APPLICATION OF TUNED MASS DAMPER FOR VIBRATION CONTROL OF FRAME STRUCTURES UNDER SEISMIC EXCITATIONS

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR

THE DEGREE OF

MASTER OF TECHNOLOGY

IN

STRUCTURAL ENGINEERING

BY

RASHMI MISHRA

209CE2044

NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA

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UNDER THE GUIDANCE OF

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DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA

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ROURKELA

Certificate

*This is to certify that the thesis entitled, "**APPLICATION OF TUNED MASS DAMPER FOR VIBRATION CONTROL OF FRAME STRUCTURES UNDER SEISMIC EXCITATIONS**" submitted by Rashmi Mishra in partial fulfillment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in "**Structural Engineering**" at National Institute of Technology, Rourkela is an authentic work carried out by her under my supervision and guidance. To the best of my knowledge, the matter embodied in this Project review report has not been submitted to any other university/ institute for award of any Degree or Diploma.*

Date: ***(Prof.K.C.Biswal)***

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ABSTRACT:

Current trends in construction industry demands taller and lighter structures, which are also more flexible and having quite low damping value. This increases failure possibilities and also problems from serviceability point of view. Now-a-days several techniques are available to minimize the vibration of the structure, out of the several techniques available for vibration control ,concept of using TMD is a newer one. This study was made to study the effectiveness of using TMD for controlling vibration of structure. At first a numerical algorithm was developed to investigate the response of a shear building fitted with a TMD. Then another numerical algorithm was developed to investigate the response of a 2D frame model fitted with a TMD. A total of three loading conditions were applied at the base of the structure. First one was a sinusoidal loading, the second one was corresponding to compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil with (PGA = 1g) and the third one was 1940 El Centro Earthquake record with (PGA = 0.313g).

From the study it was found that, TMD can be effectively used for vibration control of structures. TMD was more effective when damping ratio of the structure is less. Gradually increasing the mass ratio of the TMD results in gradual decrement in the displacement response of the structure.

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CHAPTER-1

INTRODUCTION

1.1 Introduction

Vibration control is having its roots primarily in aerospace related problems such as tracking and pointing, and in flexible space structures, the technology quickly moved into civil engineering and infrastructure-related issues, such as the protection of buildings and bridges from extreme loads of earthquakes and winds.

The number of tall buildings being built is increasing day by day. Today we cannot have a count of number of low-rise or medium rise and high rise buildings existing in the world. Mostly these structures are having low natural damping. So increasing damping capacity of a structural system, or considering the need for other mechanical means to increase the damping capacity of a building, has become increasingly common in the new generation of tall and super tall buildings. But, it should be made a routine design practice to design the damping capacity into a structural system while designing the structural system.

The control of structural vibrations produced by earthquake or wind can be done by various means such as modifying rigidities, masses, damping, or shape, and by providing passive or active counter forces. To date, some methods of structural control have been used successfully and newly proposed methods offer the possibility of extending applications and improving efficiency.

The selection of a particular type of vibration control device is governed by a number of factors which include efficiency, compactness and weight, capital cost, operating cost, maintenance requirements and safety.

Tuned mass dampers (TMD) have been widely used for vibration control in mechanical engineering systems. In recent years, TMD theory has been adopted to reduce vibrations of tall buildings and other civil engineering structures. Dynamic absorbers and tuned mass dampers are the realizations of tuned absorbers and tuned dampers for structural vibration control applications. The inertial, resilient, and dissipative elements in such devices are: mass, spring and dashpot (or material damping) for linear applications and their rotary counterparts in rotational applications. Depending on the application, these devices are sized from a few ounces (grams) to many tons. Other configurations such as pendulum absorbers/dampers, and sloshing liquid absorbers/dampers have also been realized for vibration mitigation applications.

TMD is attached to a structure in order to reduce the dynamic response of the structure. The frequency of the damper is tuned to a particular structural frequency so that when that frequency is excited, the damper will resonate out of phase with the structural motion. The mass is usually attached to the building via a spring-dashpot system and energy is dissipated by the dashpot as relative motion develops between the mass and the structure.

1.2 Passive energy dissipation:

All vibrating structures dissipate energy due to internal stressing, rubbing, cracking, plastic deformations, and so on; the larger the energy dissipation capacity the smaller the amplitudes of vibration. Some structures have very low damping of the order of 1% of critical damping and consequently experience large amplitudes of vibration even for moderately strong earthquakes. Methods of increasing the energy dissipation capacity are very effective in reducing the amplitudes of vibration. Many different methods of increasing damping have been utilized and many others have been proposed.

Passive energy dissipation systems utilises a number of materials and devices for enhancing damping, stiffness and strength, and can be used both for natural hazard mitigation and for rehabilitation of aging or damaged structures. In recent years, efforts have been undertaken to develop the concept of energy dissipation or supplemental damping into a workable technology and a number of these devices have been installed in structures throughout the world (Soong and Constantinou 1994; Soong and Dargush 1997). In general, they are characterized by the capability to enhance energy dissipation in the structural systems in which they are installed. This may be achieved either by conversion of kinetic energy to heat, or by transferring of energy among vibrating modes. The first method includes devices that operate on principles such as frictional sliding, yielding of metals, phase transformation in metals, deformation of viscoelastic solids or fluids, and fluid orificing. The later method includes supplemental oscillators, which act as dynamic vibration absorbers.

1.3 Types of passive control devices

1.3.1) Metallic yield dampers

One of the effective mechanisms available for the dissipation of energy, input to a structure from an earthquake is through inelastic deformation of metals. The idea of using metallic energy dissipators within a structure to absorb a large portion of the seismic energy began with the conceptual and experimental work of Kelly et al. (1972) and Skinner et al. (1975). Several of the devices considered include torsional beams, flexural beams, and V-strip energy dissipators. Many of these devices use mild steel plates with triangular or hourglass shapes so that yielding is spread almost uniformly throughout the material. A typical X-shaped plate damper or added damping and stiffness (ADAS) device is shown in Fig.

