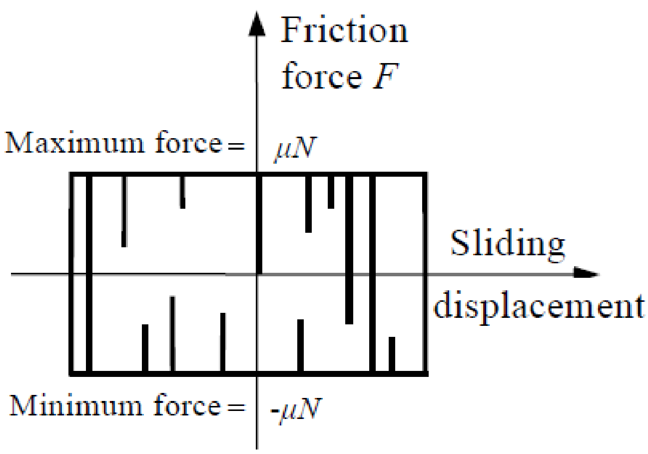


Figure 4.5: Rectangular Hysteric Loop of Friction Damper

The theoretical hysteresis loops of dry friction are shown in Figure 4.5. The maximum and minimum values of the friction force are $\pm\mu$N. The concept of Coulomb friction provides the basis for most of research concerning friction dampers. However, it should be emphasized that frictional processes are seldom..

4.3.2 Advantages and Disadvantages

Pall friction dampers are simple and full proof in construction. They offer reliable and repeatable performance at low cost. It reduces the problems created by dependence on structural ductility by dissipating most of the seismic energy mechanically, independent of the primary structure. This makes the design of damage-free structures economically possible. Unlike traditional concrete shear walls, earthquake friction dampers do not have to be located continuously one on top of the other which allows for greater space-planning flexibility. Other **advantages** are as follow [18].

- By adding friction damper seismic load carrying capacity of structure will be increased by means of reducing demand of seismic resistance.

- Their behavior is not affected seriously by amplitude and frequency contents of the number of cycles of the driving force.

- They are not active during service loads and wind. Hence, no possibility of failure due to fatigue before an earthquake.

- They do not require any repair or replacement before and after earthquake.

- They are easily adaptable to any site condition and it can be available for all types of bracing, including tension cross bracing, and expansion joints. It can be welded or bolted with structure.

The **disadvantages** of friction damper are as follow.

- Due to the frequent and sudden change in the sticking-sliding conditions, high frequencies can be introduced in the response.

- Durability is also a controversial issue, mostly due to the high sensitivity of the coefficient of friction to the conditions in the sliding surfaces.

4.3.3 Types of Friction Damper

A wide variety of friction dampers has been developed and installed in building structures. Friction dampers can provide mechanism for dissipation of large amount of energy and they have good performance characteristics and their behavior is less affected by the load frequency, number of load cycles or changes in temperature. Friction damper exhibit rigid-plastic behavior and force response is modeled by Coulomb friction.

1. X-Braced Damper

It was proposed by Pall et al.(1982). It is shown in Figure 4.4. In this type of damper, the braces in a moment resisting frame incorporated frictional devices. When load is applied to this damper, the tension brace induces slippage at the friction joint. Consequently, the four links force compression brace to slip. Energy is dissipated in both braces even though they were designed to be effective in tension only. However, this is only valid if the slippage of the device is sufficient to completely straighten any buckled braces [19].

2. Bracing-damper systems

It was proposed by Filiatrault and Cherry (1987). Bracing-damper system, which is a more detailed model for the device in which each member of the bracing-damper system is represented by element reflecting its individual axial and bending characteristics. The structural braces are assumed to yield in tension, but buckle elastically in compression. Bending stiffness is included to maintain stability of the damper mechanism.

3. Uniaxial friction damper

Uniaxial friction damper which is shown in Figure 4.6 is manufactured by Sumitomo Metal Industries Ltd., utilizes a slightly more sophisticated design. It is also known as Sumitomo friction damper. The pre-compressed internal spring exerts a force that is converted through the action of inner and outer Wedges into a normal force on the friction pads. These copper alloy friction pads contain graphite plug inserts, which provides dry lubrication. This helps to maintain a consistent coefficient of friction between the pads and the inner surface of the stainless steel casing. The effect of loading frequency and amplitude, number of cycles or ambient temperature on damper response was reported as insignificant. The reduction in displacements however, depend on the input ground motion because friction dampers are not activated and do not dissipate energy for forces smaller than threshold.

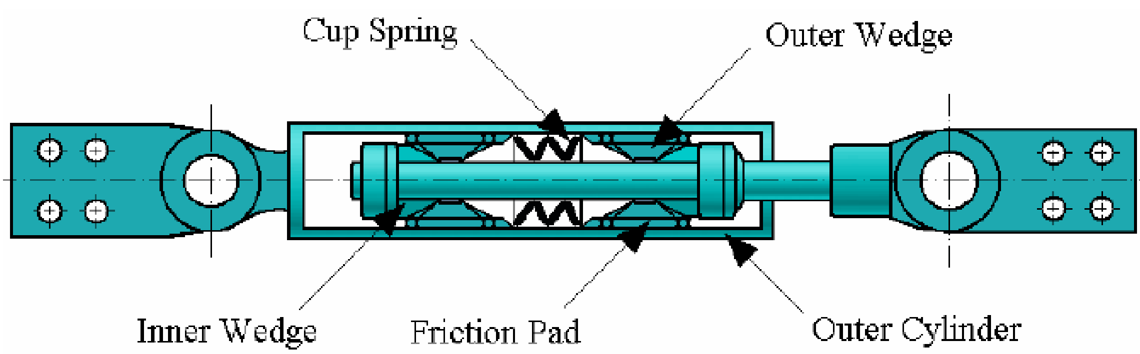


Figure 4.6: Uniaxial Friction Damper

4. Energy Dissipating Restraint

It was manufactured by Fluor Daniel Inc., which is shown in Figure 4.7. The design of this friction damper is superficially similar to the Sumitomo damper, since this device also includes an internal spring and wedges encased in a steel cylinder. However, there are some novel features of the EDR that produce very different response characteristics. The EDR utilizes steel and bronze friction wedges to convert the axial spring force into normal pressure acting outward on the cylinder wall. Therefore, the frictional surface is created by the interface between the bronze wedges and the steel cylinder. Internal stops are installed within the cylinder in order to create the tension and compression gaps. The length of the internal spring can be changed during operation to provide a variable friction slip force.

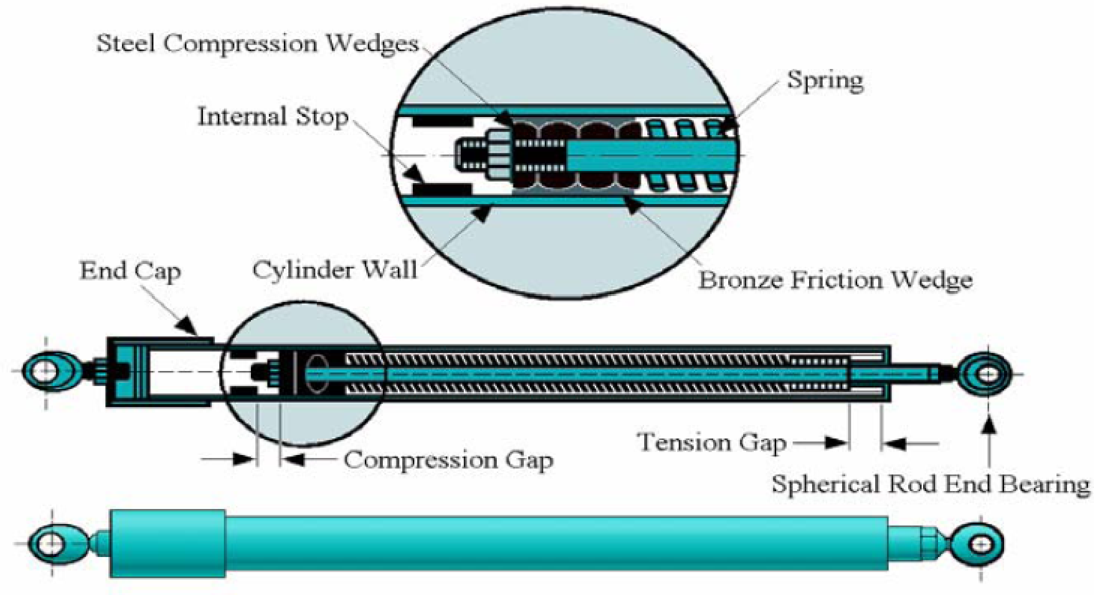


Figure 4.7: Energy Dissipating Restraint

5. Slotted Bolted Damper

It was proposed by Fitzgerald (1989) and it can be seen in Figure 4.8. A slotted-bolted damper is a wide concept that refers to a bolted connection where the slots in the main connecting plate, in which the bolts are seated. In addition a Belleville washer is placed under the nut. Upon tightening of the bolts, the main plate is compressed directly between the brass insert plates. The holes in such plates and in the steel plates are of conventional size. When the tensile or compressive force applied to the connection exceeds the friction forces developed between the frictional surfaces, the main plate slides relatively with respect to the brass insert plates. This process is repeated with sliding in the opposite direction upon reversal of the direction of the frictional force. Energy is dissipated by means of friction between the sliding surfaces. It can also be used in steel structures [20].

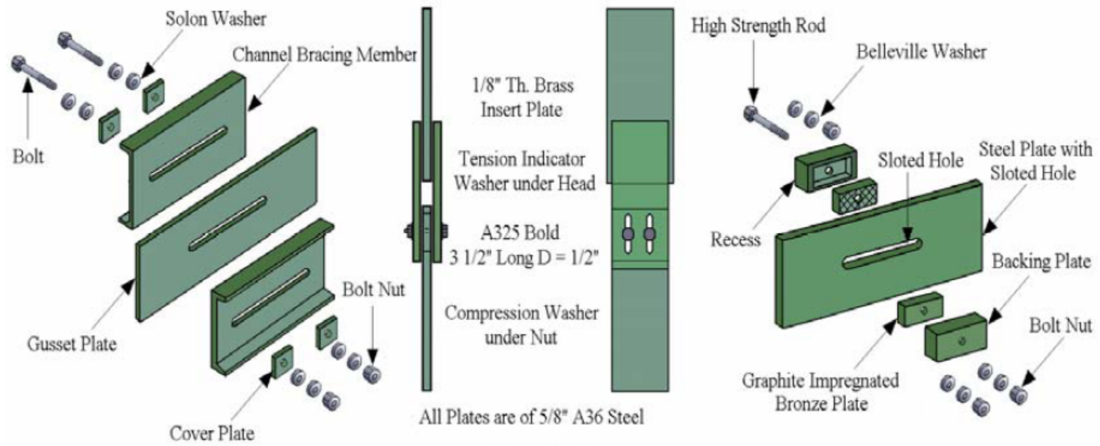


Figure 4.8: Slotted Bolted Damper

4.3.4 Design Criteria

The quasi-static design procedure given in the Indian Standards and building codes in other countries are ductility based and do not explicitly apply to friction-damped buildings. However, the building codes in the U.S, Canada and some other countries allow the use of friction-dampers for seismic control of buildings. It requires that non-linear analysis must demonstrate that the building so equipped will perform equally well in seismic events. In the past few years, several guidelines on the analysis and design procedure of passive energy dissipation devices have been developed in the U.S. The latest and most comprehensive document is the "NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273 / 274 , issued in October 1997". These guidelines and provisions of Indian Standards IS: 1893 'Criteria for Earthquake Resistant Design of Structures', served as basis for the analysis and design of the structures [21] [22].

The guidelines require that the structure with energy dissipating devices be evaluated for response to two levels of ground shaking i.e. design basis earthquake (DBE) and maximum considered earthquake (MCE). The DBE is an event with 10% probability of exceedable in 50 years, while the MCE represents a severe ground motion of probability of 2% in 50 years. Under the DBE, the structure is evaluated to ensure that the strength demands on structural elements do not exceed their capacities and that the drift in the structure is within the tolerable limits. For the MCE, the structure is evaluated to determine the maximum displacement requirement. It is presumed that with proper ductile detailing, the structure will have sufficient reserve to resist any overstress conditions that occur during the MCE and collapse is avoided. Nonlinear time-history analysis is required for both the DBE and the MCE. The maximum response of at least three earthquake records should be used for design.

NEHRP guidelines require that friction-dampers are designed for 130% MCE displacements and all bracing and connections are designed for 130% of damper slip load. Variation in slip load from design value should not be more that ±15% [23].

4.3.5 Slip load

The friction dampers are designed not to slip during wind. During a major earthquake, they slip prior to yielding of structural members. In general, the lower bound is about 130% of wind shear and the upper bound is 75% of the shear at which the members will yield. As seen in the Figure 4.9 for Response versus Slip Load, if the slip load is very low or very high, the response is very high [15] [24].

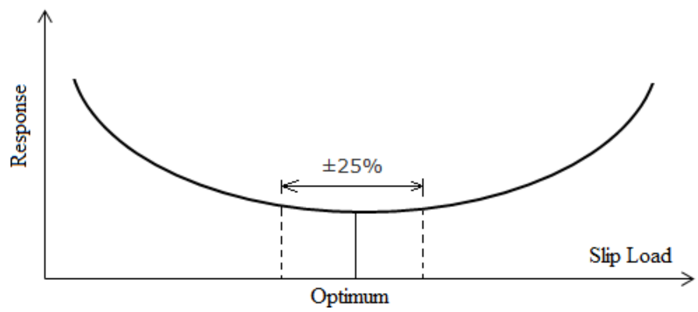


Figure 4.9: Response Vs Slip load

From past studies, it was seen that the slip load of the friction damper is the principal variable with the appropriate selection of which it is possible to tune the response of structure to an optimum value. Optimum slip load gives minimum response. Selection of slip load should also ensure that after an earthquake, the building returns to its near original alignment under the spring action of an elastic structure. Studies have also shown that variations up to ±25% of the optimum slip load do not affect the response significantly. Therefore, small variations in slip load (8-10%) over life of the building do not warrant any adjustments or replacement of friction damper.

4.3.6 Procedure for Modeling of FD in SAP2000

The computer modeling of Pall friction damper is very simple. Since the hysteric loop of the friction damper is perfectly rectangular, similar to perfectly elasto-plastic material. The friction damper can be modeled as fictitious plasticity element having yield strength equal to slip load.

The procedure of modeling of friction damper is as follow [25] [4].

Step-1 From **Define** menu commands select **Link/Support properties** to display the Non-linear properties dialog box.
Step-2 Click the **Add New Property** button to display the **Link/Support type Data** dialog box.
In this dialog box:
Select **Plastic (Wen)** from the type drop-down box.
Step-3 For **diagonal friction damper** following procedure is followed.

- Type the mass and weight property.
- Click the **U1** direction check box and also check box of Non-linear.
- Click the **Modify/Show for U1** button to display the Non-linear directional

properties dialog box.

- Type the value of yield strength equal to slip load.
- Type the effective stiffness equal to **1000** times slip load for linear as well as nonlinear case.
- Effective damping value remains as zero, because in friction damper there is nothing any viscous damping.
- Type **0.0001** in the post yield stiffness ratio edit box.
- Type **10** in the yielding exponent edit box.
- Accept the rest of the default values.
- Click the **OK** button three times to exit all dialog boxes.

Step-4 For **chevron friction damper** following procedure is followed.

- Type the mass and weight property.
- For bay **along X-axis** same properties is defined as diagonal friction damper but it is for **U2** direction.
- Also click the **U3** direction check box, do not check box of nonlinear.
- Type some value say **500** for orthogonal stiffness and damping value as remain as zero for **U3** direction.
- For bay **along Y-axis** same properties is defined as diagonal friction damper but it is for **U3** direction.
- Also click the **U2** direction check box, do not check box of nonlinear.
- Type some value say **500** for orthogonal stiffness and damping value as remain as zero for **U2** direction.

- Do not put check mark in **U1** direction dialog box for bay along X-direction as well as Y-direction.

- Accept the rest of the default values.

- Click the **OK** button three times to exit all dialog boxes.

Step-5 From the **Draw** menu, select **Draw 2-Joint Link**.
In this dialog box select property and draw a damper element in geometry as per requirement.

4.4 Method of Analysis of Structure using Passive Dampers

Today there are number of methods available for building structures subjected to seismic loading, four of them are linear static, linear dynamic nonlinear static and nonlinear dynamic. Since the passive energy dissipation devices are relatively new in the structural design, there are no clear design procedures for their implementation to building structures [26].

4.4.1 Static Analysis Procedures

A static analysis is a quick and easy way to obtain an approximate response of a structure. Generally this method determines the distribution of the earthquake base shear force in given direction through the height of structure. According to linear static procedures, static lateral force is applied to structure to obtain design displacements and forces. This method is based on two important assumptions.

a. It is implied that an adequate measure of design actions can be obtained using static analysis; even though seismic response is dynamic.

b. It is implied that an adequate measure of design actions can be obtained using a linearly-elastic model, even though nonlinear response to strong ground shaking may be anticipated.

Static procedures are only permitted if it can be demonstrated that the framing system exclusive of the energy dissipation devices remains essentially linearly elastic for the level of earthquake demand of interest after the effect of added damping are considered. Further the effective damping offered by energy dissipation devices shall not exceeds 30% of critical in the fundamental mode. The evaluation of nonlinear deformation should be carried out using nonlinear procedures that explicitly account for nonlinear deformations in deformed components. As an option, the guidelines permit evaluation to be carried out using linear procedures. In linear procedures there is direct relation between internal forces and internal deformation for all types of loading. Hence, when using linear procedures, it is simpler to express acceptability in terms of internal forces rather than internal deformation [27].

4.4.2 Dynamic Analysis Procedures

Similarly as the linear static also the linear dynamic procedure uses the same linearly elastic structural model. The linear dynamic procedure provides great insight into structural response. It is similar to linear static procedure, it does not explicitly account for effect of nonlinear response. In case of nonlinear dynamic procedure, the nonlinear load deformation behavior of individual components and element is modeled directly in the mathematical model. There are two types of dynamic analysis procedures to choose from structures subjected to seismic ground motions, which are the response spectrum and time history analysis [21].

4.4.2.1 Response Spectrum Analysis

The response spectrum is defined as a graphical relationship of the maximum acceleration response of a SDOF elastic system for various values of natural period. The response spectrum requires for dynamic analysis of structure to establish modal frequencies and mode shapes. Using standard mathematical procedures and a response spectrum corresponding to the damping in the structure, the modal frequencies and mode shapes are used to establish spectral demands. The spectral demands are then used to calculate displacements, forces, storey shears and base reactions for each mode of response. Consequently, all these results are combined by using established rule to calculate total response quantities.

Response Spectrum Data

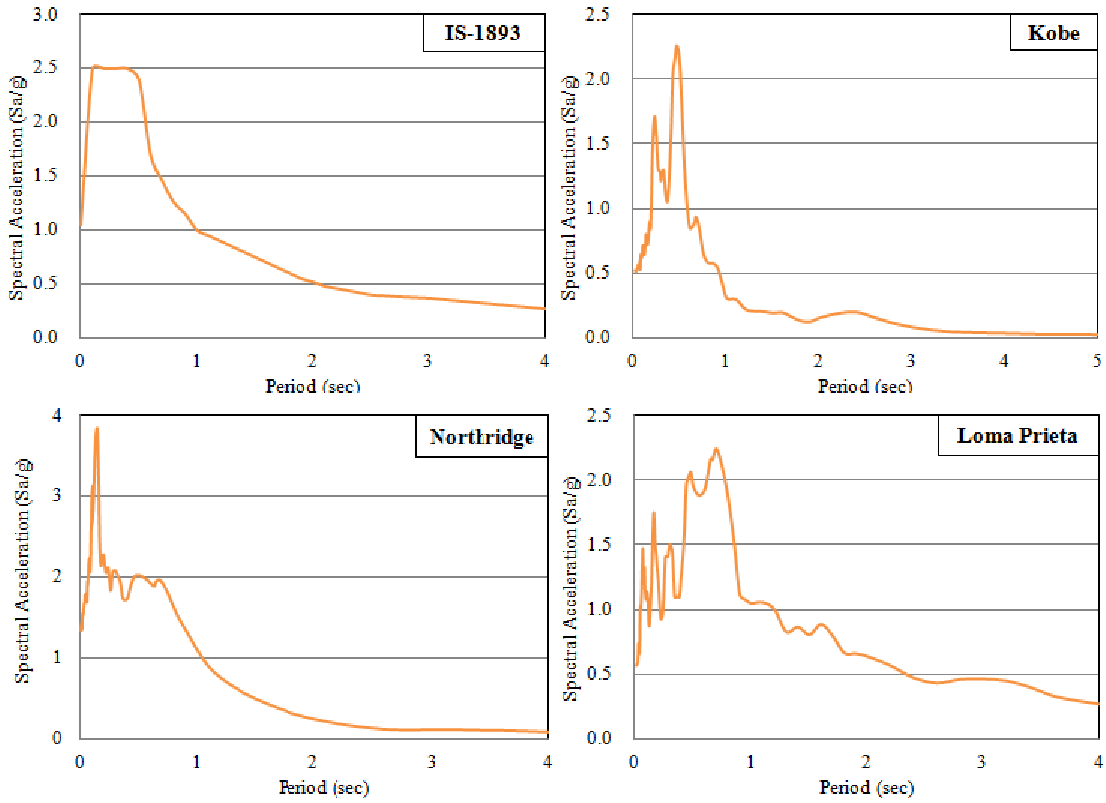


Figure 4.10: Response Spectrums for Various Earthquakes

In earthquake engineering, the seismic input for design of structures is commonly defined in form of response spectrum curves. This input model of earthquake can be directly used in the response spectrum analysis. The earthquake records, which are selected for this study are Kobe, Northridge, Loma Prieta and IS1893 (part-I):2002 response spectra. The data of response spectrum for various earthquakes are graphically represented in Figure 4.10.

4.4.2.1 Time History Analysis

For buildings installed with nonlinear devices, such as tuned mass damper or friction dampers, the computation of the seismic structural response must be performed by time history analysis. The time history analysis determines the response of a structure due to forces, displacements, velocities or accelerations those varies with time. There are two types method for time history analysis, first is direct integration method and second is modal superposition method. Both methods are suitable for linear time history as well as nonlinear time history analysis. The step by step solution of equation of motion, which is generally described as,

$$M\ddot{U} + C\dot{U} + KU = F_{(t)} \tag{4.14}$$

Where 'M', 'C' and 'K' are mass, damping and stiffness matrices respectively, '$\ddot{U}$', '$\dot{U}$' and 'U' are acceleration, velocity and displacement vectors respectively and '$F_{(t)}$' is the vector of applied forces which may vary with time [28].

In general, earthquakes have different properties such as peak acceleration, peak velocity, peak displacement, duration of strong motion and ranges of dominant frequencies; hence they have different influence on the structures. In order to ensure that the chosen mitigation procedure is effective under different types of excitations, four well-known earthquake records are used in this study. The time history data was taken from "Pacific Earthquake Engineering Research Institute" (PEER).

Time History Records

The dynamic behavior of any structural system is governed not only by the nature of applied loads but also by dynamic characteristics of the system itself. Because earthquake ground motions are erratic in nature and they cannot be represented mathematically by a single continuous function for which closed form solution can be obtained. Acceleration time history for various earthquakes is plotted in Figure 4.11.