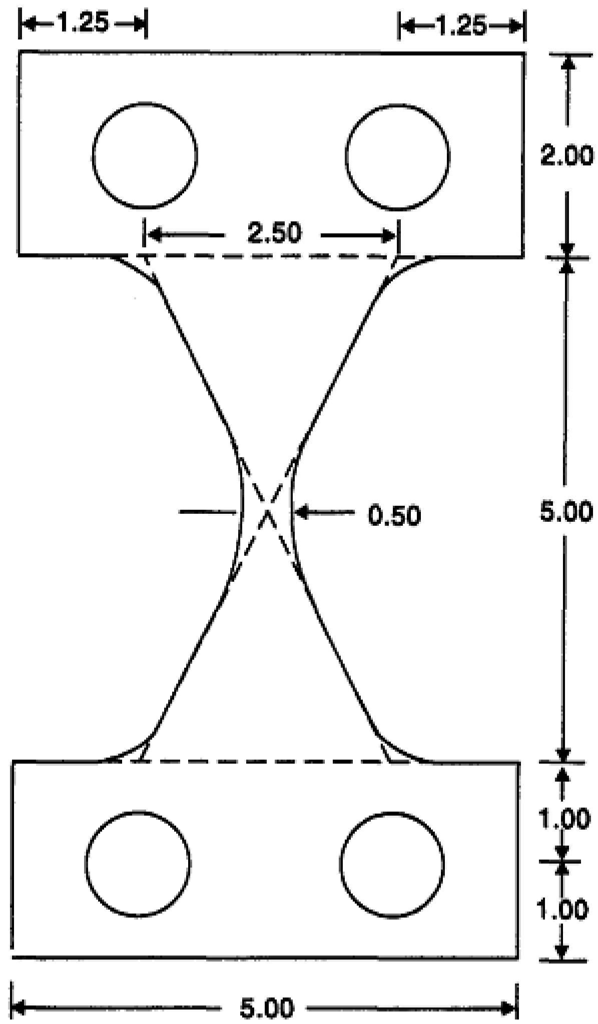


Fig 1.1 X-shaped ADAS device

1.3.2) Friction dampers

Friction provides another excellent mechanism for energy dissipation, and has been used for many years in automotive brakes to dissipate kinetic energy of motion. In the development of friction dampers, it is important to minimize stick-slip phenomena to avoid introducing high frequency excitation. Furthermore, compatible materials must be employed to maintain a consistent coefficient of friction over the intended life of the device. The Pall device is one of the damper elements utilizing the friction principle, which can be installed in a structure in an X-braced frame as illustrated in the figure (Palland Marsh 1982).

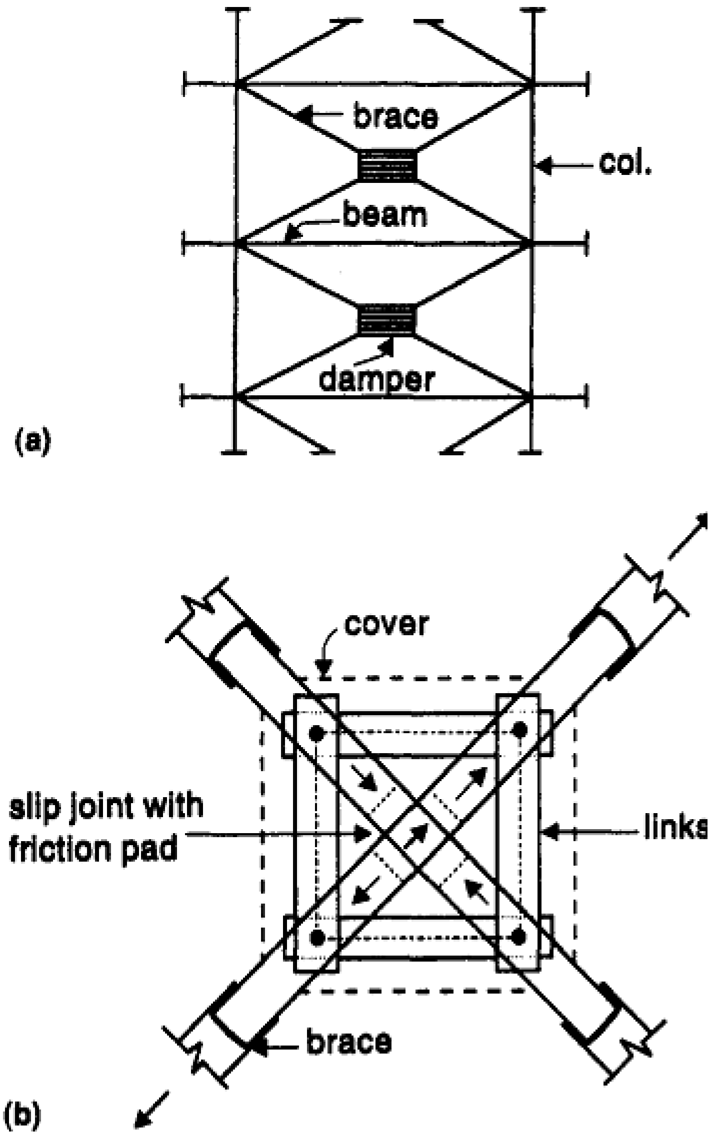


Fig 1.2 Pall Friction Damper

1.3.3) Viscoelastic dampers

The metallic and frictional devices described are primarily intended for seismic application. But, viscoelastic dampers find application in both wind and seismic application. Their application in civil engineering structures began in 1969 when approximately 10,000 visco-elastic dampers were installed in each of the twin towers of the World Trade Center in New York to reduce wind-induced vibrations. Further studies on the dynamic response of viscoelastic dampers have been carried out, and the results show that they can also be effectively used in reducing structural response due to large range of intensity levels of earthquake. Viscoelastic materials used in civil engineering structure are typical copolymers or glassy substances. A typical viscoelastic damper, developed by the 3M Company Inc., is shown in Fig. It consists of viscoelastic layers bonded with steel plates.

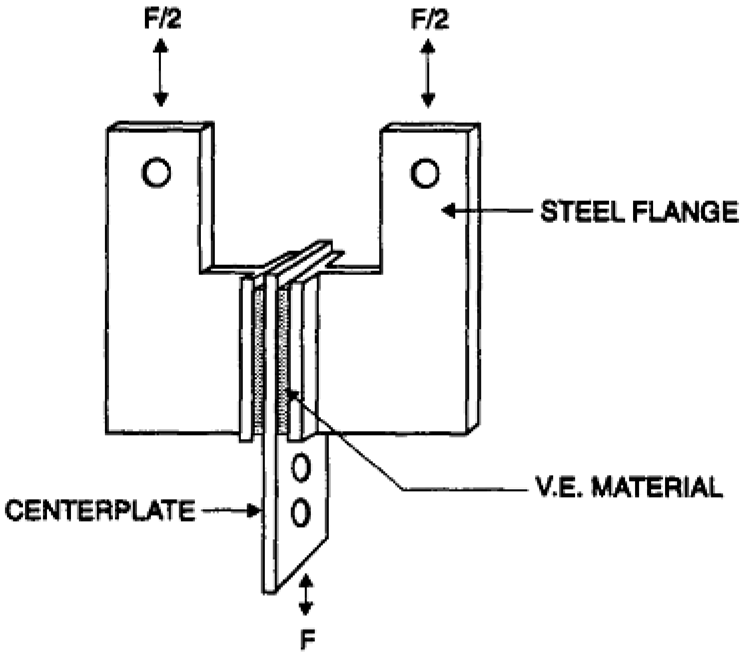


Fig. 1.3 Viscoelastic damper

1.3.4) Viscous fluid dampers

Fluids can also be used to dissipate energy and numerous device configurations and materials have been proposed. Viscous fluid dampers, are widely used in aerospace and military applications, and have recently been adapted for structural applications (Constantinou et al. 1993). Characteristics of these devices which are of primary interest in structural applications, are the linear viscous response achieved over a broad frequency range, insensitivity to temperature, and compactness in comparison to stroke and output force. The viscous nature of the device is obtained through the use of specially configured orifices, and is responsible for generating damper forces that are out of phase with displacement. A viscous fluid damper generally consists of a piston in the damper housing filled with a compound of silicone or oil (Makris and Constantinou 1990; Constantinou and Symans1992). A typical damper of this type is shown in Fig.

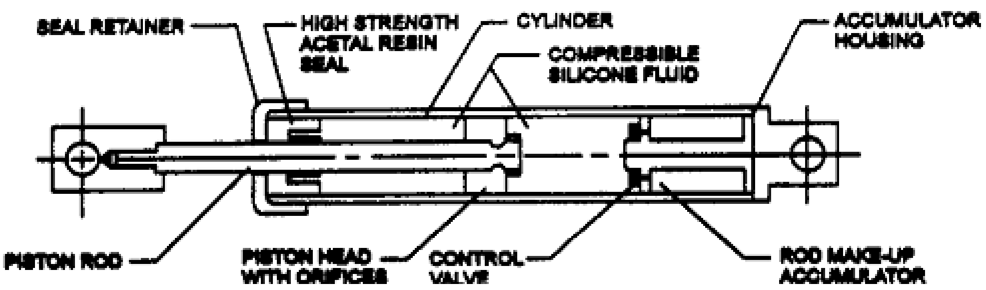


Fig 1.4 Taylor device fluid damper

1.3.5) Tuned liquid damper

A properly designed partially filled water tank can be utilized as a vibration absorber to reduce the dynamic motion of a structure and is referred to as a tuned liquid damper (TLD). Tuned liquid damper (TLD) and tuned liquid column damper (TLCD) impart indirect damping to the system and thus improve structural performance (Kareem 1994). A TLD absorbs structural energy by means of viscous actions of the fluid and wave breaking.

Tuned liquid column dampers (TLCDs) are a special type of tuned liquid damper (TLD) that rely on the motion of the liquid column in a U-shaped tube to counter act the action of external forces acting on the structure. The inherent damping is introduced in the oscillating liquid column through an orifice.

The performance of a single-degree-of-freedom structure with a TLD subjected to sinusoidal excitations was investigated by Sun(1991), along with its application to the suppression of wind induced vibration by Wakahara et al. (1989). Welt and Modi (1989) were one of the first to suggest the usage of a TLD in buildings to reduce overall response during strong wind or earthquakes.

1.3.6) Tuned mass dampers

The concept of the tuned mass damper (TMD) dates back to the 1940s (Den Hartog 1947). It consists of a secondary mass with properly tuned spring and damping elements, providing a frequency-dependent hysteresis that increases damping in the primary structure. The success of such a system in reducing wind-excited structural vibrations is now well established. Recently, numerical and experimental studies have been carried out on the effectiveness of TMDs in reducing seismic response of structures (for instance, Villaverde(1994))

1.4) Classification of Control Methods

1.4.1) Active Control

An active control system is one in which an external power source the control actuators are used that apply forces to the structure in a prescribed manner. These forces can be used to both add or dissipate energy from the structure. In an active feedback control system, the signals sent to the control actuators are a function of the response of the system measured with physical sensors (optical, mechanical, electrical, chemical, and so on).

1.4.2) Passive Control

A passive control system does not require an external power source. Passive control devices impart forces that are developed in response to the motion of the structure. Total energy (structure plus passive device) cannot increase, hence inherently stable.

1.4.3) Hybrid Control

The term "hybrid control" implies the combined use of active and passive control systems. For example, a structure equipped with distributed viscoelastic damping supplemented with an active mass damper near the top of the structure, or a base isolated structure with actuators actively controlled to enhance performance.

1.4.4) Semi-active Control

Semi-active control systems are a class of active control systems for which the external energy requirements are less than typical active control systems. Typically, semi-active control devices do not add mechanical energy to the structural system (including the structure and the control actuators), therefore bounded-input bounded-output stability is guaranteed. Semi-active control devices are often viewed as controllable passive devices.