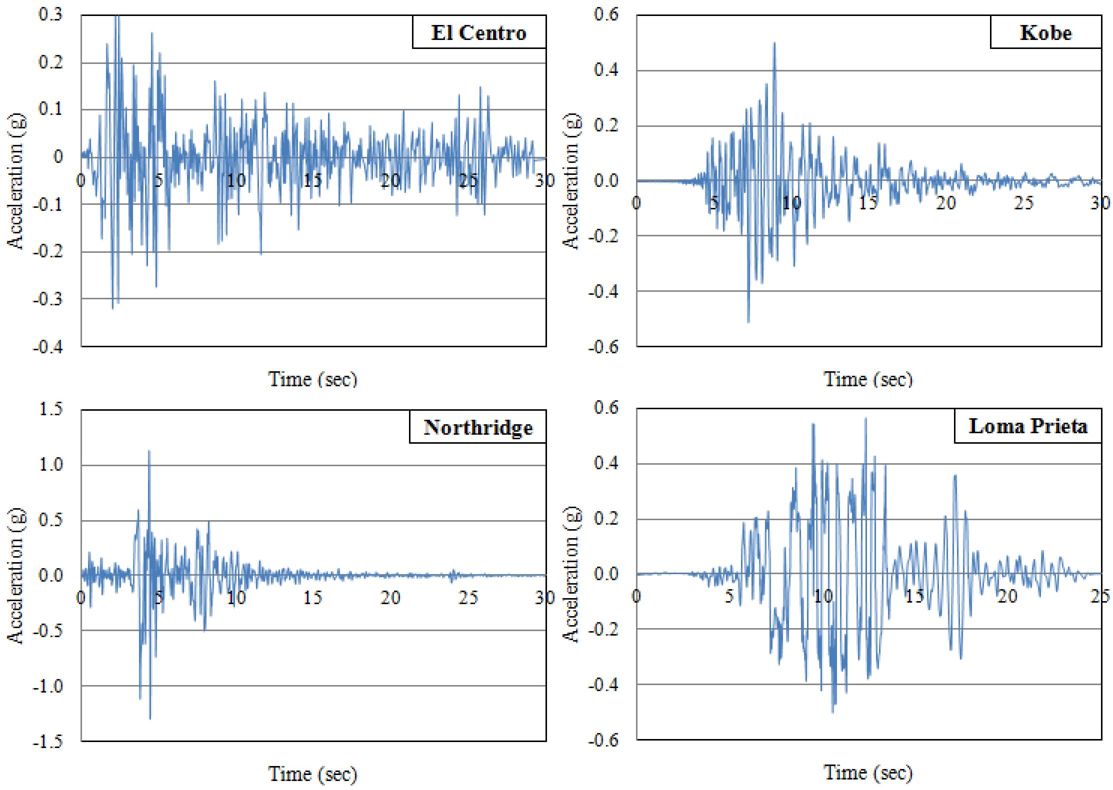


Figure 4.11: Acceleration Time History for Various Earthquakes

The earthquake time history records, which are selected for this study to investigate the dynamic response of models are summarized in Table 4.1 with peak acceleration, velocity, displacement and damping. These all are applied for the first 25 seconds of their duration during which the strong motion take place.

Table 4.1: Time History Data for Various Earthquakes

Sr. No	*Earthquake*	*Year*	*PGA* (cm/sec^2)	*PGV* *(cm/sec)*	*PGD* *(cm)*	*Damping*
1	El Centro	1940	0.298	43.8	18.3	0.05
2	Kobe	1995	0.509	37.3	9.52	0.05
3	Loma Prieta	1989	0.563	94.8	41.18	0.05
4	Northridge	1994	1.285	103.9	23.8	0.05

4.5 Summary

This chapter presents the analysis of tuned mass damper and friction damper. A mathematical model of tuned mass damper is presented. For friction damper design criteria and modeling is presented as per Avatar Pall. A procedure for computer modeling of tuned mass damper and friction damper is discussed in detail with step by step procedure. Various analysis procedures for analysis of passive dampers are discussed. For analysis of structure with passive energy dissipation devices various earthquake data was selected in from of response spectra and acceleration time history.

Chapter 5

Analysis of 5-Storey Structure

The chapter deals with the finite element analysis of 5-storey RC framed structure. As mentioned earlier SAP2000 software was used for modeling and analysis of this structure. Three kind of RC buildings were taken for analysis purpose; uncontrolled structure, structure fitted with tuned mass damper and friction dampers. Also the influence of various dampers on overall analysis of structure is finding out.

5.1 Structural Data

The first structural type investigated in this study was represented by 5-storey RC framed structure. Two different types of dampers namely Tuned Mass Damper and diagonal Friction Damper were considered for analysis of 5-storey structure. Seismic analysis of these frame structure was carried out with one type of damping system at a time. The undamped structure was also analyzed in order to compare results.

5.1.1 Structural Configuration

The structural layout of 5-storey RC framed structure considered for the modeling and analysis is shown in Figure 5.1. The plan of building considered is symmetrical, to avoid the case of non-proportional damping in a structure, which helps damping devices to be distributed symmetrically in a building.

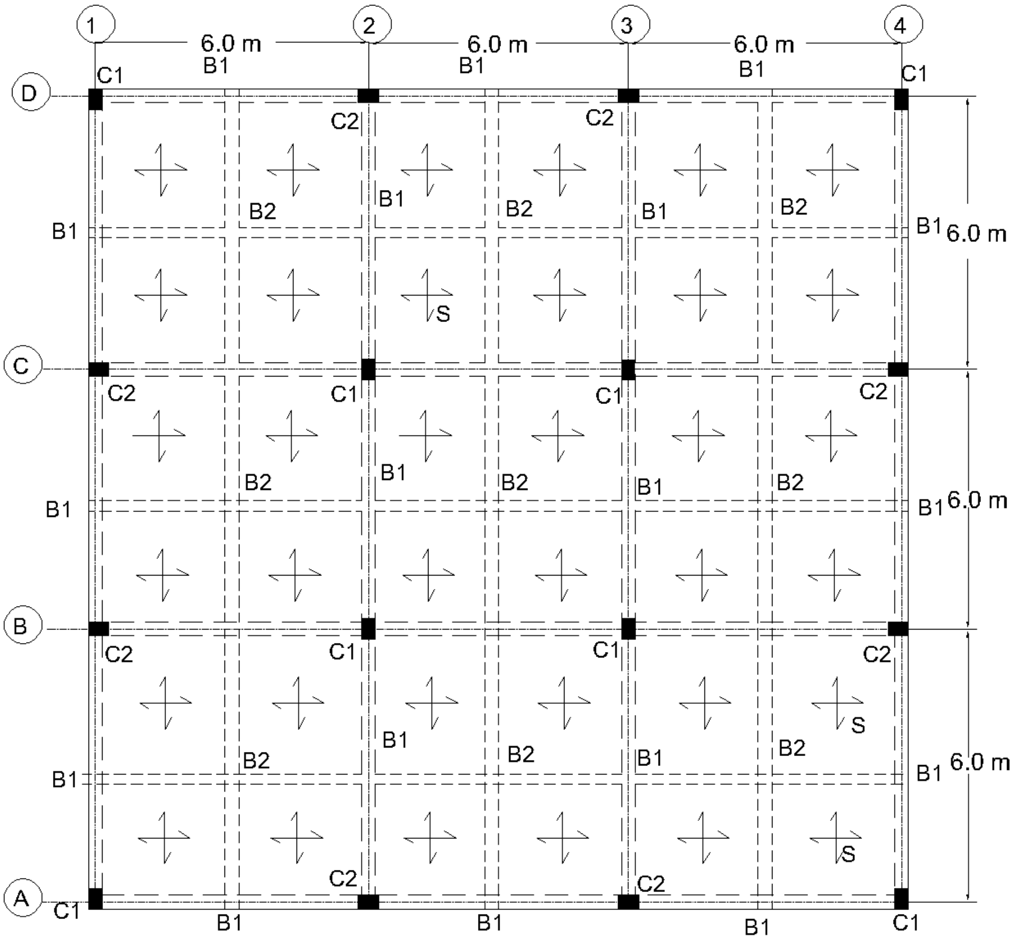


Figure 5.1: Plan of 5-Storey RC Framed Structure

5.1.2 Properties of Building

The material and loading data are enlisted below.

Material data

Concrete Grade = M30

Elasticity of concrete = 27386.1 N/mm^2

Poisson ratio for concrete = 0.2

Unit weight of concrete = 25 kN/m^3

Unit weight of brick = 20 kN/m^3

Steel Grade = Fe415

Loading data

a. Dead Load

Thickness of slab = 125 mm

Self-weight of slab = 0.125 × 25 × 1 = 3.125 kN/m^2

Floor finish load = 1 kN/m^2

Wall load (150mm thick) = 0.150 × 20 × (3 – 0.50) = 7.5 kN/m

b. Live Load

Live load on floor = 3 kN/m^2

Live load on roof = 1.5 kN/m^2

c. Earthquake Load [29]

Earthquake zone = V

Zone factor, Z = 0.36

Importance factor, I = 1

Response Reduction factor, R = 5 (SMRF)

Soil type = Medium soil (Type-II)

d. Wind Load [30]

Wind zone = V

Basic wind speed, Vb = 50 m/sec

Probability factor, k1 = 1.0

Terrain category, height and structure size factor, k2 = varies with height

Topography factor, k3 = 1.0

Sectional Properties

Approximate sizes of slab, beam and column are summarized in Table 5.1.

Table 5.1: Approximate Sizes of Member for 5-Storey Building

Sr. No	*Element*	*Notation*	*Size (mm)*
1	Main Beam	B1	300 × 500
2	Secondary Beam	B2	230 × 450
3	Column	C1	300 × 450
4	Column	C2	450 × 300
5	Slab	S	125

Boundary condition

The earthquake events used in this study are recorded as time history accelerations and response spectra in the horizontal plane. They are applied in X-direction as well as Y-direction for time history analysis and response spectrum analysis. For analysis of building fixed support is considered.

5.1.3 Properties and Location of Tuned Mass Damper

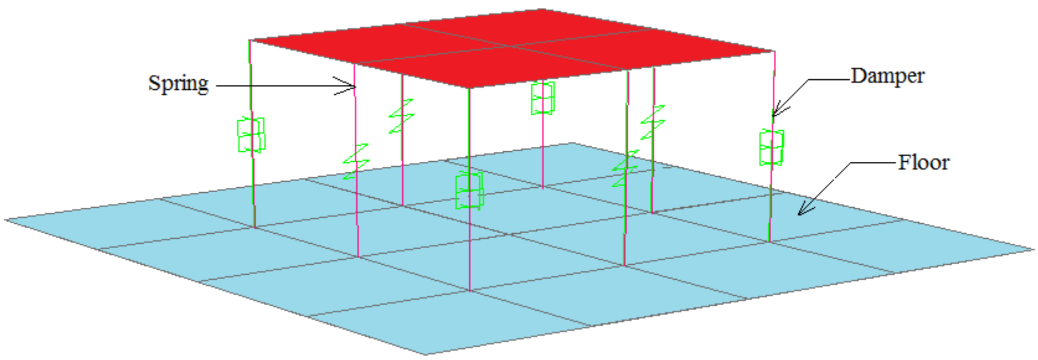


Figure 5.2: Schematic representation of Tuned Mass Damper

For achieving maximum response of tuned mass damper, it is placed at top of the structure. Details of tuned mass damper located at top of the structural model are illustrated in Figure 5.2.

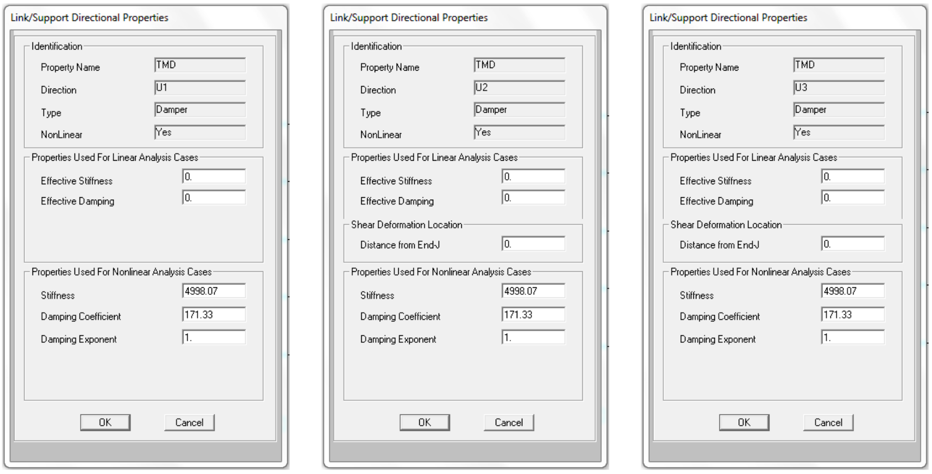


Figure 5.3: Properties of Damper element of TMD

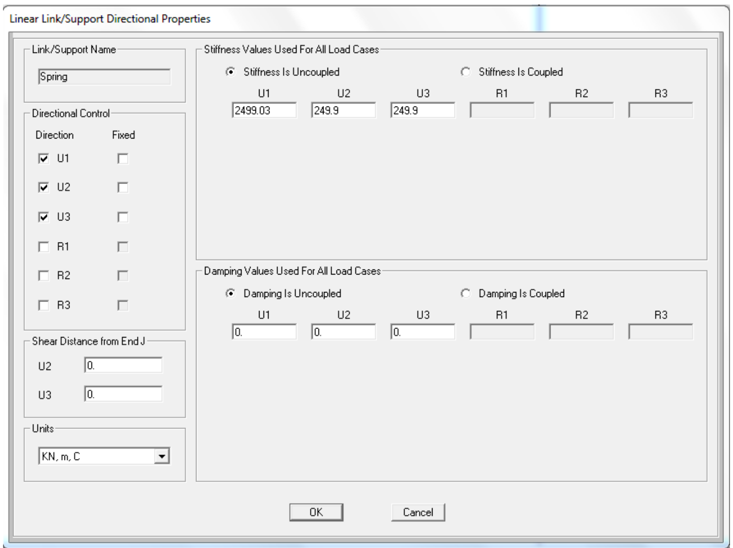


Figure 5.4: Properties of spring element of TMD

Damping characteristics of tuned mass damper are based on the Maxwell model for achieving viscoelastic behavior. For achieving nonlinear behavior, dampers are connected in a series with a spring. The screen shorts of defining properties of tuned mass damper are presented in Figure 5.3 and Figure 5.4. Characteristics of tuned mass damper, which consist of nonlinear damper elements and spring elements, are shown in Table 5.2. Assume mass of damper as 2% of total structural mass and from mass other parameters are calculated. The calculation of parameters of tuned mass damper are shown in Appendix-A. These parameters are used for modeling of tuned mass damper in SAP2000.

Table 5.2: Defining Parameters for Tuned Mass Damper

Link Property	*Damper*	*Spring*
Mass (Ton)	9.26	-
Stiffness (kN/m)	X-dir. = 4998.07	X-dir. = 249.9
	Y-dir. = 4998.07	Y-dir. = 249.9
	Z-dir. = 4998.07	Z-dir. = 2499.03
Damping Coefficient (kN.sec/m)	X-dir. = 171.33	
	Y-dir. = 171.33	
	Z-dir. = 171.33	

5.1.4 Properties and Location of Friction Damper

Friction dampers are placed at 1^{st}, 3^{rd} and 5^{th} storey in central bay on all outer periphery of building. A detail of diagonal friction damper located in 5-storey RC framed structure is shown in Figure 5.5. A total of 12 friction damper is used for reducing the response of the structure.

The modeling of Pall friction damper is very simple. The friction dampers can be modeled as fictitious plasticity element having yield force equal to slip load. The plasticity model is based on the hysteretic behavior proposed by Wen (1976). Since the hysteric loop of the damper is similar to the rectangular loop of an ideal elasto-plastic material. The yielding exponent controls the sharpness of the transition from the initial stiffness to the yielded stiffness. Characteristics of friction damper are shown in Table 5.3. These parameters are used for modeling of friction damper in SAP2000.

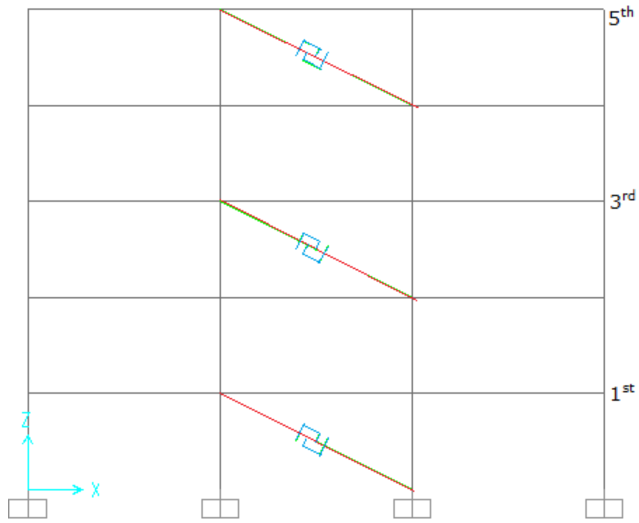


Figure 5.5: Location of Friction Damper in 5-storey RC framed structure

Table 5.3: Defining Parameters for Friction Damper

Link Property	*Plastic (Wen)*
Mass (Ton)	0.1
Yield Strength (kN)	100
Stiffness (kN/m)	100,000
Post Yield Stiffness Ratio	0.0001
Yielding Exponent	10

5.2 Seismic Response of 5-Storey RC Framed Structure

There are various ways of assessing seismic response, but computation of tip deflection is a reasonable measure of the overall effect of seismic response. For this reason any reduction in tip deflection represents a worthwhile reduction in overall seismic design force. The reduction in the tip accelerations at the top of the structure is also investigated. Efficiency of these damping systems was investigated for four earthquake excitation of different peak acceleration value, namely El Centro, Kobe, Northridge and Loma Prieta. The undamped structural model was created in order to compare its results with the results of the structures embedded with two damping systems, namely tuned mass damper and friction damper.

Fast Nonlinear time history Analysis (FNA) and response spectrum analysis were carried out for typical time histories and response spectra. It is useful for the structures, which have a linear characteristic and nonlinear acting only in the "link" elements, while plastic hinges and geometrical nonlinearity are not taken into account. The results shows that the value of reduction of tip deflection is independent on the complex characteristics of the time histories used for assessment. Hence the benefits can only be legitimately assessed if the analysis is carried out for suitable time histories.

5.3 Results of Response Spectrum Analysis

The response spectrum analysis is done using SAP2000 for IS-1893, Kobe, Northridge and Loma Prieta response spectra. Results of response spectrum analysis in form of displacement, storey drift, time period and frequency are discussed here.

5.3.1 Comparison of Displacement

The building is symmetrical in plan; therefore the properties of all the elements are same in both directions. The graphical representations of comparison of displacement for uncontrolled and controlled structure are presented in Figure 5.6. From results, it is evident that displacement of top storey is highest, so comparison of displacement is done at top of the structure.

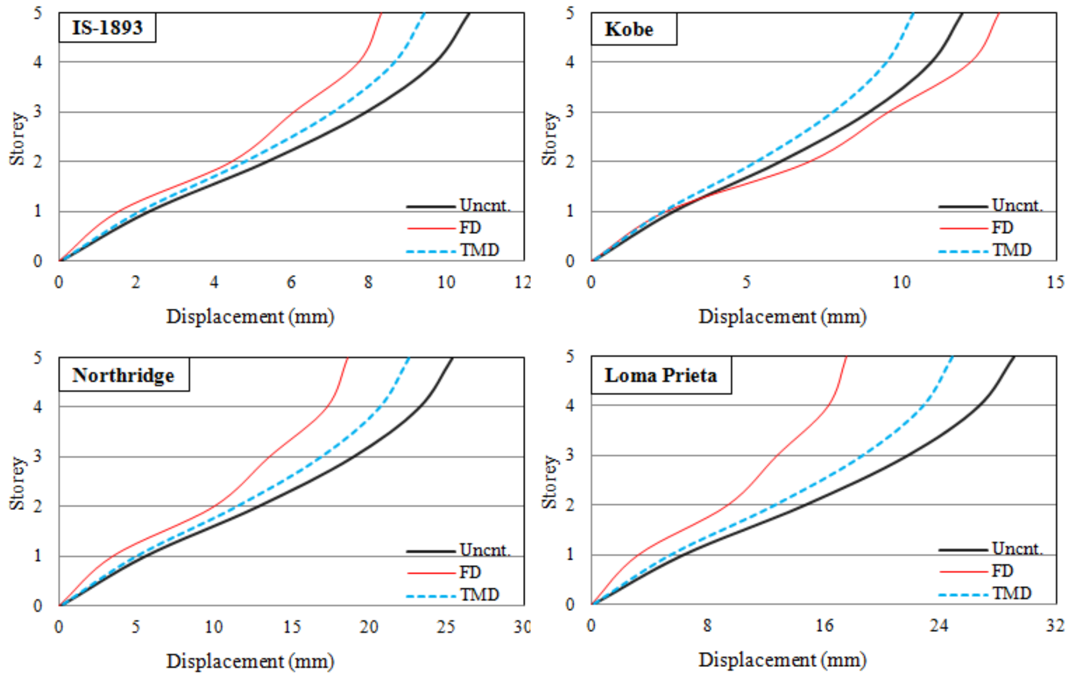


Figure 5.6: Comparison of Displacement for Response Spectrum Analysis

For tuned mass damper there is 11.1%, 13.2%, 11.4% and 14.6% reduction is observed for IS-1893, Kobe, Northridge and Loma Prieta response spectra respectively. While for friction damper there is a reduction of 21.6%, 26.8% and 39.7% is observed for IS-1893, Northridge and Loma Prieta response spectra. But for Kobe response spectra it is increased by 10% compared to uncontrolled structure. From results, it can be said that by use of tuned mass damper and friction damper there is substantial reduction in displacement except Kobe response spectra for friction damper.

5.3.2 Comparison of Storey Drift

The graphical representations of comparison of storey drift for uncontrolled and controlled structure are presented in Figure 5.7. From results, it is evident that storey drift is maximum at 2^{nd} storey level, so comparison of storey drift is done at level of 2^{nd} storey of the structure. It is clear from graphs that storey drift is constantly decreased by attaching TMD to structure. Decreasing in drift at 1^{st} 3^{rd} and 5^{th} storey is more where, friction damper is installed, as compared to other storey.

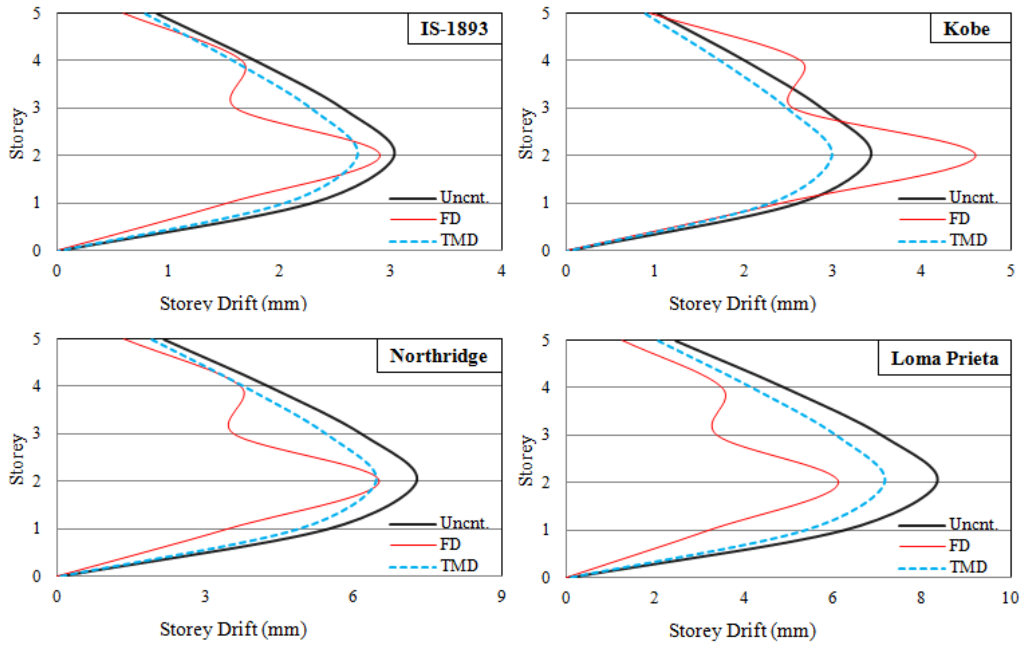


Figure 5.7: Comparison of Storey Drift for Response Spectrum Analysis

Tuned mass damper reduces the maximum storey drift by 10.9%, 12.8%, 11.2% and 14.3% for IS-1893, Kobe, Northridge and Loma Prieta response spectra respectively. While friction damper reduces the maximum storey drift by 4.3%, 10.7% and 26.3% for IS-1893, Kobe, Northridge and Loma Prieta response spectra respectively. But for Kobe response spectra it is increased by 34.5% compared to uncontrolled structure. From graphs it can be observed that by use of tuned mass damper and friction damper there is substantial reduction in storey drift except Kobe where trend in reduction of drift is opposite.