1.5) Practical Implementations:

Till date TMD have been installed in large number of structures all around the globe. The first structure in which TMD was installed is the Centrepoint Tower in Sydney, Australia.

There are two buildings in the United States equipped with TMDs; one is the Citicorp Centre in New York City and the other is the John Hancock Tower in Boston. The Citicorp Centre building is 279 m high and has a fundamental period of around 6.5 s with an inherent damping ratio of 1% along each axis. The Citicorp TMD, located on the sixty-third floor in the crown of the structure, has a mass of 366 Mg, about 2% of the effective modal mass of the first mode, and was 250 times larger than any existing tuned mass damper at the time of installation. Designed to be bi-axially resonant on the building structure with a variable operating period of , adjustable linear damping from 8 to 14%, and a peak relative displacement of , the damper is expected to reduce the building sway amplitude by about 50%.

Two dampers were added to the 60-storey John Hancock Tower in Boston to reduce the response to wind loading. The dampers are placed at opposite ends of the fifty-eighth story, 67 m apart, and move to counteract sway as well as twisting due to the shape of the building. Each damper weighs 2700 kN and consists of a lead-filled steel box about 5.2 m square and 1 m deep that rides on a 9-m-long steel plate. The lead-filled weight, laterally restrained by stiff springs anchored to the interior columns of the building and controlled by servo-hydraulic cylinders, slides back and forth on a hydrostatic bearing consisting of a thin layer of oil forced through holes in the steel plate.

Chiba Port Tower (completed in 1986) was the first tower in Japan to be equipped with a TMD. Chiba Port Tower is a steel structure 125 m high weighing 1950 metric tons and having a rhombus-shaped plan with a side length of 15 m. The first and second mode periods are 2.25 s and 0.51 s, respectively for the x direction and 2.7 sand 0.57 s for the y direction. Damping for the fundamental mode is estimated at 0.5%. Damping ratios proportional to frequencies were assumed for the higher modes in the analysis. The purpose of the TMD is to increase damping of the first mode for both the x and y directions. the damper has mass ratios with respect to the modal mass of the first mode of about 1/120 in the x direction and 1/80 in the y direction; periods in the x and y directions of 2.24 s and 2.72 s, respectively; and a damper damping ratio of 15%. The maximum relative displacement of the damper with respect to the tower is about in each direction. Reductions of around 30 to 40% in the displacement of the top floor and 30% in the peak bending moments are expected.

In Japan, counter measures against traffic-induced vibration were carried out for two two-story steel buildings under an urban expressway viaduct by means of TMDs (Inoue et al.1994). Results show that peak values of the acceleration response of the two buildings were reduced by about 71% and 64%, respectively, by using the TMDs with the mass ratio about 1%.

CHAPTER-2

LITERATURE REVIEW

2.1) Review of Literature

. The TMD concept was first applied by Frahm in **1909** (Frahm, 1909) to reduce the rolling motion of ships as well as ship hull vibrations. A theory for the TMD was presented later in the paper by Ormondroyd and Den Hartog**(1928),**followed by a detailed discussion of optimal tuning and damping parameters in Den Hartog's book on mechanical vibrations **(1940).**Hartog's book on mechanical vibrations **(1940).** The initial theory was applicable foran undamped SDOF system subjected to a sinusoidal force excitation. Extension of the theory to damped SDOF systems has been investigated by numerous researchers.

Active control devices operate by using an external power supply. Therefore ,they are more efficient than passive control devices. However the problems such as insufficient control-force capacity and excessive power demands encountered by current technology in the context of structural control against earthquakes are unavoidable and need to be overcome. Recently a new control approach-semi-active control devices, which combine the best features of both passive and active control devices, is very attractive due to their low power demand and inherent stability. The earlier papers involving SATMDs may traced to 1983. **Hrovat** *et al.*(1983) presented SATMD, a TMD with time varying controllable damping. Under identical conditions, the behaviour of a structure equipped with SATMD instead of TMD is significantly improved. The control design of SATMD is less dependent on related parameters (e.g, mass ratios, frequency ratios and so on), so that there greater choices in selecting them.

The first mode response of a structure with TMD tuned to the fundamental frequency of the structure can be substantially reduced but, in general, the higher modal responses may only be marginally suppressed or even amplified. To overcome the frequency-related limitations of TMDs, more than one TMD in a given structure, each tuned to a different dominant frequency, can be used. The concept of multiple tuned mass dampers (MTMDs) together with an optimization procedure was proposed by Clark **(1988).** Since, then, a number of studies have been conducted on the behaviour of MTMDs a doubly tuned mass damper (DTMD), consisting of two masses connected in series to the structure was proposed (Setareh 1994). In this case, two different loading conditions were considered: harmonic excitation and zero-mean white-noise random excitation, and the efficiency of DTMDs on response reduction was evaluated. Analytical results show that DTMDs are more efficient than the conventional single mass TMDs over the whole range of total mass ratios, but are only slightly more efficient than TMDs over the practical range of mass ratios (0.01-0.05).

Recently, numerical and experimental studies have been carried out on the effectiveness of TMDs in reducing seismic response of structures [for instance, Villaverde(1994)]. In Villaverde**(1994),** three different structures were studied, in which the first one is a 2D two story shear building the second is a three-dimensional (3D) one-story frame building, and the third is a 3D cable-stayed bridge, using nine different kinds of earthquake records. Numerical and experimental results show that the effectiveness of TMDs on reducing the response of the same structure during different earthquakes, or of different structures during the same earthquake is significantly different; some cases give good performance and some have little or even no effect. This implies that there is a dependency of the attained reduction in response on the characteristics of the ground motion that excites the structure. This response reduction is large for resonant ground motions and diminishes as the dominant frequency of the ground motion gets further away from the structure's natural frequency to which the TMD is tuned. Also, TMDs are of limited effectiveness under pulse-like seismic loading.

Multiple passive TMDs for reducing earthquake induced building motion. Allen J. Clark **(1988).** In this paper a methodology for designing multiple tuned mass damper for reducing building response motion has been discussed. The technique is based on extending Den Hartog work from a single degree of freedom to multiple degrees of freedom. Simplified linear mathematical models were excited by 1940 El Centro earthquake and significant motion reduction was achieved using the design technique.

Performance of tuned mass dampers under wind loads K. C. S**.** Kwok *et al***(1995).**The performance of both passive and active tuned mass damper (TMD) systems can be readily assessed by parametric studies which have been the subject of numerous research.. Few experimental verifications of TMD theory have been carried out, particularly those involving active control, but the results of those experiments generally compared well with those obtained by parametric studies. Despite some serious design constraints, a number of passive and active tuned mass damper systems have been successfully installed in tall buildings and other structures to reduce the dynamic response due to wind and earthquakes.

Mitigation of response of high-rise structural systems by means of optimal tuned mass damper. A.N Blekherman**(1996)**. In this paper a passive vibration absorber has been proposed to protect high-rise structural systems from earthquake damages. A structure is modelled by one-mass and n-mass systems(a cantilever scheme). Damping of the structure and absorber installed on top of it is represented by frequency independent one on the base of equivalent visco-elastic model that allows the structure with absorber to be described as a system with non-proportional internal friction. A ground movement is modelled by an actuator that produces vibration with changeable amplitude and frequency. To determine the optimum absorber parameters, an optimization problem, that is a minmax one , was solved by using nonlinear programming technique(the Hooke-Jeves method).

Survey of actual effectiveness of mass damper systems installed in buildings. T.Shimazu and H.Araki**(1996).** In this paper the real state of the implementation of mass damper systems, the effects of these systems were clarified based on various recorded values in actual buildings against both wind and earthquake. The effects are discussed in relation with the natural period of buildings equipped with mass damper systems, the mass weight ratios to building weight, wind force levels and earthquake ground motion levels.

A method of estimating the parameters of tuned mass dampers for seismic applications. Fahim Sadek *et al***(1997)**. In this paper the optimum parameters of TMD that result in considerable reduction in the response of structures to seismic loading has been presented. The criterion that has been used to obtain the optimum parameters is to select for a given mass ratio, the frequency and damping ratios that would result in equal and large modal damping in the first two modes of vibration. The parameters are used to compute the response of several single and multi-degree of freedom structures with TMDs to different earthquake excitations. The results show that the use of the proposed parameters reduces the displacement and acceleration responses significantly. The method can also be used for vibration control of tall buildings using the so-called 'mega-substructure configuration', where substructures serve as vibration absorbers for the main structure.

Structural control: past, present, and future G. W. Housner *et al***(1996).**This paper basically provides a concise point of departure for those researchers and practitioners who wishing to assess the current state of the art in the control and monitoring of civil engineering structures; and provides a link between structural control and other fields of control theory, pointing out both differences and similarities, and points out where future research and application efforts are likely to prove fruitful.

Structural vibration of tuned mass installed three span steel box bridge. Byung-Wan Jo *et al* **(2001).**To reduce the structural vibration of a three span steel box bridge a three axis two degree of freedom system is adopted to model the mass effect of the vehicle; and the kinetic equation considering the surface roughness of the bridge is derived based on Bernoulli-Euler beam ignoring the torsional DOF. The effects of TMD on steel box bridge shows that it is not effective in reducing the maximum deflection ,but it efficiently reduces the free vibration of the bridge. It proves that the TMD is effective in controlling the dynamic amplitude rather than the maximum static deflection.

Optimal placement of multiple tuned mass dampers for seismic structures. Genda Chen *et al***(2001).** In this paper effects of a tuned mass damper on the modal responses of a six-story building structure are studied. Multistage and multimode tuned mass dampers are then introduced. Several optimal location indices are defined based on intuitive reasoning, and a sequential procedure is proposed for practical design and placement of the dampers in seismically excited building structures. The proposed procedure is applied to place the dampers on the floors of the six-story building for maximum reduction of the accelerations under a stochastic seismic load and 13 earthquake records. Numerical results show that the multiple dampers can effectively reduce the acceleration of the uncontrolled structure by 10–25% more than a single damper. Time-history analyses indicate that the multiple dampers weighing 3% of total structural weight can reduce the floor acceleration up to 40%.

Seismic effectiveness of tuned mass dampers for damage reduction of structures. T. Pinkaew *et al***(2002).**The effectiveness of TMD using displacement reduction of the structure is found to be insufficient after yielding of the structure, damage reduction of the structure is proposed instead. Numerical simulations of a 20-storey reinforced concrete building modelled as an equivalent inelastic single-degree-of-freedom (SDOF) system subjected to both harmonic and the 1985 Mexico City (SCT) ground motions are considered. It is demonstrated that although TMD cannot reduce the peak displacement of the controlled structure after yielding, it can significantly reduce damage to the structure. In addition, certain degrees of damage protection and collapse prevention can also be gained from the application of TMD.

Tuned Mass Damper Design for Optimally Minimizing Fatigue Damage. Hua-Jun Li *et al***[47](2002).** This paper considers the environmental loading to be a long-term non-stationary stochastic process characterized by a probabilistic power spectral density function. One engineering technique to design a TMD under a long-term random loading condition is for prolonging the fatigue life of the primary structure.

Seismic structural control using semi-active tuned mass dampers. Yang Runlin *et al***(2002).**This paper focuses on how to determine the instantaneous damping of the semi-active tuned mass damper with continuously variable damping. An off-and- towards-equilibrium(OTE) algorithm is employed to examine the control performance of the structure/SATMD system by considering damping as an assumptive control action. Two numerical simulations of a five-storey and a ten-storey shear structures with a SATMD on the roof are conducted. The effectiveness on vibration reduction of MDOF systems subjected to seismic excitations is discussed

Structural vibration suppression via active/passive techniques. Devendra P**.** Garg *et al***[(2003).** The advances made in the area of vibration suppression via recently developed innovative techniques (for example, constrained layer damping (CLD) treatments) applied to civilian and military structures are investigated. Developing theoretical equations that govern the vibration of smart structural systems treated with piezo-magnetic constrained layer damping

(PMCLD) treatments; and developing innovative surface damping treatments using micro-cellular foams and active standoff constrained layer (ASCL) treatments. The results obtained from the above and several other vibration suppression oriented research projects being carried out under the ARO sponsorship are also included in this study.