5.3.3 Comparison of Time Period and Frequency

The comparison of mode Vs time period and frequency for first 5-mode are illustrated in Figure 5.8 for uncontrolled and controlled structure. The time period of first mode of structure is increased by 40% for tuned mass damper, while it is decreased by 17% for friction damper. Frequency is inversely proportional to the time period, so for tuned mass damper it is reduced by 28.6% and for friction damper it is increased by 20.8%.

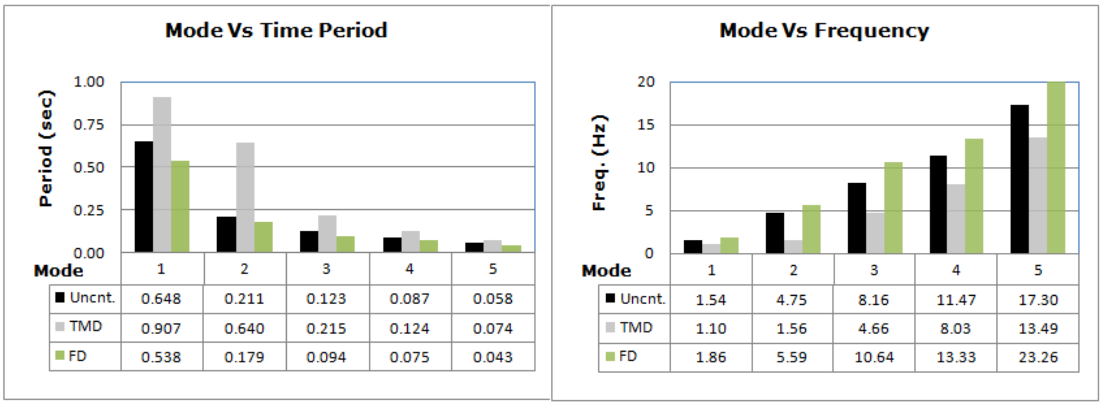


Figure 5.8: Mode Vs Time Period and Frequency

5.4 Results of Fast Nonlinear Time History Analysis (FNA)

The Fast Nonlinear time history Analysis (FNA) is done using SAP2000 for El Centro, Kobe, Northridge and Loma Prieta acceleration time histories. Results of FNA in form of hysteric loop for friction damper, energy function, displacement and acceleration time history and response spectra developed from time history are discussed here.

5.4.1 Hysteric Loop for Friction Damper

Figure 5.9 compares the force-deformation responses of the friction element located at 3^{rd} floor when structure is subjected to various time histories. Area of hysteresis loop indicates the amount of energy dissipated per cycle. The friction dampers have experienced several cycles of reversal and dissipated large amounts of seismic energy during slippage. The slope in the hysteric loop shows elastic deformation of brace.

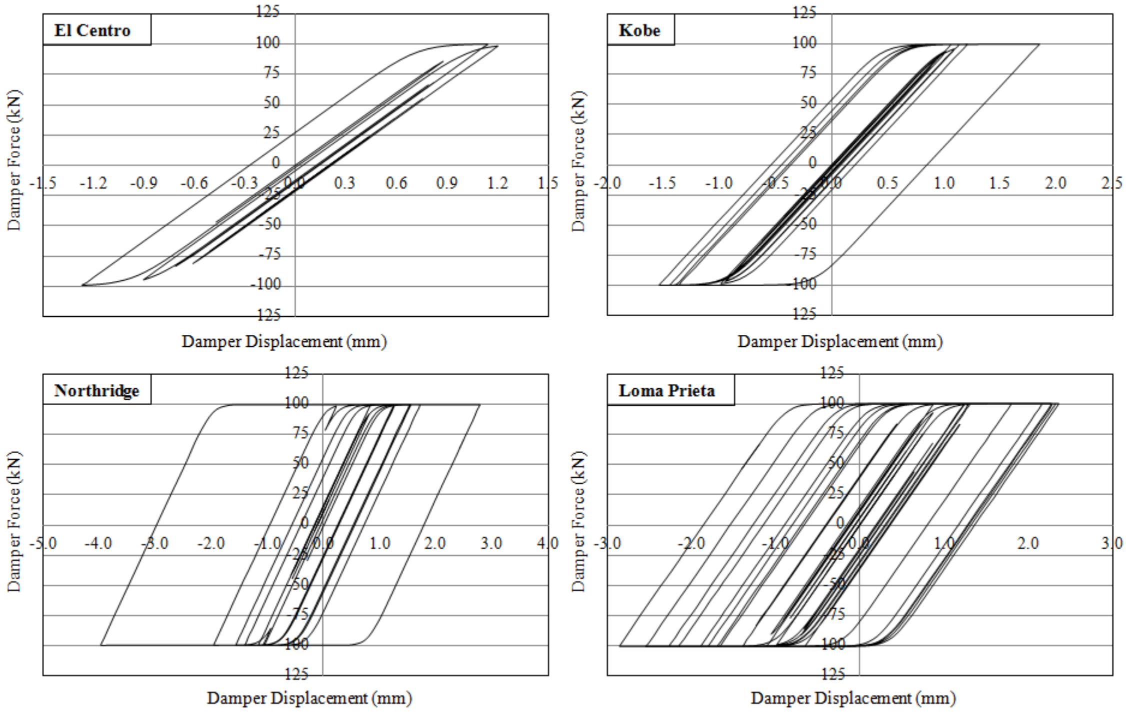


Figure 5.9: Hysteric loop for 100kN Diagonal Friction Damper

The maximum slippage in the damper is 1.27mm, 1.85mm, 3.97mm and 2.85mm for El Centro, Kobe, Northridge and Loma Prieta respectively. Area of hysteric loop of Northridge and Loma Prieta earthquakes are more, so maximum energy is dissipated in this earthquake. In El Centro and Kobe earthquake there will be less energy dissipation than other earthquake. Friction damper give better response in Northridge and Loma Prieta earthquake.

5.4.2 Comparison of Energy Function

Envelope of seismic energy input and energy dissipated by friction dampers and tuned mass damper are presented in Figure 5.10. It is seen that about 9%, 26%, 39% and 42% of seismic energy is dissipated by friction dampers, while for tuned mass damper

33%, 21%, 44% and 47% seismic energy is dissipated for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. It is seen from results that, tuned mass damper is more effective than friction damper except Kobe earthquake.

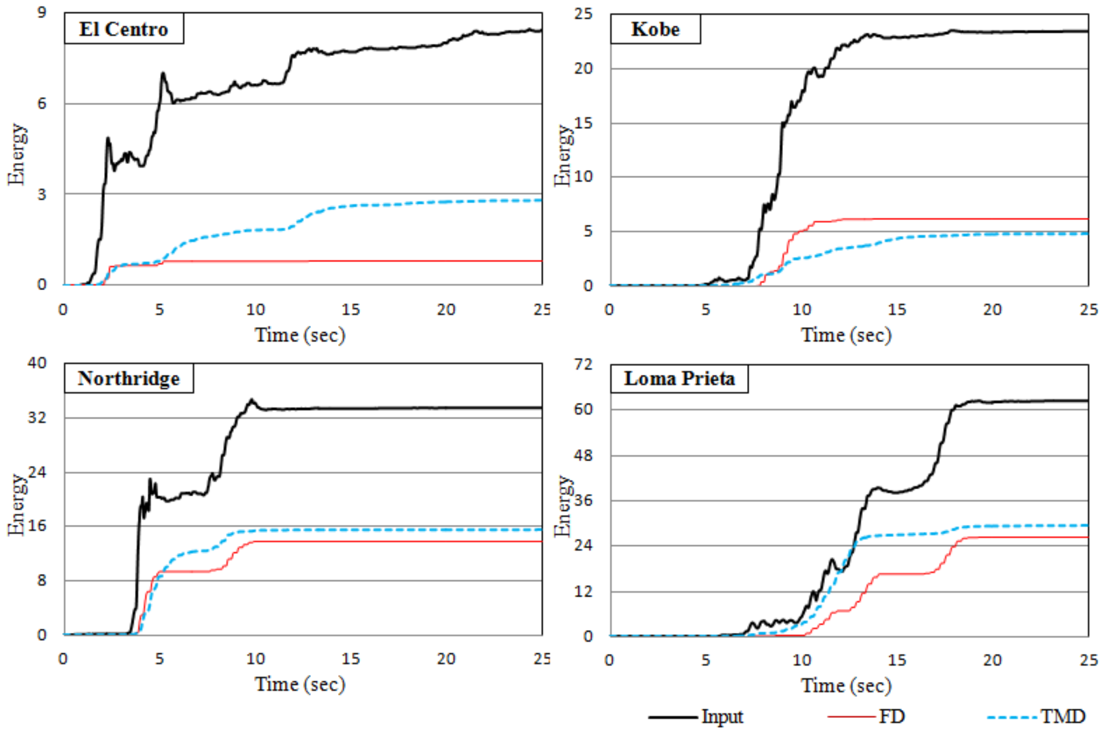


Figure 5.10: Energy Function for various earthquake

5.4.3 Comparison of Acceleration Response Spectra

Comparisons of acceleration response spectra of top storey for uncontrolled and controlled structure are shown in Figure 5.11. From results, it is evident that FD is more effective in Northridge and Loma Prieta earthquake, while TMD is more effective in El Centro and Kobe earthquake. Due to roughness characteristics of response spectra it is not directly used for design purpose. For design purpose smooth design response spectra must be made.

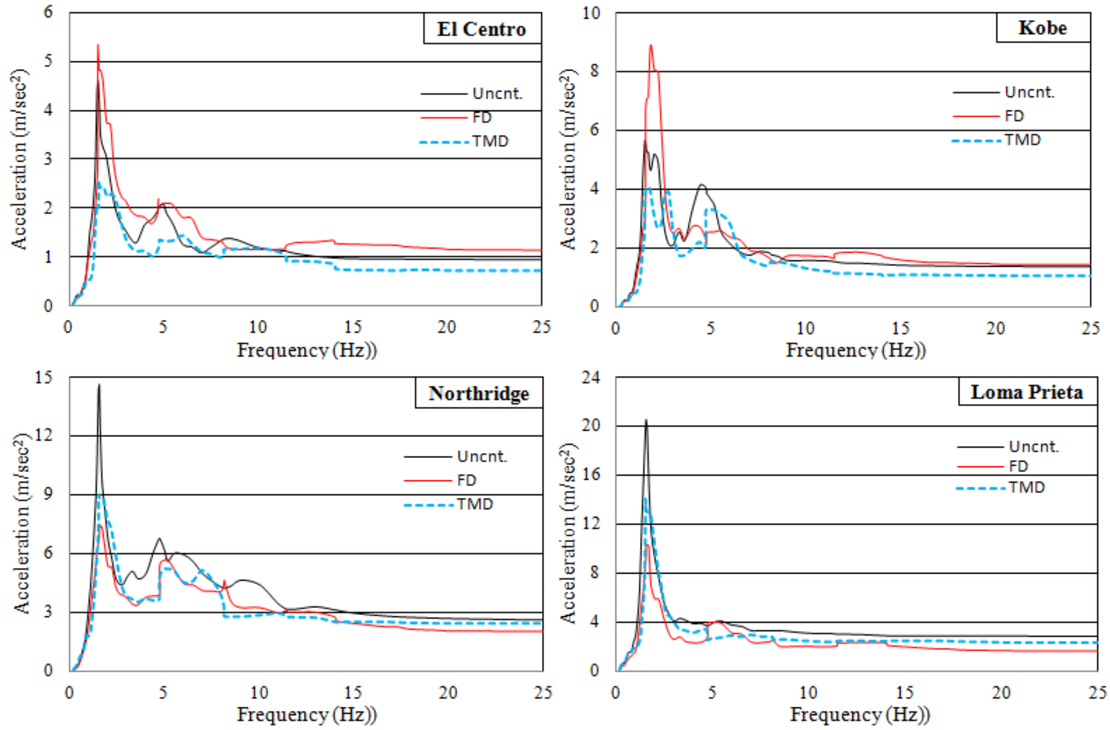


Figure 5.11: Acceleration Response Spectra

From results it is observed that for friction damper acceleration is reduced by 49% and 50% for Northridge and Loma Prieta earthquake, while for El Centro and Kobe earthquake it is increased by 15% and 58% respectively. For tuned mass damper acceleration is decreased for all earthquake i.e. 44%, 29%, 39% and 31% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively.

5.4.4 Comparison of Acceleration Time History

Figure 5.12 illustrate the tip acceleration of the structure with diagonal friction damper fitted in 1^{st}, 3^{rd} and 5^{th} storey in central bay on all outer periphery of building and tuned mass damper attached at top of the 5^{th} storey with uncontrolled structure obtained under four earthquake excitations.

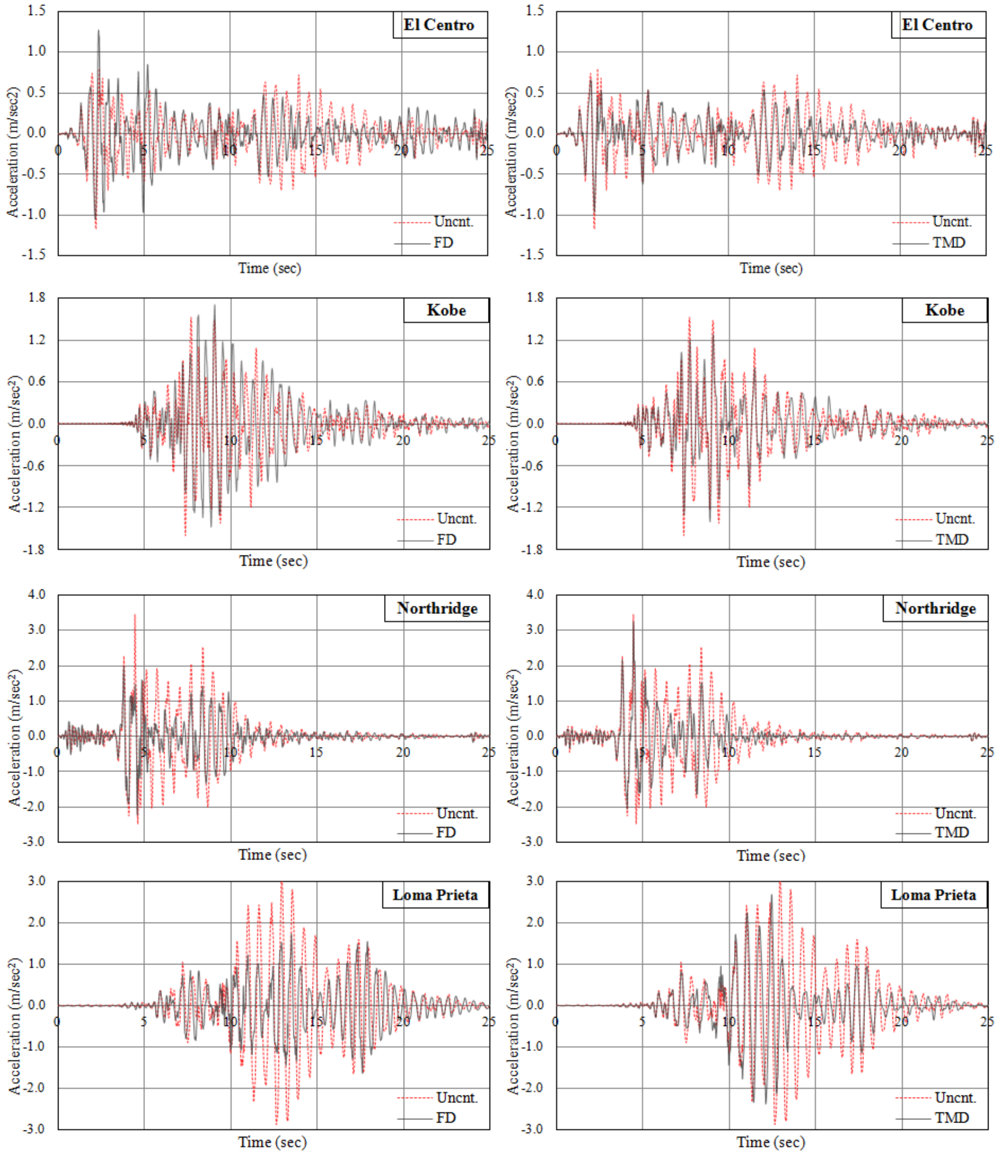


Figure 5.12: Acceleration Time Histories Recorded at Level of Storey-5

The tip acceleration of this structure experienced for four earthquake excitations are presented in Table 5.4. From results and graphs, it was evident that for friction damper there is maximum reduction of 43.6% for Loma Prieta earthquake and minimum reduction of 6.7% for Kobe earthquake is observed. While for tuned mass damper there is maximum reduction of 17% for El Centro earthquake and minimum reduction of 10.4% for Northridge earthquake is achieved. It can be seen, that the diagonal friction damper surpassed the tuned mass damper in their ability to reduce the intensity of the initial strong motion for Northridge and Loma Prieta earthquakes.

Table 5.4: Maximum Acceleration of 5-Storey Structure in m/sec^2

Earthquake	*Uncnt.*	*FD*	*% Reduction*	*TMD*	*% Reduction*
El Centro	0.985	0.897	**8**.9	0.817	**17**.0
Kobe	1.559	1.453	**6**.8	1.344	**13**.8
Northridge	2.956	2.095	**29**.1	2.648	**10**.4
Loma Prieta	2.995	1.687	**43**.6	2.528	**15**.6

5.4.5 Comparison of Displacement Time History

Figure 5.13 illustrate the tip displacement of the structure with diagonal friction damper fitted in 1^{st}, 3^{rd} and 5^{th} storey in central bay on all outer periphery of building and tuned mass damper attached at top of the 5^{th} storey with uncontrolled structure obtained under four earthquake excitations. The tip displacement of this structure experienced for four earthquake excitations are presented in Table 5.5.

Table 5.5: Maximum Displacement of 5-Storey Structure in mm

Earthquake	*Uncnt.*	*FD*	*% Reduction*	*TMD*	*% Reduction*
El Centro	8.00	7.49	**6**.3	7.01	**12**.4
Kobe	11.57	10.42	**9**.9	9.99	**13**.6
Northridge	24.78	15.93	**35**.7	23.19	**6**.4
Loma Prieta	28.87	13.28	**54**.0	27.54	**4**.6

It was seems that for Northridge and Loma Prieta earthquake there is a maximum reduction in displacement of 35.7% and 54% is observed for friction damper. While for El Centro and Kobe earthquake maximum reduction is observed as 12.4% and 13.6% for tuned mass damper. From that it can be said that friction damper is more effective for peak seismic excitation region while tuned mass damper is more effective in medium seismic excitation region for 5-storey structure.

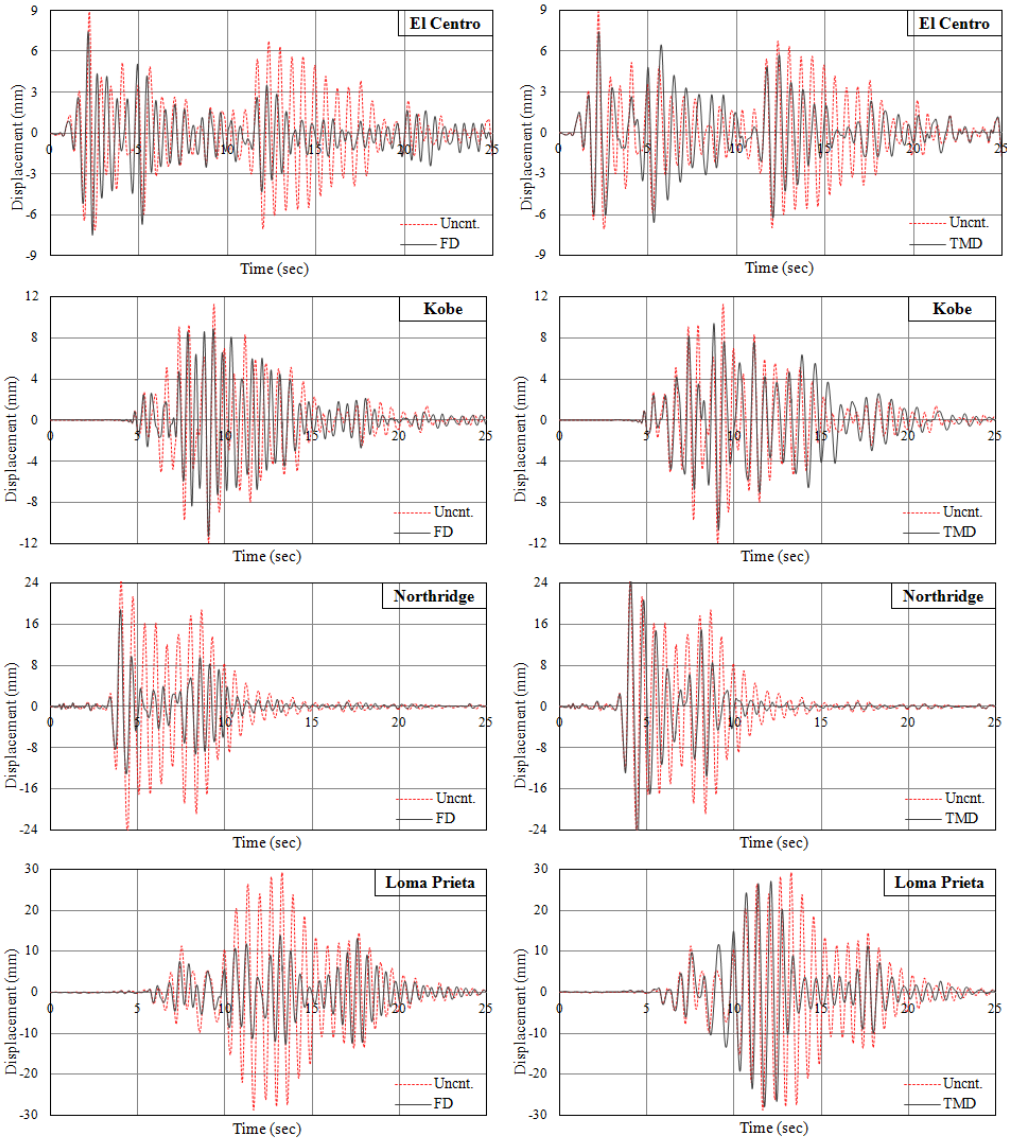


Figure 5.13: Displacement Time Histories recorded at level of storey-5

5.5 Summary

This chapter deals with the Response spectrum and Fast Nonlinear time history analysis of uncontrolled and controlled structure for various presented time histories and response spectra to finding out the effective damping system for reducing the response of 5-storey RC framed structure during earthquake.

The results of response spectrum analysis show that the impact of friction damper is more in reduction of displacement, storey drift and time period of structure except Kobe earthquake. But for friction damper reduction in storey drift is irregular, because storey drift is reduced where friction damper is attached structure.