Performance of a five-storey benchmark model using an active tuned mass damper and a fuzzy controller. Bijan Samali, Mohammed Al-Dawod**(2003)**. This paper describes the performance of a five-storey benchmark model using an active tuned mass damper (ATMD), where the control action is achieved by a Fuzzy logic controller (FLC) under earthquake excitations. The advantage of the Fuzzy controller is its inherent robustness and ability to handle any non-linear behaviour of the structure. The simulation analysis of the five-storey benchmark building for the uncontrolled building, the building with tuned mass damper (TMD), and the building with ATMD with Fuzzy and linear quadratic regulator (LQR) controllers has been reported, and comparison between Fuzzy and LQR controllers is made. In addition, the simulation analysis of the benchmark building with different values of frequency ratio, using a Fuzzy controller is conducted and the effect of mass ratio, on the five-storey benchmark model using the Fuzzy controller has been studied.

Behaviour of soil-structure system with tuned mass dampers during near-source earthquakes. Nawawi Chouw**(2004)**. In this paper the influence of a tuned mass damper on the behaviour of a frame structure during near-source ground excitations has been presented. In the investigation the effect of soil-structure interaction is considered, and the natural frequency of the tuned mass damper is varied. The ground excitations used are the ground motion at the station SCG and NRG of the 1994 Northridge earthquake. The investigation shows that the soil-structure interaction and the characteristic of the ground motions may have a strong influence on the effectiveness of the tuned mass damper. But in order to obtain a general conclusion further investigations are necessary.

Wind Response Control of Building with Variable Stiffness Tuned Mass Damper Using Empirical Mode Decomposition Hilbert Transform Nadathur Varadarajan *et al*(**2004).**The effectiveness of a novel semi-active variable stiffness-tuned mass damper ~SAIVS-TMD! for the response control of a wind-excited tall benchmark building is investigated in this study. The benchmark building considered is a proposed 76-story concrete office tower in Melbourne, Australia. Across wind load data from wind tunnel tests are used in the present study. The objective of this study is to evaluate the new SAIVS-TMD system, that has the distinct advantage of continuously retuning its frequency due to real time control and is robust to changes in building stiffness and damping. The frequency tuning of the SAIVS-TMD is achieved based on empirical mode decomposition and Hilbert transform instantaneous frequency algorithm developed by the writers. It is shown that the SAIVS-TMD can reduce the structural response substantially, when compared to the uncontrolled case, and it can reduce the response further when compared to the case with TMD. Additionally, it is shown the SAIVS-TMD reduces response even when the building stiffness changes by $\pm$15%.

Effect of soil interaction on the performance of tuned mass dampers for seismic applications. A. Ghosha, B. Basu**(2004).**The properties of the structure used in the design of the TMD are those evaluated considering the structure to be of a fixed-base type. These properties of the structure may be significantly altered when the structure has a flexible base, i.e. when the foundation of the structure is supported on compliant soil and undergoes motion relative to the surrounding soil. In such cases, it is necessary to study the effects of soil-structure interaction (SSI) while designing the TMD for the desired vibration control of the structure. In this paper, the behaviour of flexible-base structures with attached TMD, subjected to earthquake excitations has been investigated. Modified structural properties due to SSI has been covered in this paper.

Optimal design theories and applications of tuned mass dampers. Chien-Liang Lee *et al***(2006)**.An optimal design theory for structures implemented with tuned mass dampers (TMDs) is proposed in this paper. Full states of the dynamic system of multiple-degree-of-freedom (MDOF) structures, multiple TMDs (MTMDs) installed at different stories of the building, and the power spectral density (PSD) function of environmental disturbances are taken into account. The optimal design parameters of TMDs in terms of the damping coefficients and spring constants corresponding to each TMD are determined through minimizing a performance index of structural responses defined in the frequency domain. Moreover, a numerical method is also proposed for searching for the optimal design parameters of MTMDs in a systematic fashion such that the numerical solutions converge monotonically and effectively toward the exact solutions as the number of iterations increases. The feasibility of the proposed optimal design theory is verified by using a SDOF structure with a single TMD (STMD), a five-DOF structure with two TMDs, and a ten-DOF structure with a STMD.

Optimum design for passive tuned mass dampers using viscoelastic materials. I Saidi, A D Mohammed *et al***(2007)**. This paper forms part of a research project which aims to develop an innovative cost effective Tune Mass Damper (TMD) using viscoelastic materials. Generally, a TMD consists of a mass, spring, and dashpot which is attached to a floor to form a two-degree of freedom system. TMDs are typically effective over a narrow frequency band and must be tuned to a particular natural frequency. The paper provides a detailed methodology for estimating the required parameters for an optimum TMD for a given floor system. The paper also describes the process for estimating the equivalent viscous damping of a damper made of viscoelastic material. Finally, a new innovative prototype viscoelastic damper is presented along with associated preliminary results.

Semi-active Tuned Mass Damper for Floor Vibration Control .Mehdi Setareh *et al***(2007)**. A semi-active magneto-rheological device is used in a pendulum tuned mass damper PTMD system to control the excessive vibrations of building floors. This device is called semi-active pendulum tuned mass damper SAPTMD. Analytical and experimental studies are conducted to compare the performance of the SAPTMD with its equivalent passive counterpart. An equivalent single degree of freedom model for the SAPTMD is developed to derive the equations of motion of the coupled SAPTMD-floor system. A numerical integration technique is used to compute the floor dynamic response, and the optimal design parameters of the SAPTMD are found using an optimization algorithm. Effects of off-tuning due to the variations of the floor mass on the performance of the PTMD and SAPTMD are studied both analytically and experimentally. From this study it can be concluded that for the control laws considered here an optimum SAPTMD performs similarly to its equivalent PTMD, however, it is superior to the PTMD when the floor is subjected to off-tuning due to floor mass variations from sources other than human presence.