The results of fast nonlinear time history Analysis of 5-storey RC framed structure confirmed that; in terms of tip deflection, and tip acceleration reduction, the friction damper is superior for Northridge and Loma Prieta earthquake, while tuned mass damper give better performance for El Centro and Kobe earthquakes. In terms of energy dissipation highest dissipation were recorded for Loma Prieta earthquake for Tuned mass damper, even these dampers remained unreliable under Kobe earthquake.

Chapter 6

Analysis of 15-Storey Structure

In order to feasibility of damping systems for wall frame structure 15-storey RC wall-framed structure is analyzed. In this chapter uncontrolled and controlled structure is taken for analysis. The comparisons of various parameters of uncontrolled and controlled structures are also presented. The nonlinear dynamic analysis of 15 storey RC wall-framed structure is carried out using SAP2000 software.

6.1 Structural Data

A 15-storey RC wall-framed structure was investigated in this study. Two different type of damping systems were considered for analysis i.e. tuned mass damper and friction damper. Seismic analysis of these wall-frame structure was carried out with one type of damping system at a time. The undamped structure was also analyzed in order to compare results.

6.1.1 Structural Configuration

The structural layout of 15-storey RC wall framed structure considered for the modeling and analysis are shown in Figure 6.1. The coupled shear wall is introduced in the configuration of building to resist lateral loading. The plan of building considered is approximately symmetrical, to avoid the case of non-proportional damping in a structure, which helps damping devices to be distributed symmetrically in a building.

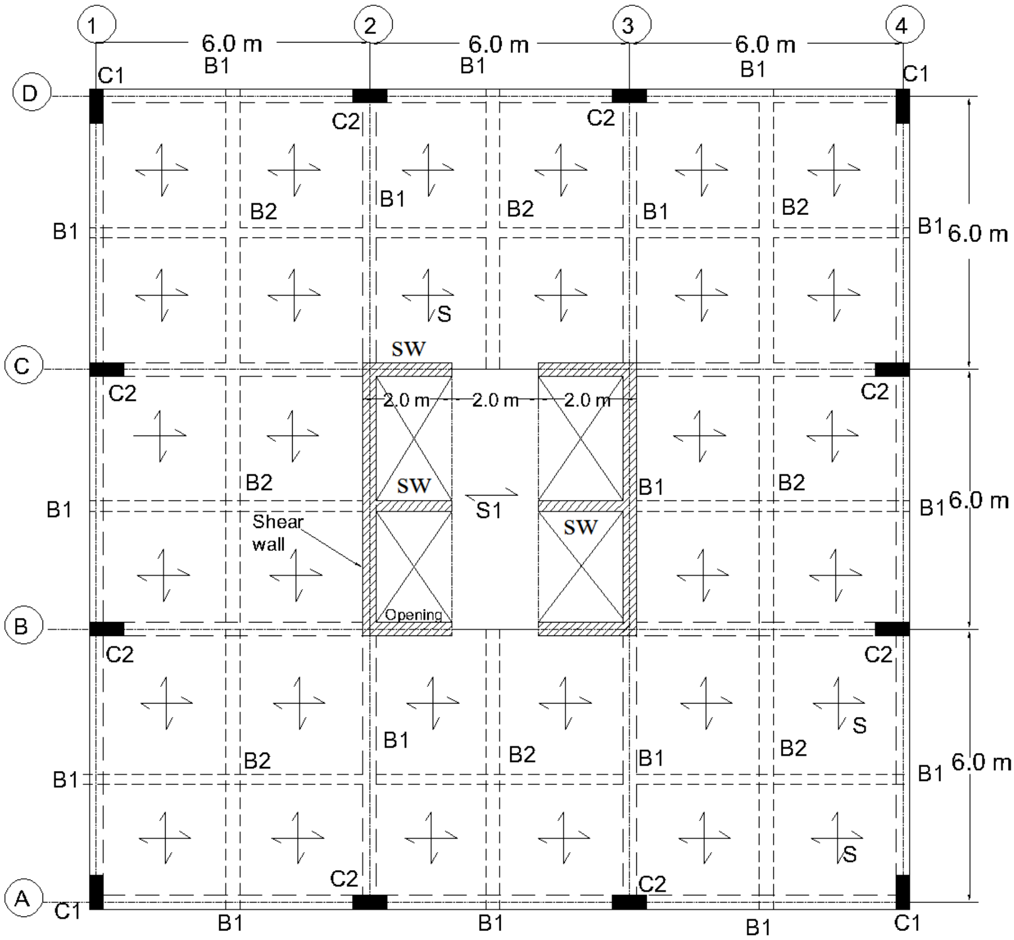


Figure 6.1: Plan of 15-Storey RC Wall-Framed Structure

6.1.2 Properties of Building

The material and loading data are enlisted below.

Material data

Concrete Grade = M30

Elasticity of concrete = 27386.1 N/mm^2

Rest of the material data are kept to be same as previously described in chapter-5, Analysis of 5-Storey Structure.

Loading data

All loading data are kept to be same as previously mentioned in chapter-5 Analysis of 5-Storey Structure.

Sectional Properties

Approximate sizes of beam, column, slab and shear wall are summarized in Table 6.1.

Table 6.1: Approximate Size of Members for 15-Storey Structure

Sr. No	*Element*	*Notation*	*Size (mm)*
1	Main Beam	B1	300 × 600
2	Secondary Beam	B2	230 × 500
3	Column	C1	300 × 750
4	Column	C2	750 × 300
5	Slab	S & S1	125
6	Shearwall	SW	150

Boundary condition

The earthquake events used in this study are recorded as time history accelerations and response spectra in the horizontal plane. The time history accelerations and response spectra are applied in X-direction as well as Y-direction for time history analysis. For analysis of building fixed support is considered.

6.1.3 Properties and Location of Tuned Mass Damper

Tuned mass damper is placed at top of the structure i.e. on 15^{th} storey, for achieving maximum response. Characteristics of tuned mass damper, which consist of nonlinear damper elements and spring elements, are shown in Table 6.2. Assume mass of damper as 2% of total structural mass and from mass other parameters are calculated. The calculation of parameters of tuned mass damper are based on Appendix-A. These parameters are used for modeling of tuned mass damper in SAP2000.

Table 6.2: Defining Parameters for Tuned Mass Damper

Link Property	*Damper*	*Spring*
Mass (Ton)	107.9	-
Stiffness (kN/m)	X-dir. = 49305.2	X-dir. = 2465.2
	Y-dir. = 49305.2	Y-dir. = 2465.2
	Z-dir. = 49305.2	Z-dir. = 24652.6
Damping Coefficient (kN.sec/m)	X-dir. = 455.38	
	Y-dir. = 455.38	
	Z-dir. = 455.38	

6.1.4 Properties and Location of Friction Damper

Friction dampers are placed at 1^{st}, 4^{th}, 7^{th}, 10^{th} and 13^{th} storey in central bay on all outer periphery of building. A detail of diagonal friction damper located in 15-storey RC framed structure is shown in Figure 6.2. A total of 20 friction damper is used for reducing the response of the structure.

Table 6.3: Defining Parameters for Friction Damper

Link Property	*Plastic (Wen)*
Mass (Ton)	0.1
Yield Strength (kN)	100
Stiffness (kN/m)	100,000
Post Yield Stiffness Ratio	0.0001
Yielding Exponent	10

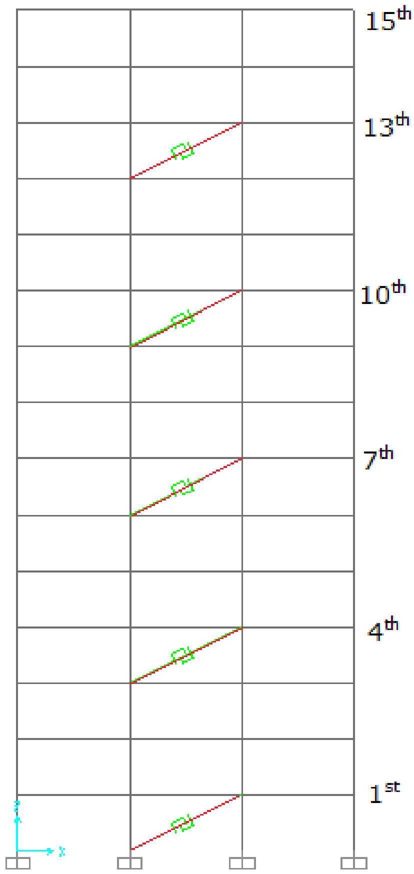


Figure 6.2: Location of Friction Damper in 15-storey RC Wall-framed Structure

6.2 Results of Response Spectrum Analysis

The response spectrum analysis is done using SAP2000 for IS-1893, Kobe, Northridge and Loma Prieta response spectra. Results of response spectrum analysis in form of displacement, storey drift, time period and frequency are discussed here.

6.2.1 Comparison of Displacement

The building is symmetrical in plan; therefore the properties of all the elements are same in both directions. The graphical representations of comparison of displacement for uncontrolled and controlled structure are presented in Figure 6.3. From results, it is evident that displacement of top storey is highest, so comparison of displacement is done at top of the structure.

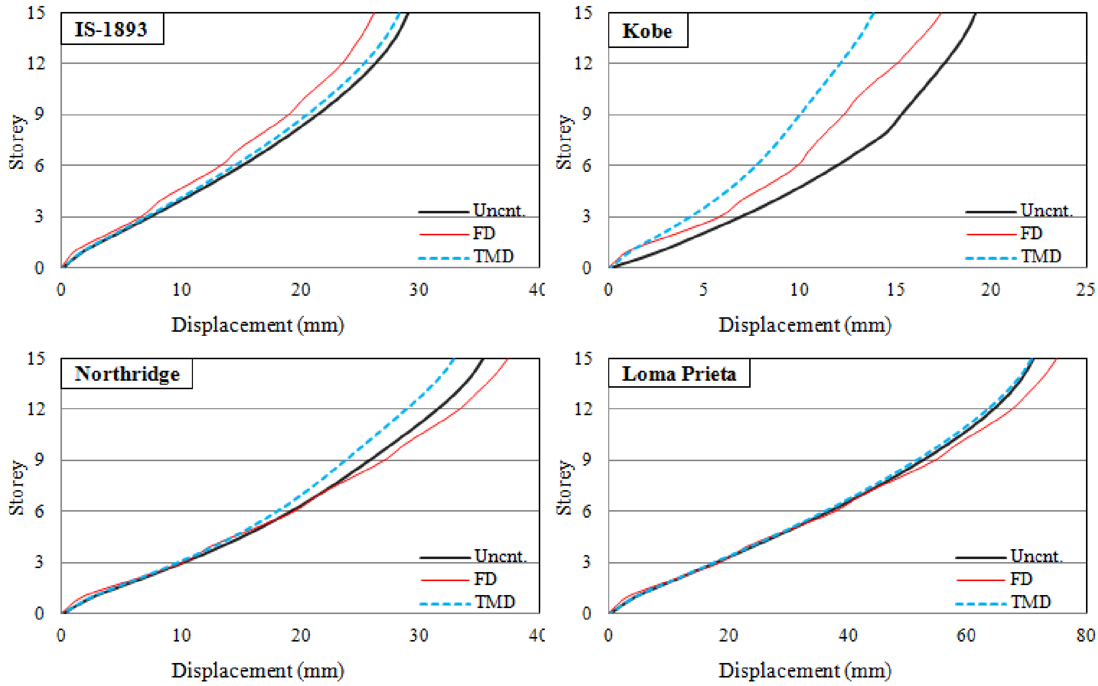


Figure 6.3: Comparison of Displacement for Response Spectrum Analysis

Tuned mass damper reduces the maximum displacement by 2.6%, 28%, 7% and 0.5% for IS-1893, Kobe, Northridge and Loma Prieta response spectra respectively. While for friction damper reduction is only observed in El Centro and Kobe earthquake by 9.9% and 9.3% respectively. But for Northridge and Loma Prieta response spectra trend of reduction of displacement is opposite. While comparing the results of tip displacement reduction for friction damper and tuned mass damper, the friction damper perform significantly better.

6.2.2 Comparison of Storey Drift

The graphical representations of comparison of storey drift for uncontrolled and controlled structure are illustrated in Figure 6.4. It is clear from graphs that storey drift is constantly decreased by attaching tuned mass damper to structure. While for friction damper reduction in storey drift is inconsistent and mostly insignificant.

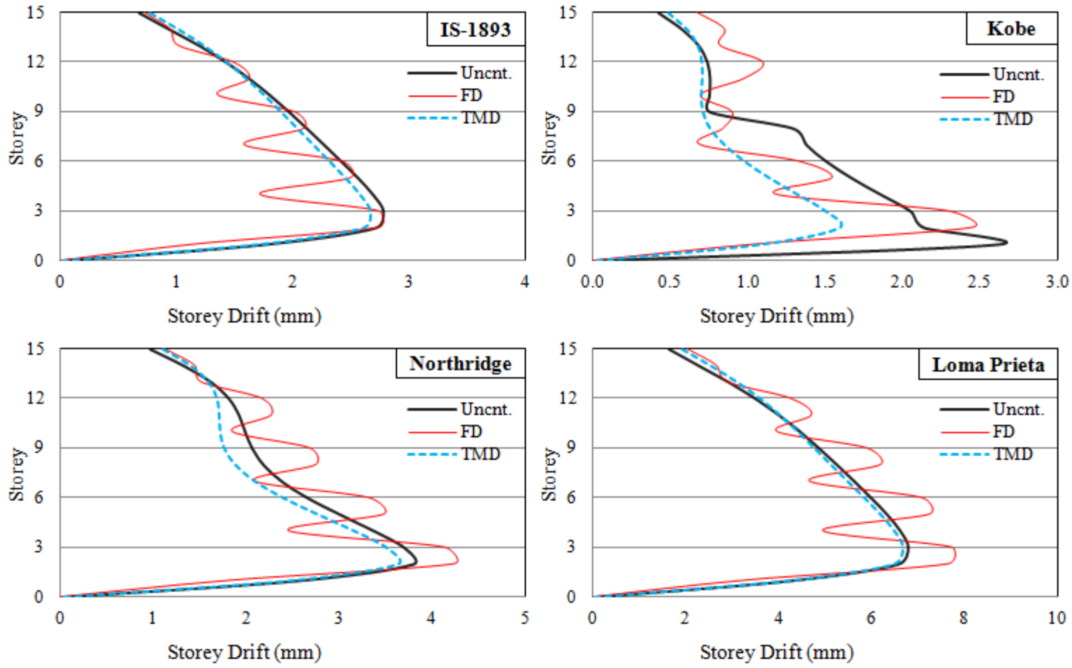


Figure 6.4: Comparison of Storey Drift for Response Spectrum Analysis

Tuned mass damper reduces the maximum storey drift by 4.2%, 39.3%, 4% and 2% for IS-1893, Kobe, Northridge and Loma Prieta response spectra. While friction damper maximum storey drift is increased for Northridge and Loma Prieta response spectra by 12% and 13%. From graphs it can be observed that by use of tuned mass damper there is considerable reduction in storey drift is observed for all earthquakes.

6.2.3 Comparison of Time Period and Frequency

The comparison of mode Vs time period and frequency for first 5-mode are shown in Figure 6.5 for uncontrolled and controlled structure. In tuned mass damper added structure time period for first mode is increased by 6.7% because mass of structure is increased by attaching tuned mass damper at top of building. While for friction damper added structure time period for first mode is decreased by 14.2%. Frequency is inversely proportional to the time period, so for tuned mass damper it is reduced by 7.1% and for friction damper it is increased by 16%.

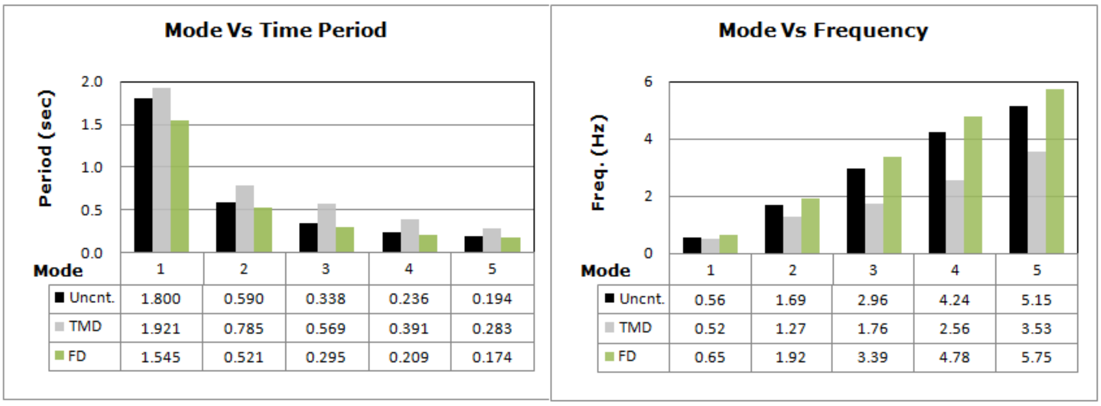


Figure 6.5: Mode Vs Time Period and Frequency

6.3 Results of Fast Nonlinear Time History Analysis (FNA)

The Fast Nonlinear time history Analysis (FNA) is done using SAP2000 for El Centro, Kobe, Northridge and Loma Prieta acceleration time histories. Results of FNA in form of hysteric loop for friction damper, energy function, displacement and acceleration time history and response spectra developed from time history are discussed here.

6.3.1 Hysteric Loop for Friction Damper

The force-deformation relationship of friction damper located at 4^{th} storey level is shown in Figure 6.6 for various earthquakes. The friction dampers have experienced several cycles of reversal and dissipated large amounts of seismic energy during slippage.

The maximum deformation of damper is 2.28mm, 3.2mm, 3.1mm and 5.68mm for El Centro, Kobe, Northridge and Loma Prieta respectively. For Kobe and Loma Prieta earthquake hysteric loop is perfectly rectangular, so maximum earthquakes energy is dissipated for these earthquakes. Friction damper also give better performance for El Centro and Northridge earthquake, but it is less than Kobe and Loma Prieta earthquakes.

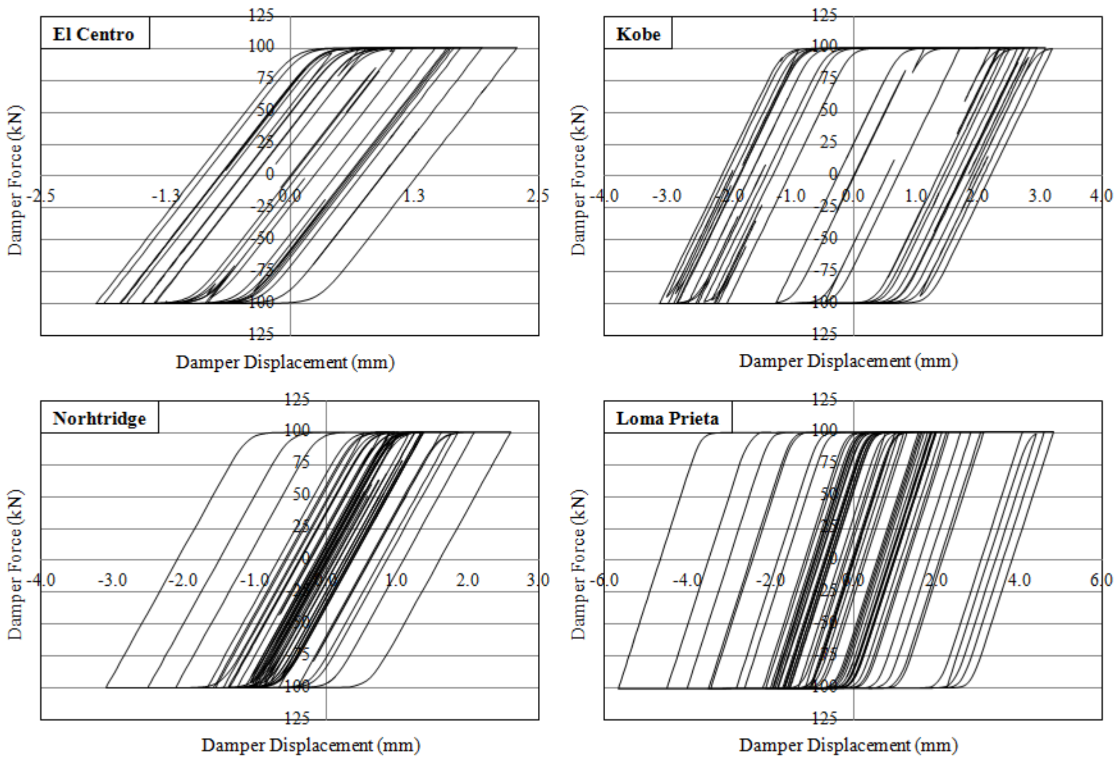


Figure 6.6: Hysteric loop for 100kN Diagonal Friction Damper

6.3.2 Comparison of Energy Function

The graphical representation of input energy and energy released by friction damper and tuned mass damper are presented in Figure 6.7. From results it can be said that seismic energy dissipated by friction damper is 17.8%, 24.8%, 20.7% and 21.5%, while for tuned mass damper reduction is 25.5%, 24.5%, 25% and 18.7% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. It is seen from results that, tuned mass damper and friction damper both give more or less similar behavior for reducing input energy of seismic excitation.

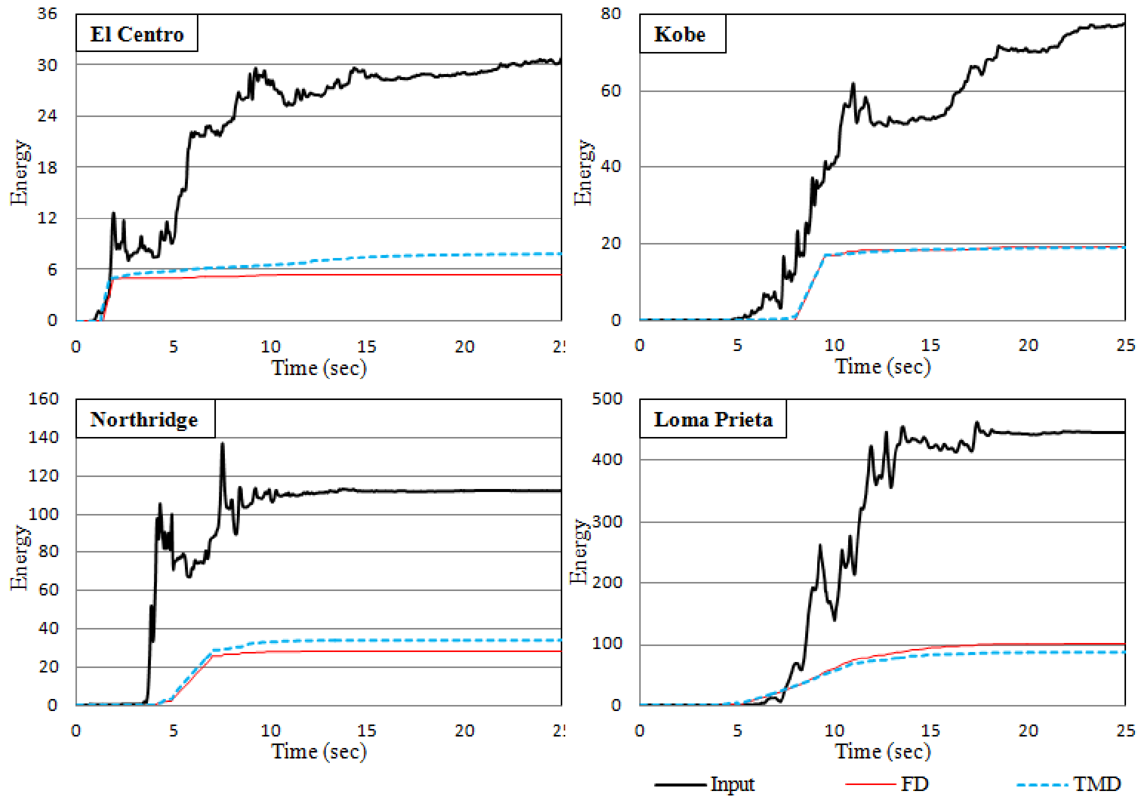


Figure 6.7: Energy Function for various earthquake

6.3.3 Comparison of Acceleration Response Spectra

Envelope of top storey response spectra of uncontrolled structure and controlled structure is disclosed in Figure 6.8. Tuned mass damper reduces the maximum acceleration response by 36.5% and 39% for El Centro and Kobe time history but it is increased by 2% for Northridge and Loma Prieta earthquake. While friction damper reduces the maximum acceleration response by 34.2%, 29.7% and 27.3% for El Centro, Northridge and Loma Prieta earthquake. But for Kobe earthquake it is increased by 20%. From results, it can be said that friction damper give better performance under all earthquake except Kobe earthquake.

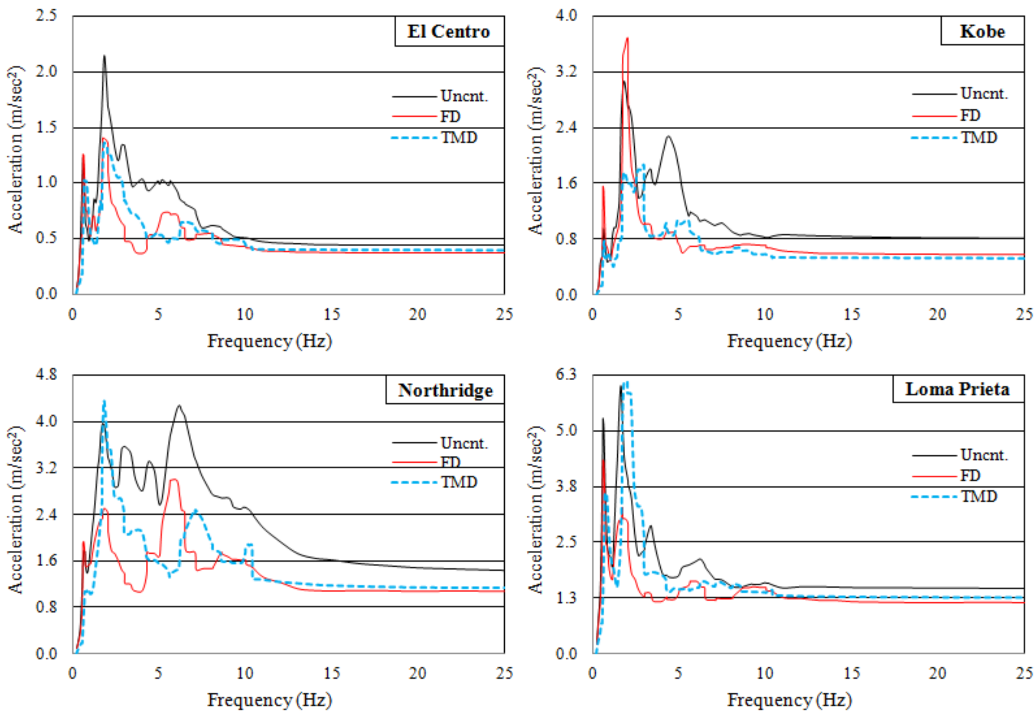


Figure 6.8: Acceleration Response Spectra

6.3.4 Comparison of Acceleration Time History

The tip acceleration of uncontrolled and controlled structure is illustrated in Figure 6.9. A friction damper of 100kN slip load is attached to the structure at 1^{st}, 4^{th}, 7^{th}, 10^{th} and 13^{th} storey level in central bay on all outer periphery of building. A tuned mass damper of 107.9 ton is attached to top storey of structure. The comparison of tip acceleration of structure is illustrated in Table 6.4.

From results, it is evident that friction damper reduces the tip acceleration by 15%, 36.3%, 8.6% and 12.9% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. While tuned mass damper reduces the maximum tip acceleration by

31.5%, 50.4%, 11.8% and 15.1% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. While comparing the results of tip acceleration of reductions experienced by structure for controlled and uncontrolled structure, the tuned mass damper perform significantly better under all earthquakes.

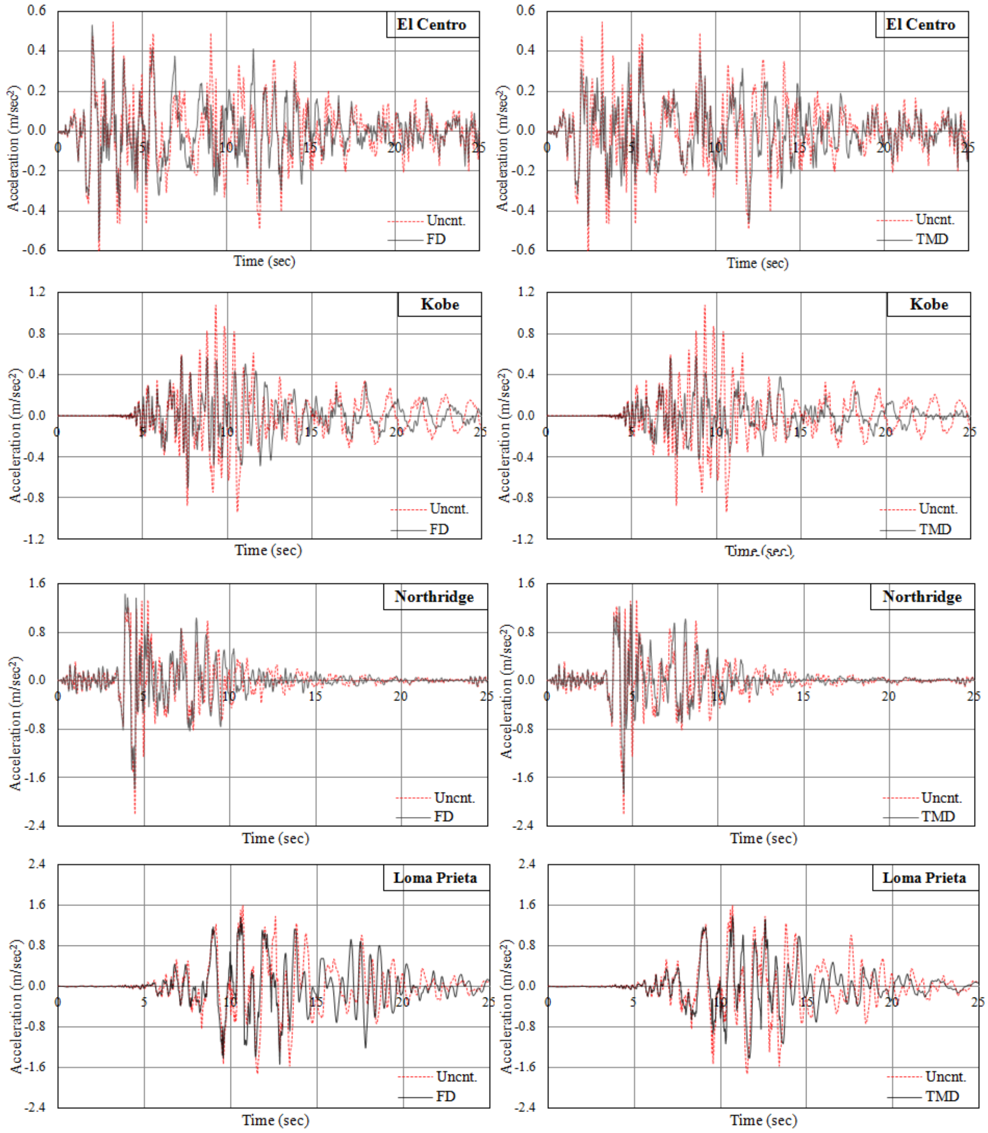


Figure 6.9: Acceleration Time Histories Recorded at level of Storey-15

Table 6.4: Maximum Acceleration of 15-Storey Structure in m/sec^2

Earthquake	*Uncnt.*	*FD*	*% Reduction*	*TMD*	*% Reduction*
El Centro	0.634	0.539	**15.0**	0.434	**31.5**
Kobe	1.002	0.638	**36.3**	0.497	**50.4**
NR	1.761	1.609	**8.6**	1.554	**11.8**
LP	1.660	1.446	**12.9**	1.410	**15.1**

6.3.5 Comparison of Displacement Time History

Figure 6.10 illustrate the tip displacement of the structure with diagonal friction damper and tuned mass damper with uncontrolled structure obtained under four earthquake excitations. The comparison of maximum tip displacement of structure experienced for four earthquake excitations are presented in Table 6.5. It can be seems

Table 6.5: Maximum Displacement of 15-Storey Structure in mm

Earthquake	*Uncnt.*	*FD*	*% Reduction*	*TMD*	*% Reduction*
El Centro	16.6	15.0	**9.7**	14.2	**14.4**
Kobe	17.3	13.6	**21.7**	13.2	**23.6**
NR	34.1	31.1	**8.6**	30.5	**10.4**
LP	71.4	57.0	**20.2**	55.1	**22.8**

that for tuned mass damper, tip storey displacement is reduction by 14.4%, 23.6%, 10.4% and 22.8% observed under El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. In case of friction damper reduction in maximum displacement is observed by 9.7%, 21.7%, 8.6% and 20.2% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. It can be seen that the tuned mass damper suppressed the friction damper in their ability to reduce the tip storey displacement for all above mentioned earthquakes.

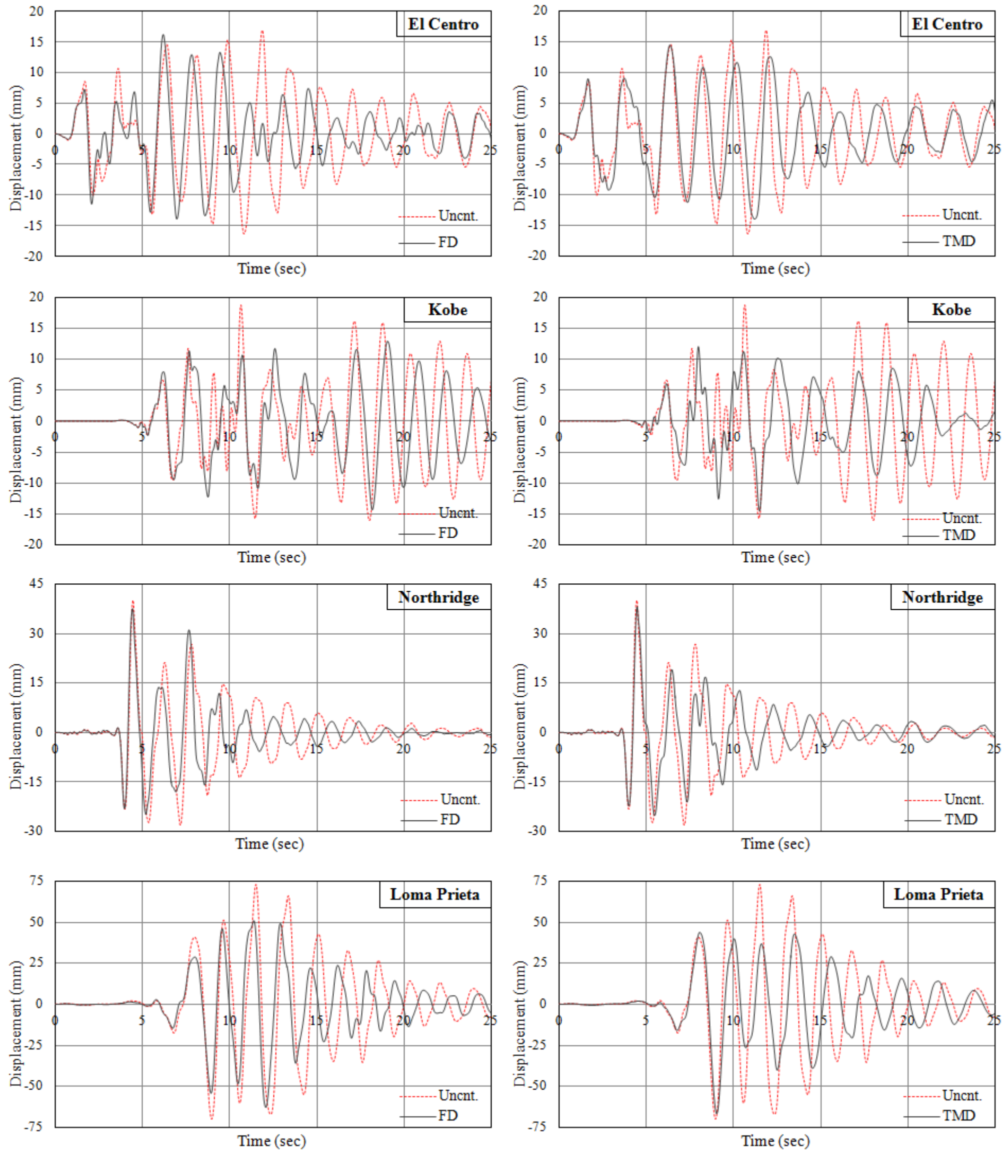


Figure 6.10: Displacement Time Histories Recorded at level of Storey-15

6.4 Summary

In this chapter the results of Dynamic analysis of 15-storey RC wall-framed structure were presented to illustrate the application of dampers to reduce the response of structure subjected to seismic disturbances. Also comparison between friction damper and tuned mass damper is done for appropriate selecting damping system for 15 storey wall-framed structure.

The results of response spectrum analysis of 15-storey RC wall-frame structure confirmed that; friction damper have a less influence in reduction of displacement, storey drift and frequency of structure and for storey drift reduction is fluctuated. While reducing the response of structure, influence of tuned mass damper is considerable.

It is shown from results of Fast Nonlinear time history Analysis; tuned mass damper is significantly reduces the dynamic response of structure in terms of tip acceleration, tip displacement and energy dissipation. However friction damper are also participate in seismic energy dissipation.

Chapter 7

Analysis of 25-Storey Structure

Finite element analysis of 25-storey RC framed structure is discussed. Three kind of RC wall framed buildings were taken for analysis purpose; uncontrolled structure, structure embedded with tuned mass damper and friction dampers. Also the influence of various dampers on overall analysis of structure is finding out. For modeling and analysis of structures SAP2000 software was used.

7.1 Structural Data

The third structural type investigated in this study was represented by 25-storey RC wall-framed structure. Two different types of dampers namely Tuned Mass Damper and diagonal Friction Damper were considered for analysis of 25-storey structure. Seismic analysis of these frame structure was carried out with one type of damping system at a time. The undamped structure was also analyzed in order to compare results.

7.1.1 Structural Configuration

The structural layout of 25-storey RC wall framed structure considered for the modeling and analysis are shown in Figure 7.1. The coupled shear wall is introduced in the configuration of building. The plan of building considered is approximately symmetrical, to avoid the case of non-proportional damping in a structure, which helps damping devices to be distributed symmetrically in a building.

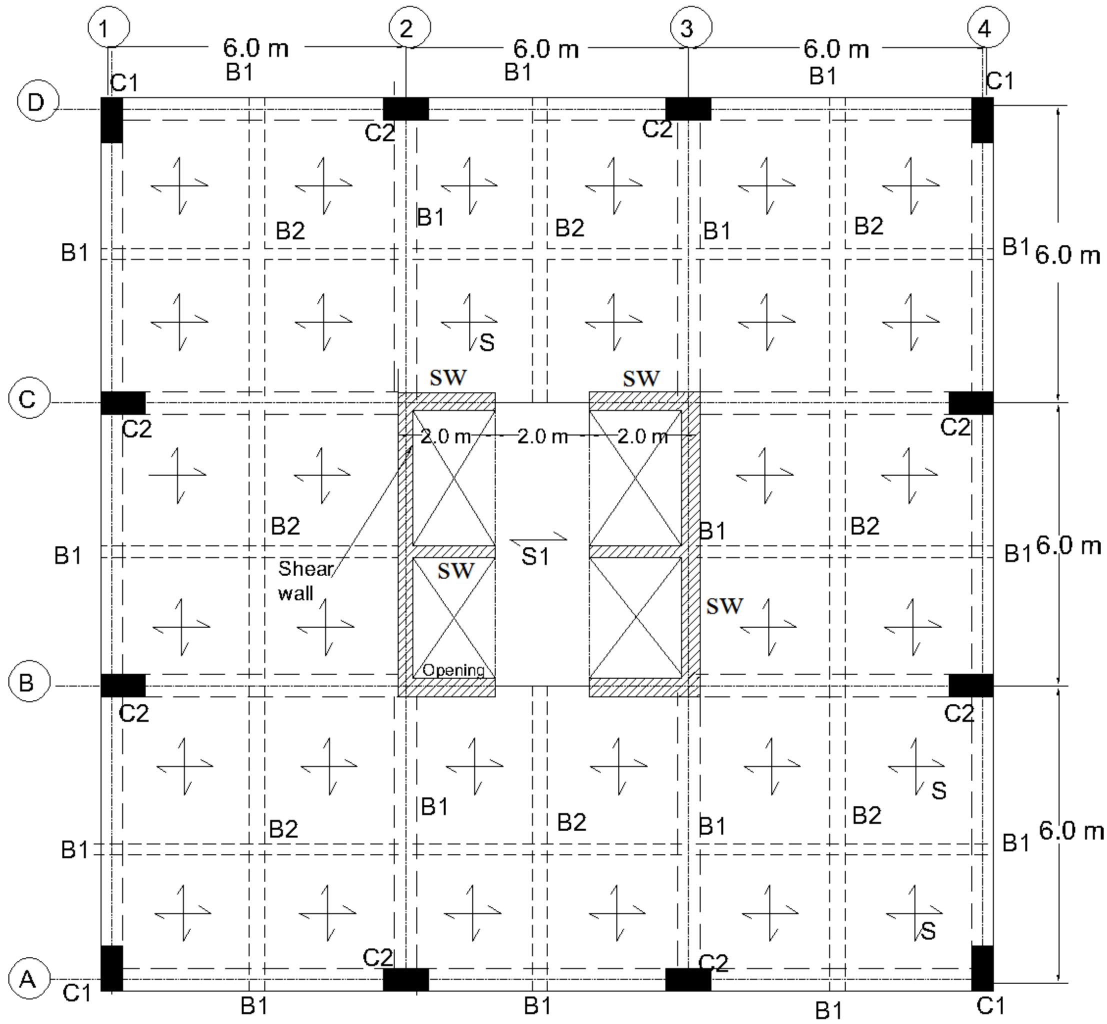


Figure 7.1: Plan of 25-Storey RC Wall-Framed Structure

7.1.2 Properties of Building

The material and loading data are enlisted below for 25-storey RC wall-framed structure.

Material data

Concrete Grade = M40

Elasticity of concrete = 31622.7 N/mm^2

Rest of the material data are kept to be same as previously described in chapter-5, Analysis of 5-Storey Structure.

Loading data

All loading data are kept to be same as previously mentioned in chapter-5, Analysis of 5-Storey Structure.

Sectional Properties

Approximate sizes of shear wall, slab, beam and column are summarized in Table 7.1.

Table 7.1: Approximate Size of Members for 25-Storey Structure

Sr. No	*Element*	*Notation*	*Size (mm)*
1	Main Beam	B1	450 × 900
2	Secondary Beam	B2	300 × 750
3	Column	C1	450 × 900
4	Column	C2	900 × 450
5	Slab	S & S1	125
6	Shearwall	SW	250

Boundary condition

The earthquake events used in this study are recorded as time history accelerations and response spectra in the horizontal plane. The time history accelerations and response spectra are applied in X-direction as well as Y-direction for time history analysis. For analysis of building fixed support is considered.

7.1.3 Properties and Location of Tuned Mass Damper

Tuned mass damper is placed at top of the structure for achieving maximum response. Characteristics of tuned mass damper, which consist of nonlinear damper elements and spring elements, are shown in Table 7.2. Assume mass of damper as 2% of total structural mass and from mass other parameters are calculated. The calculation of parameters of tuned mass damper are based on Appendix-A. These parameters are used for modeling of tuned mass damper in SAP2000. .

Table 7.2: Defining Parameters for Tuned Mass Damper

Link Property	*Damper*	*Spring*
Mass (Ton)	204.18	-
Stiffness (kN/m)	X-dir. = 93680.4	X-dir. = 4684.0
	Y-dir. = 93680.4	Y-dir. = 4684.0
	Z-dir. = 93680.4	Z-dir. = 46840.2
Damping Coefficient (kN.sec/m)	X-dir. = 732.51	
	Y-dir. = 732.51	
	Z-dir. = 732.51	

7.1.4 Properties and Location of Friction Damper

Friction dampers are placed at 1^{st}, 4^{th}, 7^{th}, 10^{th}, 13^{th}, 16^{th}, 19^{th}, 22^{nd} and 25^{th} storey in central bay on all outer periphery of building. A detail of diagonal friction damper located in 25-storey RC framed structure is shown in Figure 7.2. A total 32 number of friction damper is used for reducing the response of the structure. Characteristics of friction damper are shown in Table 7.3. These parameters are used for modeling of friction damper in SAP2000. Slip load of friction damper is assumed as 100kN.

Table 7.3: Defining Parameters for Friction Damper

Link Property	*Plastic (Wen)*
Mass (Ton)	0.1
Yield Strength (kN)	100
Stiffness (kN/m)	100,000
Post Yield Stiffness Ratio	0.0001
Yielding Exponent	10

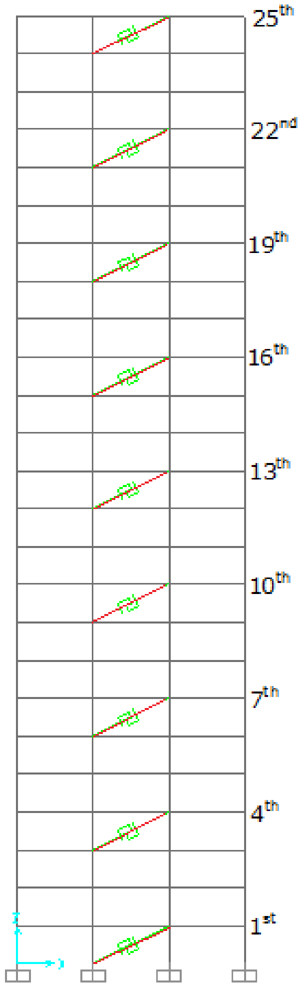


Figure 7.2: Location of Friction Damper in 25-storey RC Wall-framed Structure

7.2 Results of Response Spectrum Analysis

The response spectrum analysis is done using SAP2000 for IS-1893, Kobe, Northridge and Loma Prieta response spectra. Results of response spectrum analysis in form of displacement, storey drift, time period and frequency are discussed here.

7.2.1 Comparison of Displacement

The building is symmetrical in plan; therefore the properties of all the elements are same in both directions. The graphical representations of comparison of displacement for uncontrolled and controlled structure are presented in Figure 7.3.