Seismic Energy Dissipation of Inelastic Structures with Tuned Mass Dampers. K. K. F. Wong**(2008).**The energy transfer process of using a tuned mass damper TMD in improving the ability of inelastic structures to dissipate earthquake input energy is investigated. Inelastic structural behaviour is modelled by using the force analogy method, which is the backbone of analytically characterizing the plastic energy dissipation in the structure. The effectiveness of TMD in reducing energy responses is also studied by using plastic energy spectra for various structural yielding levels. Results show that the use of TMD enhances the ability of the structures to store larger amounts of energy inside the TMD that will be released at a later time in the form of damping energy when the response is not at a critical state, thereby increasing the damping energy dissipation while reducing the plastic energy dissipation. This reduction of plastic energy dissipation relates directly to the reduction of damage in the structure.

Dynamic analysis of space structures with multiple tuned mass dampers. Y.Q. Guo, W.Q.Chen**(2008).** Formulations of the reverberation matrix method (RMM) are presented for the dynamic analysis of space structures with multiple tuned mass dampers (MTMD). The theory of generalized inverse matrices is then employed to obtain the frequency response of structures with and without damping, enabling a uniform treatment at any frequency, including the resonant frequency. For transient responses, the Neumann series expansion technique as suggested in RMM is found to be confined to the prediction of accurate response at an early time. The artificial damping technique is employed here to evaluate the medium and long time response of structures. The free vibration, frequency response, and transient response of structures with MTMD are investigated by the proposed method through several examples. Numerical results indicate that the use of MTMD can effectively alter the distribution of natural frequencies as well as reduce the frequency/transient responses of the structure. The high accuracy, lower computational cost, and uniformity of formulation of RMM are also highlighted in this paper.

Exploring the performance of a nonlinear tuned mass damper. Nicholas A. Alexander, Frank Schilder **(2009).**In this the performance of a nonlinear tuned mass damper (NTMD), which is modelled as a two degree of freedom system with a cubic nonlinearity has been covered. This nonlinearity is physically derived from a geometric configuration of two pairs of springs. The springs in one pair rotate as they extend, which results in a hardening spring stiffness. The other pair provides a linear stiffness term. In this paper an extensive numerical study of periodic responses of the NTMD using the numerical continuation software AUTO has been done. Two techniques have been employed for searching the optimal design parameters;

optimization of periodic solutions and parameter sweeps. In this paper the writers have discovered a family of resonance curves for vanishing linear spring stiffness

Application of semi-active control strategies for seismic protection of buildings with MR dampers. Maryam Bitaraf *et al***(2010).**Magneto-rheological (MR) dampers are semi-active devices that can be used to control the response of civil structures during seismic loads. They are capable of offering the adaptability of active devices and stability and reliability of passive devices. One of the challenges in the application of the MR dampers is to develop an effective control strategy that can fully exploit the capabilities of the MR dampers. This study proposes two semi-active control methods for seismic protection of structures using MR dampers. The first method is the Simple Adaptive Control method which is classified as a direct adaptive control method. The controller developed using this method can deal with the changes that occur in the characteristics of the structure because it can modify its parameters during the control procedure. The second controller is developed using a genetic-based fuzzy control method. In particular, a fuzzy logic controller whose rule base determined by a multi-objective genetic algorithm is designed to determine the command voltage of MR dampers.

Vibration control of seismic structures using semi-active friction multiple tuned mass dampers. Chi-Chang Lin *et al***.(2010)** There is no difference between a friction-type tuned mass damper and a dead mass added to the primary structure if static friction force inactivates the mass damper. To overcome this disadvantage, this paper proposes a novel semi-active friction-type multiple tuned mass damper (SAF-MTMD) for vibration control of seismic structures. Using variable friction mechanisms, the proposed SAF-MTMD system is able to keep all of its mass units activated in an earthquake with arbitrary intensity. A comparison with a system using passive friction-type multiple tuned mass dampers (PF-MTMDs) demonstrates that the SAF-MTMD effectively suppresses the seismic motion of a structural system, while substantially reducing the strokes of each mass unit, especially for a larger intensity earthquake.

2.2) Aim and Scope of this work

The aim of the present work is to study the effect of TMD on the dynamic response of multi-storey frame structures under earthquake excitations. The scope of the work includes the modelling the multi-storey building as 1D and 2D models. The finite elements have been used to discretize the building frame structures and TMD. The Newmark Beta method is used to solve the dynamic equations for the structure-TMD system.

CHAPTER-3

MATHEMATICAL FORMULATIONS

3.1) Concept of tuned mass damper using two mass system

The equation of motion for primary mass as shown in figure 3.1 is:

$$(1+\overline{m})\ddot{u} + 2\varepsilon\omega m\,\dot{u}+\omega^2 u = \frac{p}{m} - \overline{m}\,\ddot{u}_d$$

$\overline{m}$ is defined as the mass ratio, $\overline{m} = m_d/m$

$$\omega^2 = k/m\ ,\ C = 2\varepsilon\omega m,\ \ C_d = 2\varepsilon w_d m_d$$

where , $\dot{u}$ is the velocity, $\ddot{u}$ is the acceleration, ε is the damping factor of the primary mass