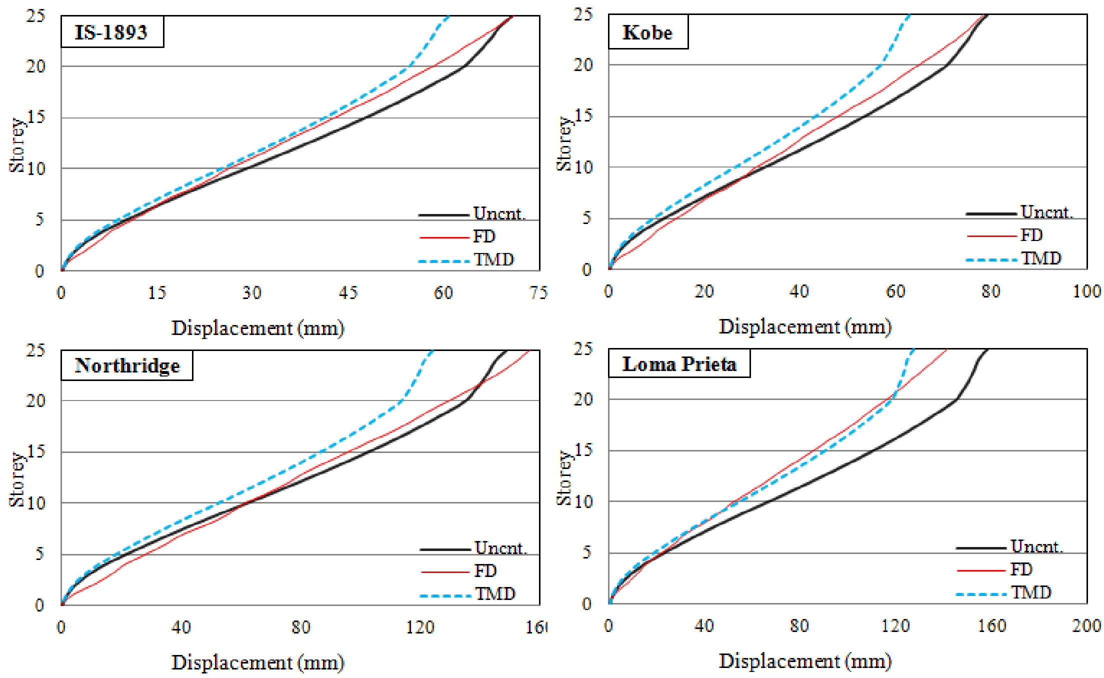


Figure 7.3: Comparison of Displacement for Response Spectrum Analysis

Tuned mass damper reduces the maximum top storey displacement by 14%, 20.6%, 16.4% and 19.3% for IS-1893, Kobe, Northridge and Loma Prieta response spectra respectively. While for friction damper top storey displacement is more or less similar to uncontrolled structure except Loma Prieta response spectra. From results, it is evaluated that friction damper do not give satisfactory performance for 25-storey wall frame structure, while for tuned mass damper there is substantial reduction in top storey displacement..

7.2.2 Comparison of Storey Drift

From results, it is evident that storey drift is excessive at near about 10^{th} storey level, so comparison of storey drift is done at 10^{th} storey level. The graphical representations of comparison of storey drift for uncontrolled and controlled structure are illustrated in Figure 7.4. It is clear from graphs that storey drift is continuously decreased by attaching TMD to structure. Decreasing in drift at 1^{st}, 4^{th}, 7^{th}, 10^{th}, 13^{th}, 16^{th}, 19^{th}, 22^{nd} and 25^{th} storey is more where, friction damper is installed, as compared to other storey.

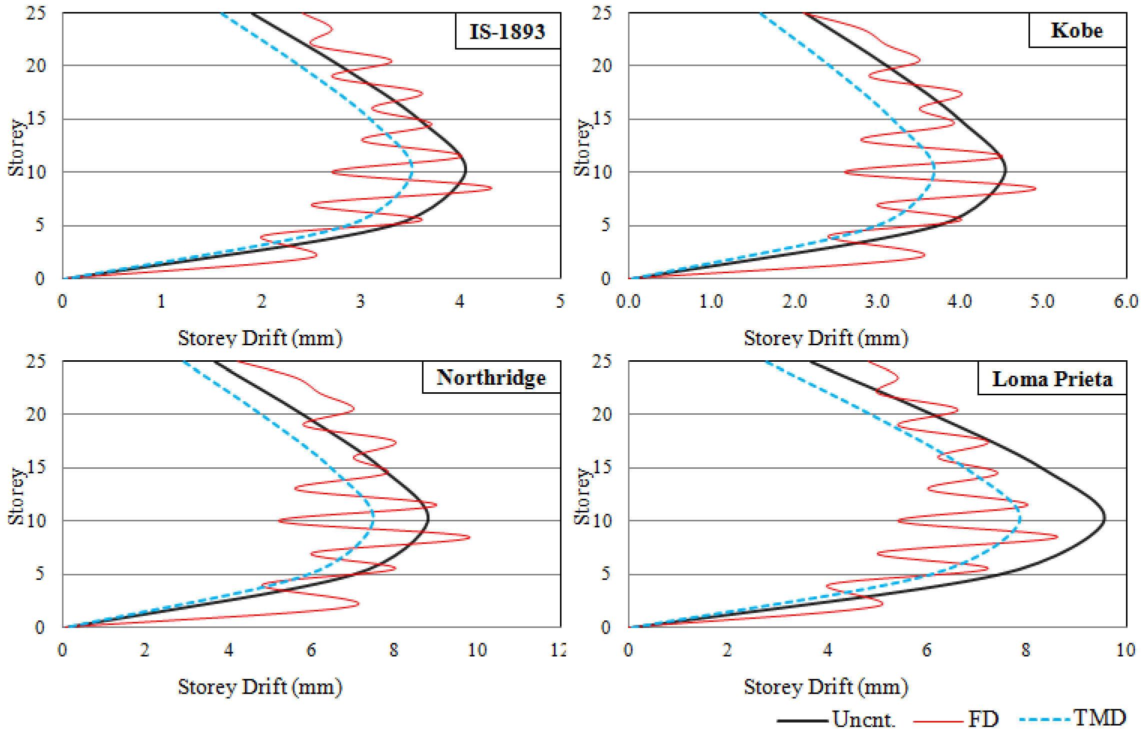


Figure 7.4: Comparison of Storey Drift for Response Spectrum Analysis

Tuned mass damper reduces the maximum storey drift by 13.2%, 18.6%, 17.7% and 15.1% for IS-1893, Kobe, Northridge and Loma Prieta response spectra. While friction damper maximum storey drift is increased. From graphs it can be observed that by use of tuned mass damper there is substantial reduction in storey for all earthquake.

7.2.3 Comparison of Time Period and Frequency

The comparison of mode Vs time period and frequency for first 5-mode are shown in Figure 7.5 for uncontrolled and controlled structure. For tuned mass damper time period of first mode increased by 4.3%, while for friction damper it is decreased by 7%. Frequency is inversely proportional to the time period, so for tuned mass damper it is reduced by 5% and for friction damper it is increased by 7.3%.

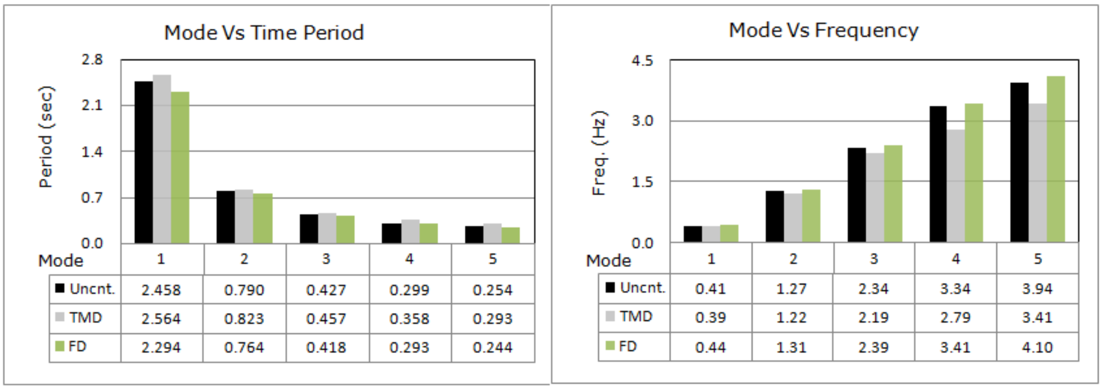


Figure 7.5: Mode Vs Time Period and Frequency

7.3 Results of Fast Nonlinear Time History Analysis (FNA)

The Fast Nonlinear time history Analysis (FNA) is done using SAP2000 for El Centro, Kobe, Northridge and Loma Prieta acceleration time histories. Results of FNA in form of hysteric loop for friction damper, energy function, displacement and acceleration time history and response spectra developed from time history are discussed here.

7.3.1 Hysteric Loop for Friction Damper

The hysteric loop of friction element located at 4th floor is shown in Figure 7.6 for various acceleration time histories. The maximum displacement in the damper is 1.45mm, 1.5mm, 2.1mm and 4.1mm for El Centro, Kobe, Northridge and Loma Prieta respectively. In Loma Prieta and Northridge earthquake maximum energy is released as compared to El Centro and Kobe earthquake, because area of hysteric loop is more for these earthquakes. So friction damper gives better results under Loma Prieta and Northridge earthquake.

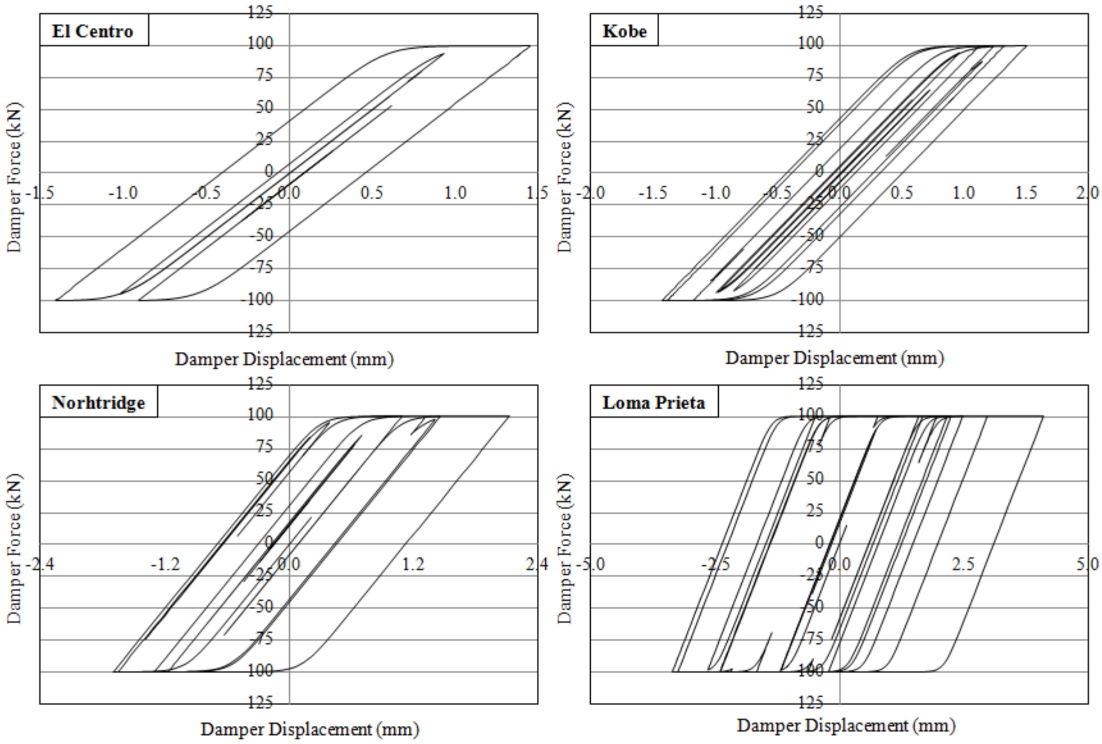


Figure 7.6: Hysteric loop for 100kN Diagonal Friction Damper

7.3.2 Comparison of Energy Function

Envelope of seismic energy input and energy dissipated by friction dampers and tuned mass damper is presented in Figure 7.7. From results it can be said that seismic energy dissipated by friction damper is 18%, 16%, 16% and 14%, while for tuned mass damper it is 27%, 21%, 38% and 33% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. It is seen from results that, tuned mass damper is more effective than friction damper for all earthquakes

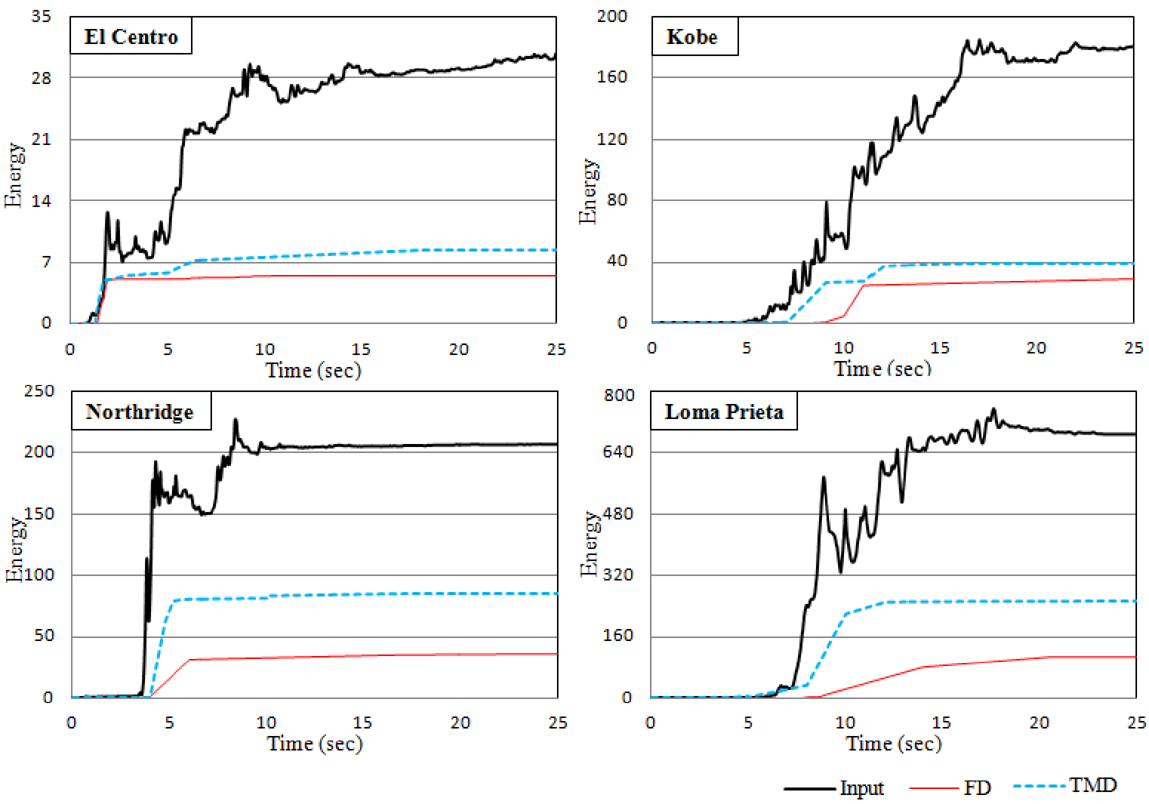


Figure 7.7: Energy Function for various earthquake

7.3.3 Comparison of Acceleration Response Spectra

Figure 7.8 represent comparison of acceleration response spectra for various earthquakes at level of top storey. From results it is observed that for friction damper, acceleration is reduced by 10%, 17%, 7% and 1% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. For tuned mass damper acceleration is decreased for all earthquake i.e. 28%, 16%, 9% and 14% for El Centro, Kobe, Northridge and Loma Prieta earthquake respectively. From results, it can be said that TMD is more effective for reducing the acceleration while FD is less effective.

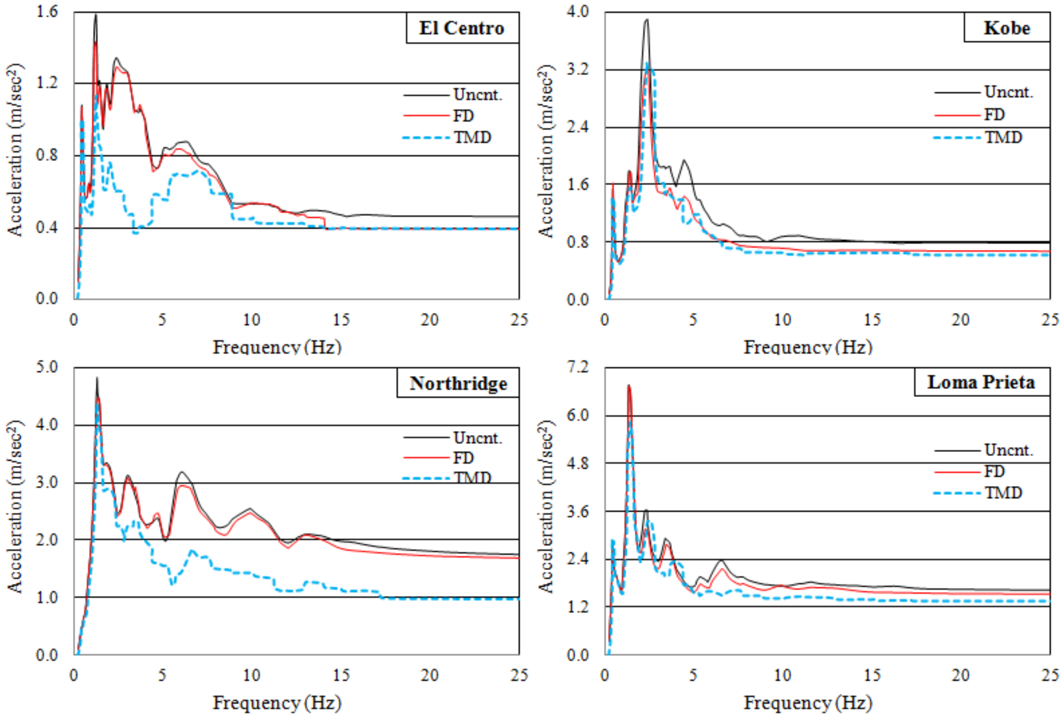


Figure 7.8: Acceleration Response Spectra

7.3.4 Comparison of Acceleration Time History

The comparisons of tip acceleration of structure for various earthquakes are shown in Figure 7.9. A friction damper of 100kN slip load is attached to the structure at 1^{st}, 4^{th}, 7^{th}, 10^{th}, 13^{th}, 16^{th}, 19^{th}, 22^{nd} and 25^{th} storey level in central bay on all outer periphery of building. A tuned mass damper of 204.18 ton is attached to top storey of structure. The comparison of tip acceleration of structure are illustrated in Table 7.4.

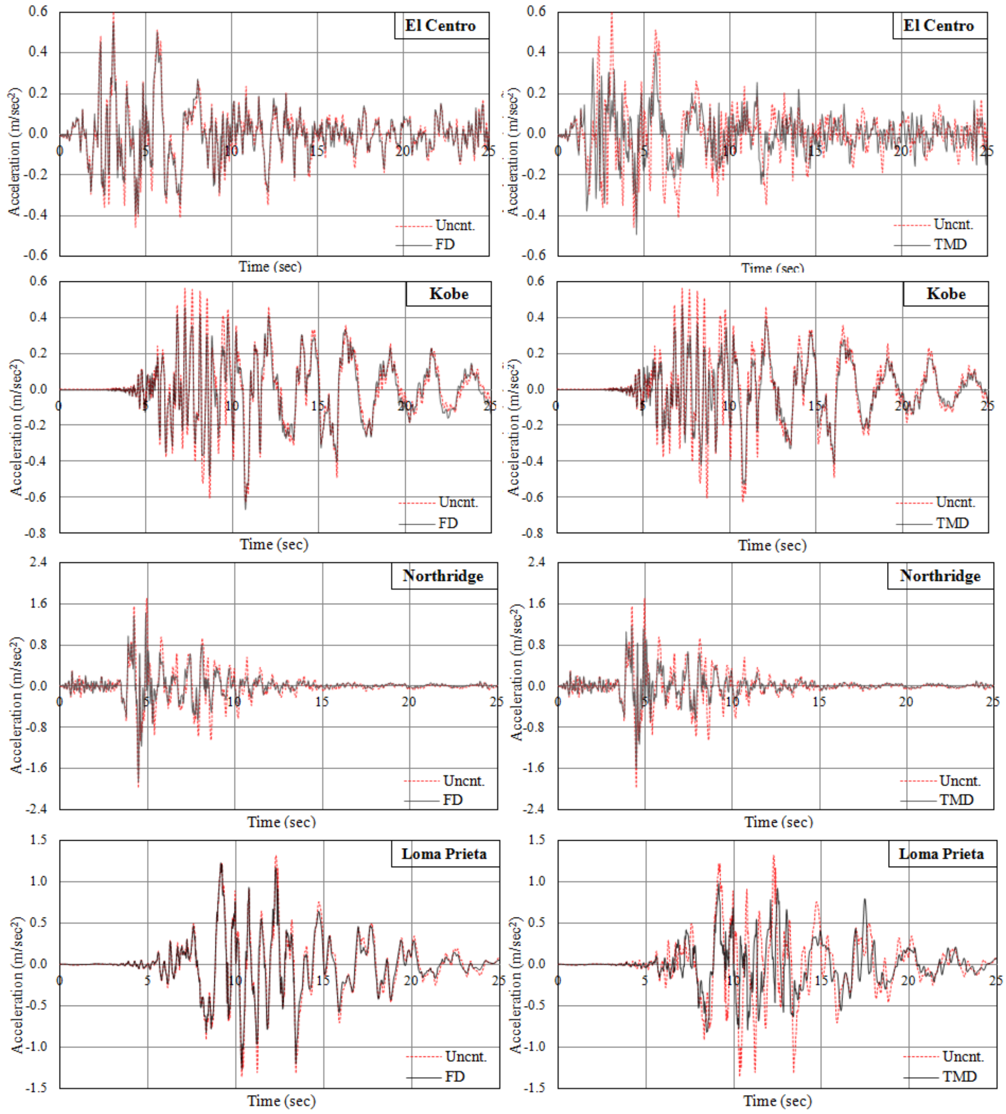


Figure 7.9: Acceleration Time Histories Recorded at level of Storey-15

It is shown from results that, tuned mass damper is more effective for reducing acceleration of structure as compared to friction damper. There is a reduction of 10.9%, 5.2%, 11% and 7.2% for friction damper and 15.1%, 16.2%, 25.3% and 33.1% for tuned mass damper for El Centro, Kobe, Northridge and Loma Prieta earthquakes respectively. It can be said that tuned mass damper surpassed friction damper in their ability to reduce the intensity of strong ground motion for all earthquakes.

Table 7.4: Maximum Acceleration of 25-Storey Structure in m/sec^2

Earthquake	*Uncnt.*	*FD*	*% Reduction*	*TMD*	*% Reduction*
El Centro	0.530	0.472	**10.9**	0.450	**15.1**
Kobe	0.593	0.562	**5.2**	0.497	**16.2**
NR	1.841	1.639	**11.0**	1.375	**25.3**
LP	1.342	1.246	**7.2**	0.898	**33.1**

7.3.5 Comparison of Displacement Time History

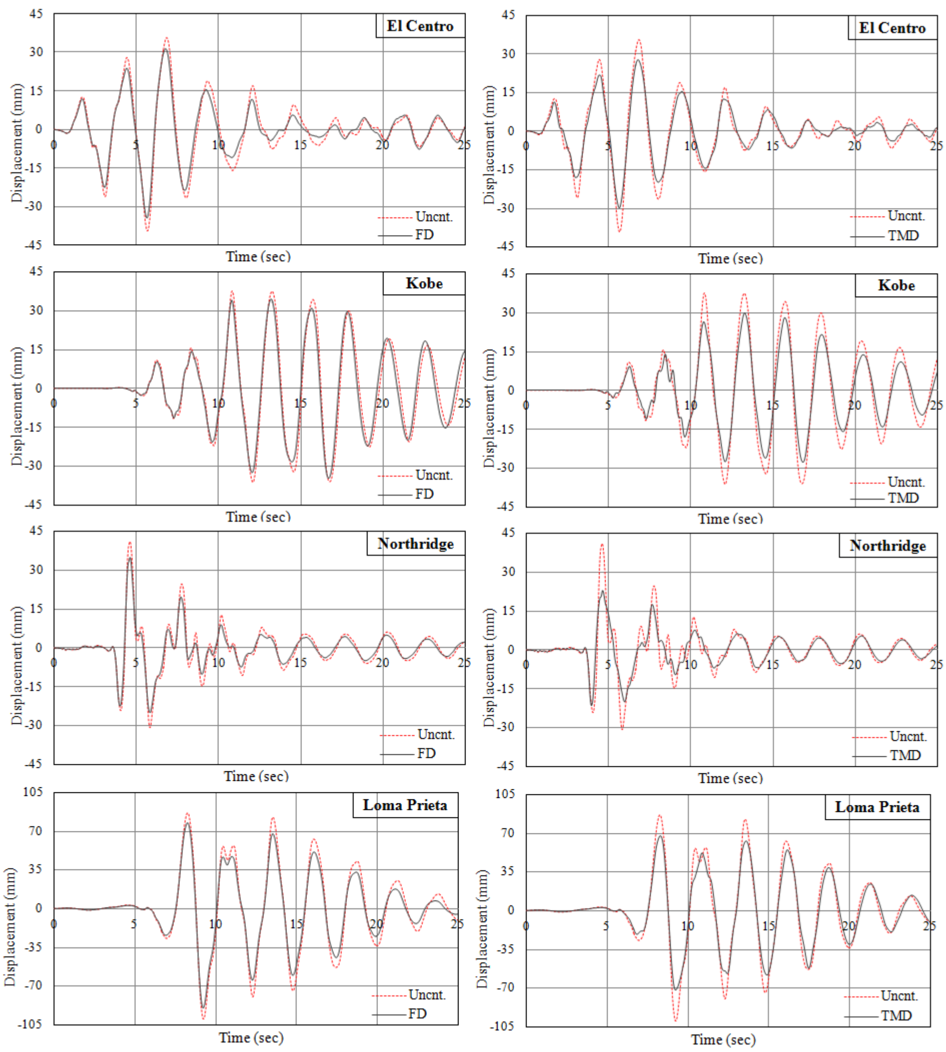


Figure 7.10: Displacement Time Histories Recorded at level of Storey-25

Figure 7.10 illustrate the tip displacement of the structure with diagonal friction damper and tuned mass damper with uncontrolled structure obtained under four earthquake excitations. The comparison of maximum tip displacement of this structure experienced for four earthquake excitations are presented in Table 7.5.

Table 7.5: Maximum Displacement of 25-Storey Structure in mm

Earthquake	*Uncnt.*	*FD*	*% Reduction*	*TMD*	*% Reduction*
El Centro	37.50	32.83	**12.5**	28.81	**23.2**
Kobe	36.85	34.35	**6.8**	28.69	**22.1**
NR	35.85	29.86	**16.7**	22.13	**38.3**
LP	92.96	83.46	**10.2**	69.41	**25.3**

From results it is evaluated that for friction damper there is maximum reduction in tip displacement is 12.5%, 6.8%, 16.7% and 10.2%, while for tuned mass damper it reduced by 23.2%, 22.1%, 38.3% and 25.3% for El Centro, Kobe, Northridge and Loma Prieta respectively. Tuned mass damper is more effective then friction damper for reducing the tip displacement of structure.

7.4 Summary

Nonlinear time history and response spectrum analysis of 25-storey RC wall-framed structure is carried out for various time histories and response spectra by using friction damper and tuned mass damper and they are compared to uncontrolled structure to understand the behavior of damping system to improve the response of building during earthquake.

Results of response spectrum analysis shows that the influence in reduction of displacement and storey drift are less when using friction damper. For tuned mass damper variation in displacement and storey drift is constant, while for friction damper it is fluctuated where damper is attached.

It is shown from results of Fast Nonlinear time history Analysis of 25-storey RC wall-framed structure; tuned mass damper surpassed the diagonal friction damper in their ability to reduce the intensity of strong ground motion for all earthquakes. For seismic energy dissipation tuned mass damper is more effective as compared to friction damper.

Chapter 8

Parametric Study, Results and Discussions

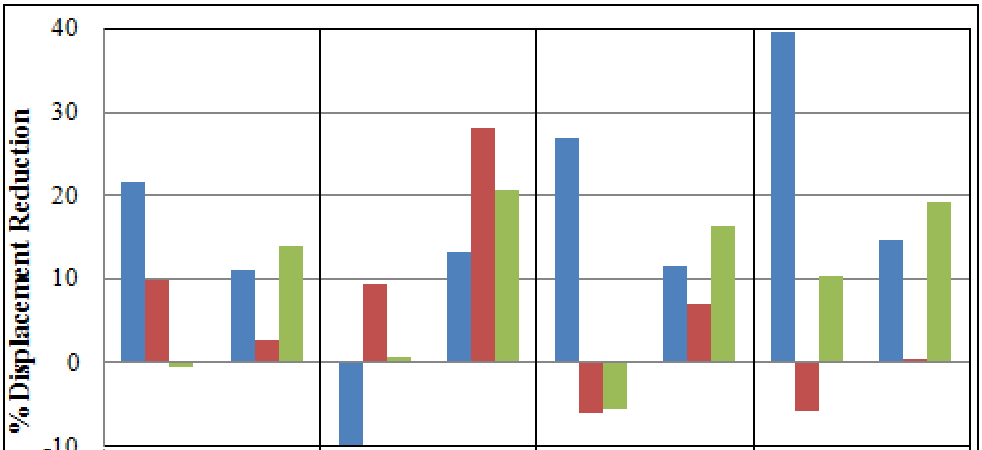
In order to control the vibration response of medium and high-rise structures during earthquakes, passive damper as energy dissipation devices are mostly used. Determining the type of damping devices and their optimal placement and size is highly an iterative trial and error process for different earthquakes. What makes the problem even more difficult is the uncertainty of seismic inputs as the force of nature can vary tremendously. The ranges of the results presented in this study illustrated the complexity of the problem of optimization in the use of damping systems.

This study has investigated the use of friction damper and tuned mass damper to mitigate the seismic input energy within medium and high-rise structures. These damping devices were embedded in a variety of different placement (one at a time). The appropriate damper for reducing the seismic response of structure is finding out for medium and high-rise structure. Parametric study is carried out by varying location and type of damper. Response of building under four seismic excitations are studied in terms of displacement, storey drift, energy dissipation capacity of damper and acceleration of structure.

8.1 Results and Discussion Based on Storey Height

This section provides the comparison of results for study carried out on 5-storey, 15-storey and 25-storey RC structure for Friction and Tuned Mass Damper, for finding the appropriate damping devices for medium and high-rise structures.

The percentage of reductions in tip deflection experienced by the structures embedded with diagonal friction damper and tuned mass damper under four earthquakes are presented in Figure 8.1. The results are compiled for response spectrum analysis.



	IS-1893		Kobe		Northridge		Loma Prieta	
Storey	FD	TMD	FD	TMD	FD	TMD	FD	TMD
5	21.6	11.1	-10	13.2	26.8	11.4	39.7	14.6
15	9.9	2.6	9.3	28	-6	7	-5.8	0.5
25	-0.6	14	0.8	20.6	-5.5	16.4	10.2	19.3

Figure 8.1: Percentage Reduction in Tip Deflection under different Earthquakes

As it can be seen from this figure, the friction damper displayed extraordinary performance under El Centro, Northridge and Loma Prieta earthquake excitations for 5-storey structure. On the other hand, in the case of all four earthquake excitations, tuned mass damper gives the better performance under 25-storey structure. The best performance occurred under the Loma Prieta earthquake with tip deflection reduction of 39.7% for friction damper. The reduction experienced by tuned mass damper is increased when height of building is increased, under El Centro, Northridge and Loma Prieta earthquake. Friction damper gives better performances for lower height of structures under most of the earthquake.

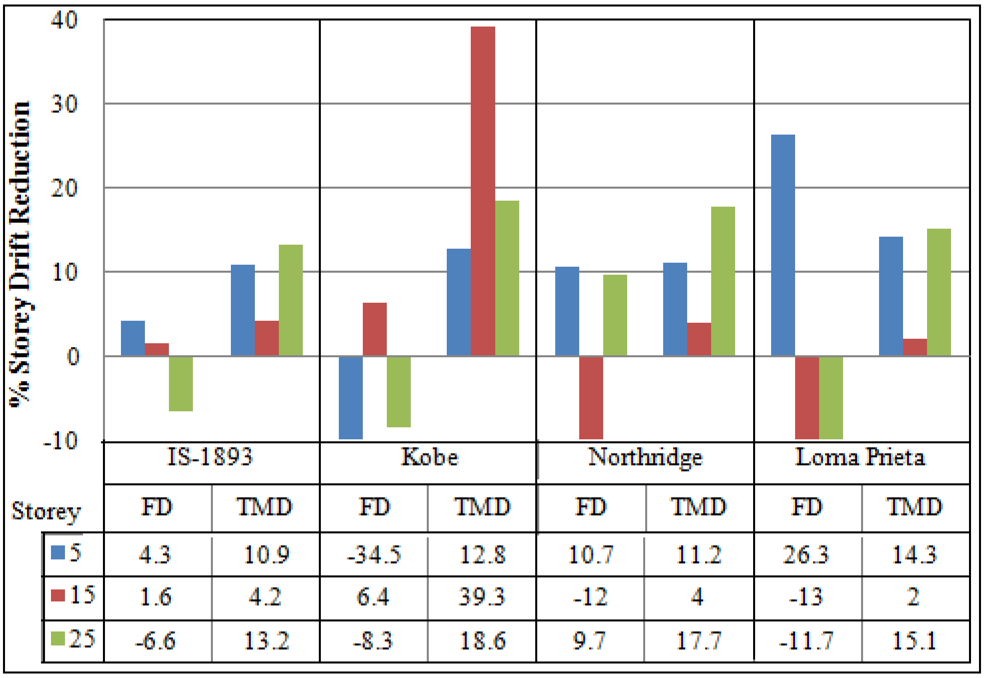


Figure 8.2: Percentage Reduction in Maximum Storey Drift under different Earthquakes

The results of same structure in terms of maximum storey drift reduction are illustrated in Figure 8.2. The structure experienced wide range of results. The highest storey drift reduction of 39.3% was achieved under the Kobe earthquake for 15-storey structure embedded with tuned mass damper. The reduction recorded under the El Centro, Kobe and Loma Prieta earthquake were clearly unfavorable for 25-storey structure attached with friction damper. On the other hand performance of tuned mass damper is enhanced with increased in height of structure. The reduction observed for the tuned mass damper is quite satisfactory than the friction damper for major earthquakes.

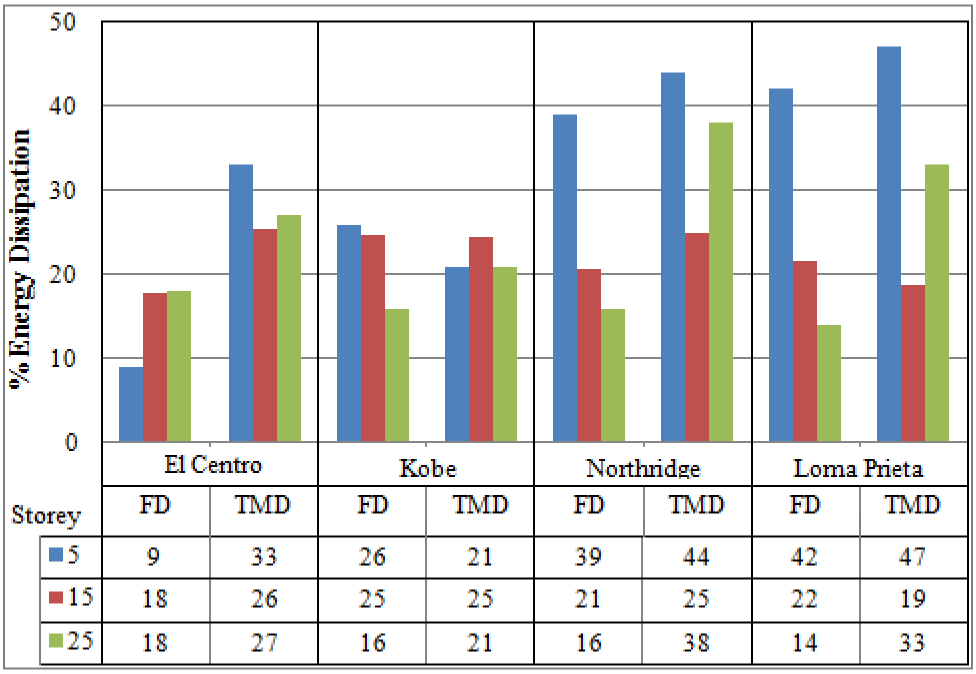
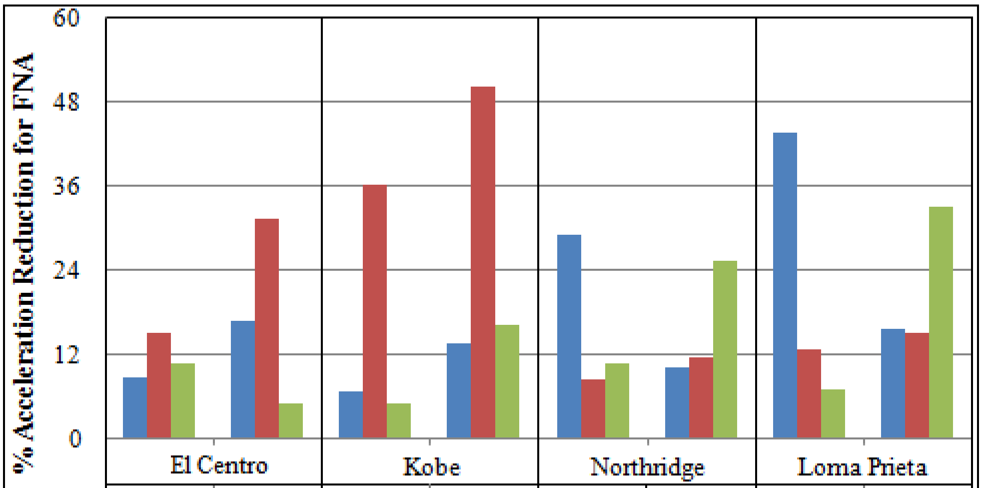


Figure 8.3: Percentage of Energy Dissipation under different Earthquakes (FNA)

The energy dissipation by friction damper and tuned mass damper under four earthquakes excitations are displayed in Figure 8.3. The energy dissipation capacity of friction damper is decreased when the height of structure under all earthquake except Kobe earthquake. In case of tuned mass damper energy dissipation capacity was decreased for 15-storey, but it is increased for 25-storey structure. The maximum energy dissipation of 47% is observed under the Loma Prieta earthquake for structure embedded with tuned mass damper.

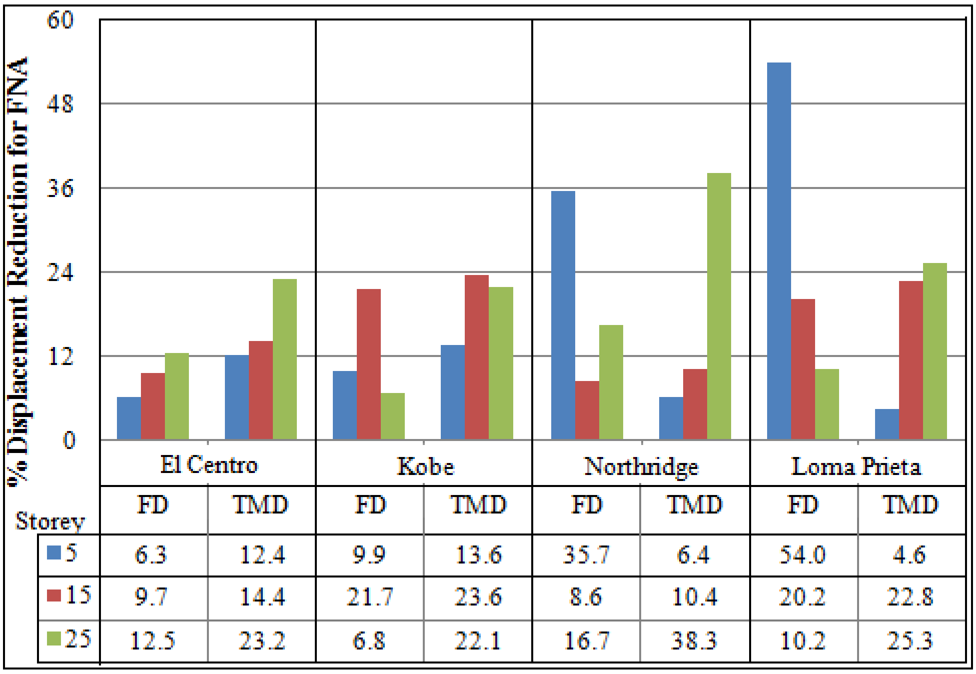
The percentage reduction in peak values of the tip acceleration for the structures fitted with the diagonal friction damper and tuned mass damper are shown in Figure 8.4. Significant tip acceleration reductions of 50.4% were experienced under Kobe earthquake for 15-storey structure fitted with tuned mass damper. The reduction in tip acceleration is increased with height of structure increased under Northridge and Loma Prieta earthquake excitations. While in case of El Centro and Kobe earthquake it is inconsistent. Both the damping systems give quite satisfactory results for reduction in tip acceleration of structures. In terms of tip acceleration reduction, the efficiency of the dampers was rather inconsistent without any obvious trends.



Storey	El Centro FD	El Centro TMD	Kobe FD	Kobe TMD	Northridge FD	Northridge TMD	Loma Prieta FD	Loma Prieta TMD
5	8.9	17.0	6.8	13.8	29.1	10.4	43.6	15.6
15	15.0	31.5	36.3	50.4	8.6	11.8	12.9	15.1
25	10.9	5.1	5.2	16.2	11.0	25.3	7.2	33.1

Figure 8.4: Percentage Reduction in Tip Acceleration under different Earthquakes (FNA)

Figure 8.5 shows the tip deflection reduction for the structure embedded with the diagonal friction damper and tuned mass damper for time history analysis. The highest tip deflection reduction of 54% occurred under Loma Prieta earthquake excitation for 5-storey structure integrated with friction damper. The performance of friction damper is inconsistent with increasing height of structure, but in case of tuned mass damper performance of tuned mass damper is enhanced with increasing height of structure under all earthquake. The nature of earthquake is also affecting the results of damping systems. Both the damper are gives the quite satisfactory results for reduction of tip deflection in all earthquake excitations.



Storey	El Centro FD	El Centro TMD	Kobe FD	Kobe TMD	Northridge FD	Northridge TMD	Loma Prieta FD	Loma Prieta TMD
5	6.3	12.4	9.9	13.6	35.7	6.4	54.0	4.6
15	9.7	14.4	21.7	23.6	8.6	10.4	20.2	22.8
25	12.5	23.2	6.8	22.1	16.7	38.3	10.2	25.3

Figure 8.5: Percentage Reduction in Tip Deflection under different Earthquakes (FNA)

8.2 Parametric Study Based on Location of Damper

One of the main aims of this study was to investigate the efficiency of energy dissipating dampers in vibration control for variety of placements under different earthquake excitations and for this 5-storey RC framed structure is selected. For this purpose five different damper placements were selected to study the influence of location on the seismic response of these models. These models were designated by case-1, case-2, case-3, case-4 and case-5. As can be seen Figure 8.6, the designating numbers correspond to location of the storey at which friction dampers were placed. The friction dampers were placed on all outer periphery of building, here in Figure 8.6 only one outer periphery is shown and other are similar to these. The tuned mass damper is placed 1^{st}, 2^{nd}, 3^{rd}, 4^{th} and 5^{th} storey in case-1 to case-5 respectively. The undamped structure was also analyzed in order to compare results. The properties of friction damper and tuned mass damper are same as mentioned in Table 5.2 and 5.3.

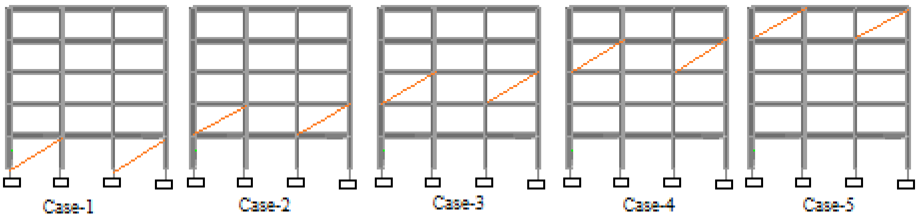
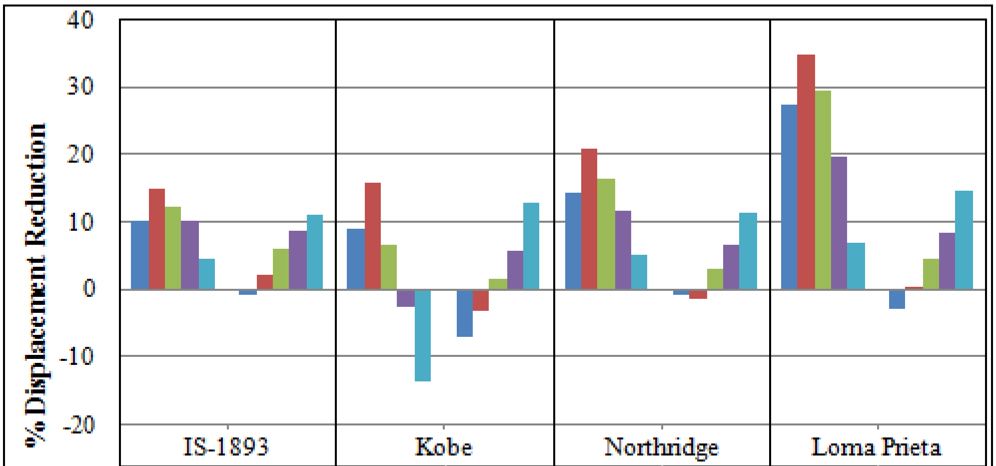


Figure 8.6: Placement of Friction Dampers within 5-Storey Structure

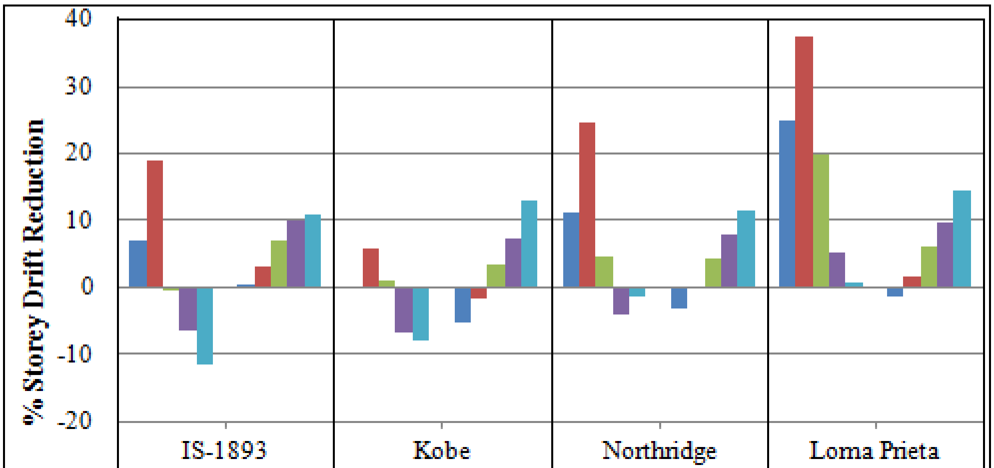


	IS-1893 FD	IS-1893 TMD	Kobe FD	Kobe TMD	Northridge FD	Northridge TMD	Loma Prieta FD	Loma Prieta TMD
Case-1	10.4	-0.6	9.2	-6.9	14.6	-0.6	27.6	-2.9
Case-2	15.1	2.3	16.0	-3.2	20.9	-1.2	34.8	0.4
Case-3	12.3	6.0	6.7	1.7	16.5	3.1	29.7	4.7
Case-4	10.4	8.9	-2.5	5.7	11.8	6.6	19.7	8.4
Case-5	4.7	11.3	-13.4	12.9	5.1	11.5	6.9	14.7

Figure 8.7: Percentage Reduction in Tip Deflection for Different Damper Placements

The results of tip deflection reduction for structure embedded with diagonal friction damper and tuned mass damper in terms of damper placement are shown in Figure 8.7 for response spectrum analysis.

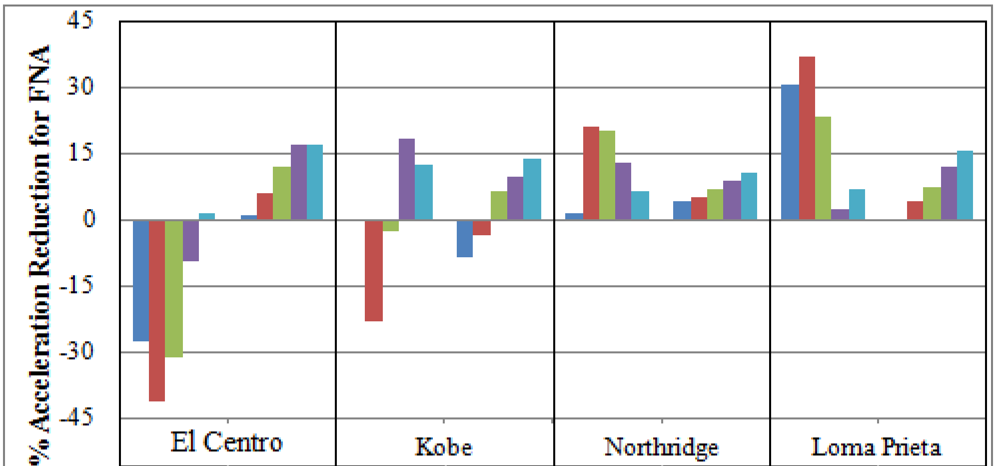
For tuned mass damper placement, maximum tip deflection reduction occurred under the Loma Prieta earthquake was increasing when the damper moved towards top of the structure. In case of tuned mass damper, clearly the poorest results were obtained when damper is moved towards the base of structure. The tuned mass damper gives the superior performance when it is placed near or at top of the structure. In terms of efficiency of different placement of friction damper, the greatest tip deflection reduction occurred when dampers were placed in storeys 1 to 3. In terms of efficiency of different damper, the maximum tip deflection reduction is occurred for structure located with friction damper under Loma Prieta earthquake. From results, it can be said that friction damper gives better performance under most of earthquakes for 5-storey RC framed structure.



	IS-1893 FD	IS-1893 TMD	Kobe FD	Kobe TMD	Northridge FD	Northridge TMD	Loma Prieta FD	Loma Prieta TMD
Case-1	6.9	0.4	-0.2	-5.3	11.0	-3.4	24.8	-1.6
Case-2	18.9	3.1	5.7	-1.7	24.5	-0.2	37.4	1.6
Case-3	-0.6	6.8	0.9	3.2	4.5	4.2	19.6	5.9
Case-4	-6.7	9.8	-6.8	7.1	-4.3	7.7	5.1	9.5
Case-5	-11.7	10.9	-8.2	12.9	-1.3	11.3	0.5	14.3

Figure 8.8: Percentage Reduction in Maximum Drift for Different Damper Placements

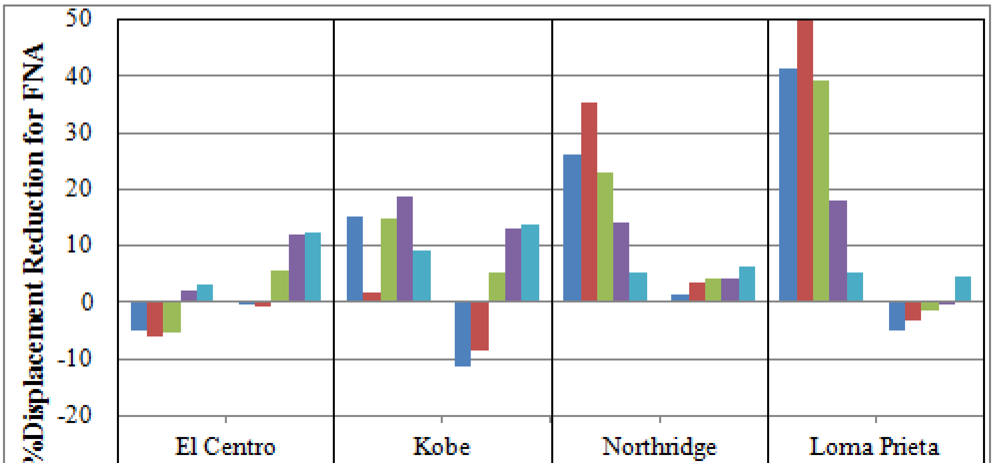
The results of maximum storey drift reduction in terms of damper placement are illustrated in Figure 8.8. The best performance occurred when the friction dampers were placed in the 2^{nd} storey, while their subsequent repositioning towards the top storeys responded in a gradually decrease in maximum drift reduction under all earthquake excitations. So it can be said that the performance of friction damper is increased, when it is placed near the maximum drift region. The results experienced under all earthquake excitations confirmed the best performance experienced by the tuned mass damper when it is placed at top storey. But when it was moved towards the lowest storey a gradual reduction in their efficiency was experienced. From results it can be said that friction damper gives better performance under Northridge and Loma Prieta earthquakes while for El Centro and Kobe earthquake poor results was obtained.



	El Centro FD	El Centro TMD	Kobe FD	Kobe TMD	Northridge FD	Northridge TMD	Loma Prieta FD	Loma Prieta TMD
Case-1	-27.8	1.0	-0.1	-8.5	1.7	4.2	30.4	-0.1
Case-2	-41.4	6.1	-23.0	-3.4	21.0	5.0	37.1	4.4
Case-3	-31.3	12.0	-2.7	6.3	20.1	7.1	23.2	7.4
Case-4	-9.4	16.8	18.5	9.6	12.7	8.6	2.6	11.9
Case-5	1.4	17.1	12.4	13.8	6.3	10.4	7.1	15.6

Figure 8.9: Percentage Reduction in Tip Acceleration for Different Damper Placements (FNA)

The results of percentage reductions in tip acceleration in terms of damper placement are shown in Figure 8.9. In terms of tip acceleration reduction at different placement the friction damper achieved highest reduction when it placed at 2^{nd} storey, while their moving towards the top of the structure caused decreases in tip acceleration reductions for all earthquakes except El Centro earthquake. For tuned mass damper placement, maximum tip acceleration reduction occurred under the El Centro earthquake when it is placed at top of the structure, while their moving towards the base of the structure tip acceleration reduction is decreased. It is said that friction damper perform well in all earthquakes except El Centro earthquake.



	El Centro		Kobe		Northridge		Loma Prieta	
	FD	TMD	FD	TMD	FD	TMD	FD	TMD
Case-1	-5.0	-0.4	15.1	-11.5	26.2	1.3	41.1	-5.2
Case-2	-6.0	-0.9	1.8	-8.6	35.2	3.5	49.9	-3.2
Case-3	-5.3	5.7	14.7	5.3	22.9	4.0	39.1	-1.5
Case-4	2.0	11.9	18.7	12.9	14.0	4.2	18.0	-0.5
Case-5	3.0	12.4	9.2	13.5	5.0	6.4	5.3	4.6

Figure 8.10: Percentage Reduction in Tip Deflection for Different Damper Placements (FNA)

The results of tip deflection reduction for structure embedded with diagonal friction damper and tuned mass damper in terms of damper placement are shown in Figure 8.10 for time history analysis. The same trend is observed in reduction in displacement like as acceleration. Friction damper is achieved highest reduction in Loma Prieta earthquake when it is placed at 2^{nd}storey, while tuned mass damper achieved highest reduction in Kobe and El Centro earthquakes when it is placed at top of the structure.

8.3 Summary

In this chapter, the behavior of medium and high-rise structure is investigated under four seismic excitations for friction damper and tuned mass damper. Results of medium and high-rise structure are compared for choosing the appropriate damping system for appropriate height of structure. Parametric studies were carried out based on location of damper placement.

Friction damper gives the better performance for 5-storey structures for reducing the dynamic response of building, but for 15-storey and 25-storey structure response of structure is considerably decreases with increasing in height. In terms of displacement, interstorey drift, acceleration and response spectra friction damper gives significant response, but in case of energy dissipation tuned mass damper gives the incredible performance.

Tuned mass damper are most effective when it is placed at top of the structure, while friction dampers are most effective when placed close to regions of maximum storey drift. From results, it can be said that for medium height structure friction damper should be used, while for high-rise structure tuned mass damper should be preferable.

Chapter 9

Summary and Conclusions

Conventional design permits inelastic action on buildings, providing significant energy dissipation, but often resulting in important damage to structural members and nonstructural elements when the building is subjected of dynamic loads. In response to these shortcomings several counter measures have been developed, one of them is supplement passive energy dissipation system, which is considered for this study. The supplementary passive energy dissipation devices are known to be effective in reducing the earthquake-induced response of structural systems.

A strategy for protecting building from earthquakes is to limit the tip deflection, which provides an overall assessment of the seismic response of the structure. Different building structures require different damping systems for the best results. However, the present study demonstrated that some trends common for all investigated structures can be observed.

9.1 Conclusions

Three medium to high-rise structures embedded with two damping systems were investigated under four different seismic excitations. Each of these damping systems performed in a different manner and also their performance varied considerably when treated in different structures. It is important to note that results obtained by this work were based on Pall Dynamics for friction damper and improved Frahm vibration absorber for tuned mass damper, which is explained by T.T. Soong and Rahul Rana. Based on the work carried out following conclusions are made.

a. The performance of friction damper and tuned mass damper are dependent on the characteristics of earthquake ground motion, so four earthquakes are considered for this study.

b. It is possible to achieve seismic mitigation, under all earthquake excitations, for all the structures considered in this study, by using appropriate damper types suitably located within the structure.

c. Friction dampers are most effective when placed close to regions of maximum storey drift, while the best performance of tuned mass damper was achieved when placed at top storey.

d. Friction damper gives the significant performance for medium-rise structures for reducing the dynamic response of building in terms of displacement, interstorey drift, acceleration and response spectra but in case of energy dissipation tuned mass damper gives the incredible performance.

e. Tuned mass damper gives the significant performance with increasing the height of structure, so for high-rise structure tuned mass damper is preferred instead of friction damper.

f. The time period of structure embedded with tuned mass damper is increased because of not proper tuning of mass in medium-rise structure, while for high-rise structure time period of structure is more or less similar to the uncontrolled structure.

g. Numerical results of time history analysis have shown that friction damper and tuned mass damper both are quite effective in reducing the structural dynamic response in the form of acceleration, displacement and storey drift.

9.2 Future Scope of Work

a. The method of optimizing the location and size of the dampers within the structure is developed.

b. The study can be extended by involving the Metallic, Tuned Liquid, Magnetorheological fluid and Shape Memory Alloy damper

c. Explore an active, semi-active and hybrid damping techniques and their applications in civil engineering structures. Each and every techniques has their own advantages compared to passive damping techniques.

d. The performance based analysis is carried out using various damping devices to achieve the desired performance.

e. The effect of combination of damping systems is investigated instead of one damping system at a time.

f. Conduct a study on applications of passive dampers in other type of structures such as Bridges, Steel Buildings, Chimney and Water tanks.

g. Examine the effectiveness of passive damping techniques in other aspects such as economical aspects in terms of cost-effectiveness.

Appendix A

Design of Tuned Mass Damper

A.1 Structural Data

The structural layout of 5-storey RC frame building considered for the modeling and analysis are shown in Figure A.1. The building undertaken was symmetrical in plan, to avoid the case of non-proportional damping in a structure. The Tuned Mass Damper is installed at top of roof storey. The approximate sizes of various elements is shown in Table A.1

Table A.1: Approximate Sizes of Members for 5-Storey Building

Sr. No	*Element*	*Notation*	*Size (mm)*
1	Main Beam	B1	300 × 500
2	Secondary Beam	B2	230 × 450
3	Column	C1	300 × 450
4	Column	C2	450 × 300
5	Slab	S1	125

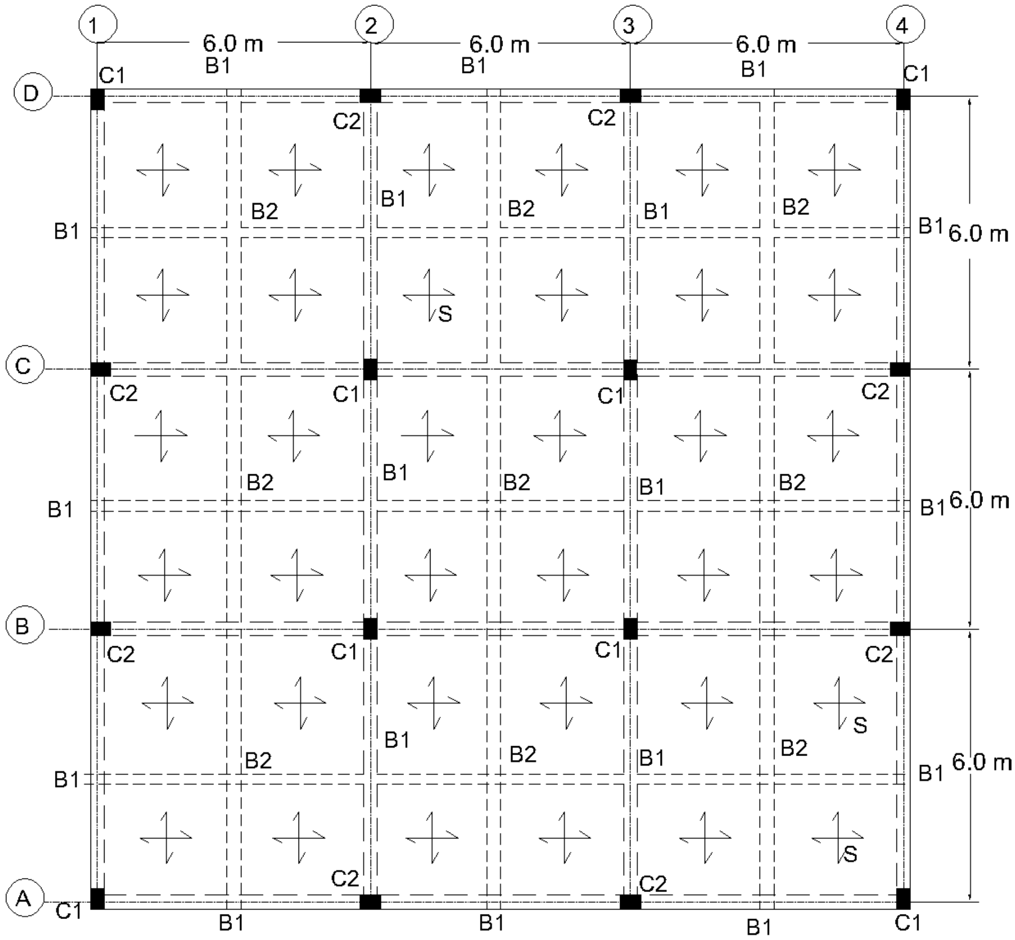


Figure A.1: Plan of 5-Storey RC Framed structure

A.2 Calculation of Earthquake Force

The earthquake force is calculated as per IS:1893 (Part-I):2002, which is described as below.

A.2.1 Calculation of Lumped Weight of Floor

Weight of slab	=	1012.5 kN/floor
Weight of beam	=	765.5 kN/floor
Weight of column	=	162.0 kN/floor
Weight of wall	=	550.8 kN/floor
Floor finish load	=	324 kN/floor
Live load on floor	=	972 kN/floor
Typical floor weight	=	3786.8 kN
Roof storey weight	=	3025.4 kN
Total weight of building	=	18172.6 kN

A.2.2 Calculation of Seismic Force

Live load factor	=	0.25
Seismic weight of building, W	=	DL + 0.25 × LL
	=	14770.7 kN
Time Period, $T_a = 0.075H^{0.75}$	=	0.572 sec
Design horizontal seismic coefficient,		
$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{Sa}{g}$	=	0.0856
Design base Shear, $V_B = A_hW$	=	1265kN
Distribution of Seismic force, Q_i	=	$V_B \frac{W_ih_i^2}{\sum_{j=1}^{n} W_jh_j^2}$

Table A.2: Distribution of Seismic Force

Storey	$W_i(kN)$	$h_i(m)$	$W_ih_i^2$	$Q_i(kN)$	$V_i(kN)$
5	2539.35	15	571354	517.4	517.4
4	3057.75	12	440316	398.7	916.1
3	3057.75	9	247678	224.3	1140.4
2	3057.75	6	110079	99.7	1240.1
1	3057.75	3	27520	24.9	1265.0
Total	14770.35		1396946	1265.0	

A.3 Calculation of Wind Force

The wind force is calculated as per IS:875 (Part-3):1984, which is described as below.

Risk Coefficient, k1	=	1.0
Structure size factor, k2	=	as per Table A.3
Topography factor, k3	=	1.0
Basic wind speed, V_b	=	50 m/sec
Design wind speed, V_z	=	$V_b \times k1 \times k2 \times k3$
Design wind pressure, P_z	=	$0.6V_z^2$

Table A.3: k2 Factor

Storey	*K2*	*Vz (m/sec)*	$Pz(kN/m_2)$
1	1.00	50.0	1.500
2	1.00	50.0	1.500
3	1.00	50.0	1.500
4	1.02	51.0	1.561
5	1.05	52.5	1.654

Length of building	=	width of building;	a = b
a/b ratio	=	1.0 and h/b ratio	= 0.83
Force coefficient, C_f	=	1.2	
Wind Force, F	=	$C_f \times Area \times P_d$	

Table A.4: Distribution of Wind Force

Storey	*Wind Force (kN)*		*Storey Shear (kN)*	
	X-dir.	*Y-dir.*	*X-dir.*	*Y-dir.*
1	97.2	97.2	446.3	446.3
2	97.2	97.2	349.1	349.1
3	97.2	97.2	251.9	251.9
4	101.1	101.1	154.7	154.7
5	53.6	53.6	53.6	53.6
	446.3	446.3		

From TableA.2 and TableA.4, it is clearly seen that earthquake force is critical for analysis. Hence earthquake force is considered in analysis and design of this building, while wind force is neglected.

A.4 Calculation of Tuned Mass Damper Parameters

$$\begin{aligned}\text{Stiffness of each storey} &= n \times \frac{12EI}{L^3} \\ &= 221827.6 \text{ kN/m}\end{aligned}$$

$$\text{Stiffness matrix [k],} = \begin{vmatrix} 443655 & -221828 & 0 & 0 & 0 \\ -221828 & 443655 & -221828 & 0 & 0 \\ 0 & -221828 & 443655 & -221828 & 0 \\ 0 & 0 & -221828 & 443655 & -221828 \\ 0 & 0 & 0 & -221828 & 221828 \end{vmatrix} \text{kN/m}$$

$$\text{Mass matrix [k],} = \begin{vmatrix} 312 & 0 & 0 & 0 & 0 \\ 0 & 312 & 0 & 0 & 0 \\ 0 & 0 & 312 & 0 & 0 \\ 0 & 0 & 0 & 312 & 0 \\ 0 & 0 & 0 & 0 & 259 \end{vmatrix} \text{ton}$$

By the help of MATLAB software eigen value and eigen vector are find out.

$$\text{Eigen value } [\omega_n^2] = \begin{vmatrix} 61.3 & 0 & 0 & 0 & 0 \\ 0 & 518.7 & 0 & 0 & 0 \\ 0 & 0 & 1271.1 & 0 & 0 \\ 0 & 0 & 0 & 2060.9 & 0 \\ 0 & 0 & 0 & 0 & 2638.3 \end{vmatrix}$$

$$\text{Eigen vector} = \begin{vmatrix} 0.010 & -0.027 & 0.034 & 0.030 & -0.017 \\ 0.019 & -0.034 & 0.007 & -0.027 & 0.030 \\ 0.027 & -0.016 & -0.033 & -0.006 & -0.033 \\ 0.032 & 0.013 & -0.014 & 0.033 & 0.027 \\ 0.035 & 0.033 & 0.030 & -0.023 & -0.013 \end{vmatrix}$$

It is clearly seen that value eigen vector of first mode at 5^{th} storey is maximum, hence the Tuned Mass Damper is installed at 5^{th} storey.

Time Period of mode shapes,

$T = 2\pi/\omega_n = \begin{vmatrix} 0.802 & 0.275 & 0.176 & 0.138 & 0.122 \end{vmatrix}$

Natural frequency of building without TMD

$f = 1/T = \begin{vmatrix} 1.24 & 3.62 & 5.67 & 7.22 & 8.17 \end{vmatrix}$

Mode shape of 1^{st} mode,

$\Phi_1 = \begin{vmatrix} 0.010 & 0.019 & 0.027 & 0.032 & 0.035 \end{vmatrix}$

$\Phi_1 = \begin{vmatrix} 0.292 & 0.558 & 0.777 & 0.928 & 1.000 \end{vmatrix}$

Modal mass of 1^{st} mode,

$M_1 = \Phi_1^T M \Phi_1$

$M_1 = 8234.25kN$

$$\begin{aligned}
&\text{Assume approximate weight of TMD} &&= 2\% \text{ of total weight of building} \\
&\text{Mass of Damper,m} &&= 2 \times 18172.4/100 = 363.45 \text{ kN} \\
&\text{Mass ratio, } \mu = \frac{M_1}{m} &&= \frac{363.45}{8234.25} \\
& &&= 0.044 = 4.4\%
\end{aligned}$$

Optimum value of Frequency,

$$f_{opt} = \frac{1}{1+\mu}\left(\sqrt{\frac{2-\mu}{2}}\right)$$

$$f_{opt} = \frac{1}{1+0.044}\left[\sqrt{\frac{2-0.044}{2}}\right] = 0.947$$

Damping Ratio for TMD

$$\xi_{opt} = \sqrt{\frac{3\mu}{8(1+\mu)}\left(\sqrt{\frac{2}{2-\mu}}\right)}$$

$$\xi_{opt} = \sqrt{\frac{3\times0.044}{8(1+0.044)}\left[\frac{2}{2-0.044}\right]} \quad = \quad 0.127$$

Natural frequency of 1^{st} mode

$$\Omega = 2\pi/T = 2\pi/0.802 \quad = \quad 7.831 \text{ rad/sec}$$

Optimum value of Stiffness of TMD

$$k_{opt} = f_{opt}^2 \Omega^2 m$$

$$k_{opt} = 0.947^2 \times 7.831^2 \times 363.45 \quad = \quad 19992.28 \text{ kN/m}$$

Optimum value of Damping of TMD

$$C_{opt} = 2mf_{opt}\Omega\xi_{opt}$$

$$C_{opt} = 2 \times 0.127 \times 0.947 \times 7.83 \times 363.4 \quad = \quad 685.32 \text{ kN.sec/m}$$

Appendix B

List of Papers Presented/Communicated

List of Paper Presented

1. K.N. Karavadia and G.N. Patel, "Effectiveness of Tuned Mass Damper Under Seismic Excitations", *Dr. S.N. Patel Seminar*, Birla Vishvakarma Mahavidyalaya, Vallabh Vidyanagar, January 2010.

List of Paper Communicated

1. K.N. Karavadia and G.N. Patel,"Application of Tuned Mass Damper in 3D Structure using Nonlinear Analysis", *ICIWSE-2010*, International conference on Innovative World of Structural Engineering, September 17-19, 2010, Aurangabad, India. (Abstract Selected)

Appendix C

List of Useful Websites

1. www.csiberkly.com
2. www.ctbuh.org
3. www.fema.gov/library
4. www.iitk.ac.in/library
5. http://mceer.buffalo.edu
6. www.nicee.ac.org
7. http://nisee.berkeley.edu/elibrary
8. www.nist.gov
9. www.palldynaimcs.com
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