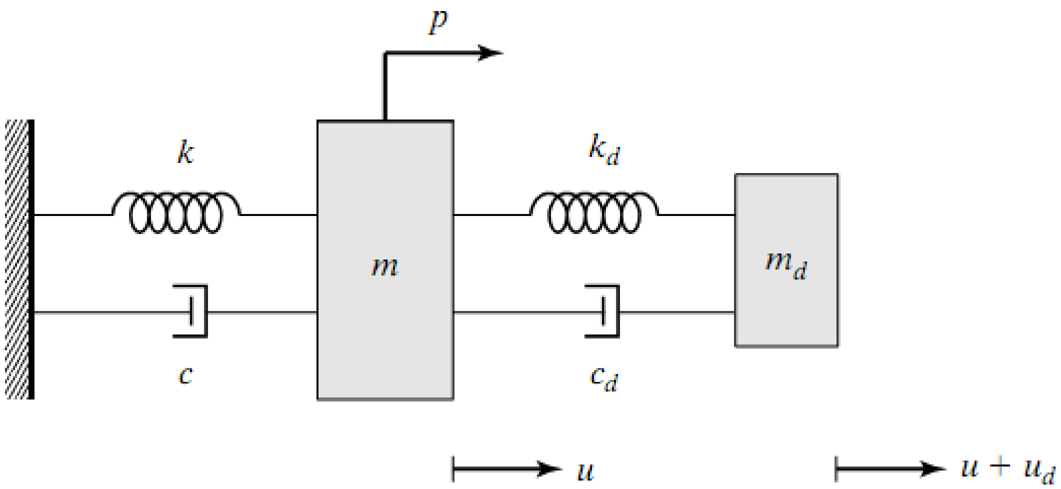


Fig 3.1 SDOF-TMD SYSTEM

The equation of motion for tuned mass is given by:

$$\ddot{u}_d + 2\varepsilon_d\omega_d\dot{u}_d + \omega_d^2 u_d = -\ddot{u}$$

The purpose of adding the mass damper is to control the vibration of the structure when it is subjected to a particular excitation. The mass damper is having the parameters; the mass m_d, stiffness k_d, and damping coefficient c_d. The damper is tuned to the fundamental frequency of the structure such that

$$\omega_d = \omega$$

$$k_d = \overline{m}\, k$$

The primary mass is subjected to the following periodic sinusoidal excitation

$$p = \hat{p}\sin\Omega t$$

then the response is given by

$$u = \hat{u}\sin(\Omega t + \delta_1)$$

$$u_d = \hat{u}_d\sin(\Omega t + \delta_1 + \delta_2)$$

where $\hat{u}$ and δ denote the displacement amplitude and phase shift, respectively.

The critical loading scenario is the resonant condition, The solution for this

case has the following form:

$$\hat{u} = \frac{\hat{p}}{mk}\sqrt{1/(1 + (2\varepsilon/\overline{m} + 1/2\varepsilon d)2)} \quad \text{.......................................(1)}$$

$$\hat{u}_d = (1/2\varepsilon_d)\hat{u}$$

$$\tan\delta_1 = -(2\varepsilon/\overline{m} + 1/2\varepsilon_d)$$

$$\tan\delta_2 = -\Pi/2$$

The above expression shows that the response of the tuned mass is 90º out of phase with the response of the primary mass. This difference in phase produces the energy dissipation contributed by the damper inertia

$$\hat{u}=\frac{\hat{p}}{k}\left(\frac{1}{2\varepsilon_{\varsigma}}\right)$$

$$\delta_1=-\frac{\pi}{2}$$

To compare these two cases, we can express Eq (1) in terms of an equivalent damping ratio:

$$\hat{u}=\frac{\hat{p}}{k}(1/2\varepsilon_{\varsigma e})$$

where

$$\varepsilon_{\varsigma e}=\frac{\bar{m}}{2}\sqrt{1/1+(2\varepsilon_{\varsigma}/\bar{m}+1/2\varepsilon_{\varsigma d})^{2}} \text{...(2)}$$

Equation (2) shows the relative contribution of the damper parameters to the total damping.

Increasing the mass ratio magnifies the damping. However, since the added mass also increases, so there is a practical limit on it.

3.2) Tuned Mass Damper theory for SDOF systems

Various cases ranging from fully undamped to fully damped conditions are considered and are presented as follows

3.2.1) Undamped Structure: Undamped TMD

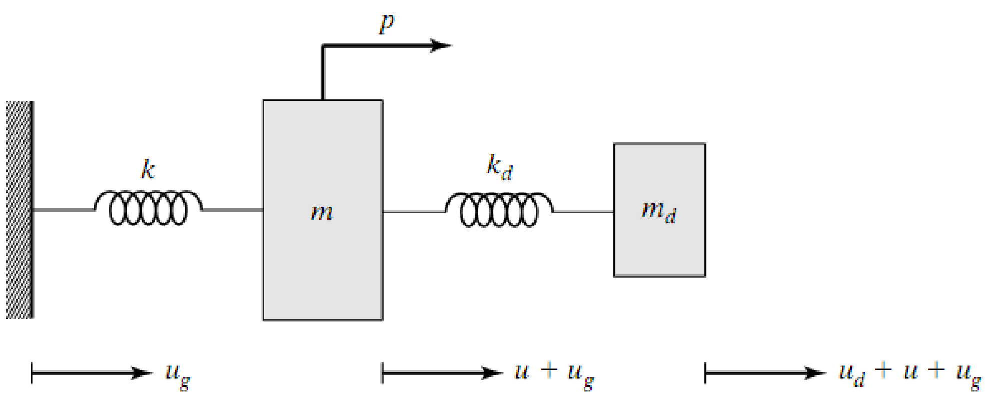


Fig 3.3 Undamped SDOF system coupled with a damped TMD system.

Figure shows a SDOF system having mass m and stiffness k , subjected to both external forcing and ground motion. A tuned mass damper with mass m_d and stiffness k_d is attached to the primary mass. The various displacement measures are u_g the absolute ground motion; ,the relative motion between the primary mass and the ground u; and u_d, the relative displacement between the damper and the primary mass.

The equations for secondary mass and primary mass are as follows:

$$m_d(\ddot{u}_d + \ddot{u}) + k_d u_d = -m_d a_g \text{..................(3)}$$

$$m\ddot{u} + ku - k_d u_d = -m a_g + p \text{....................(4)}$$

where a_g is the absolute ground acceleration and p is the loading applied to the primary mass.

The excitation applied on the primary mass is considered to be periodic of frequency , Ω

$$a_g = \hat{a}_g \sin\Omega t$$

$$p = \hat{p} \sin\Omega t$$

Then the response is given by

$$u = \hat{u} \sin\Omega t$$

$$u_d = \hat{u}_d \sin\Omega t$$

and substituting the values of u and u_d, the equations (3) and (4) can be written as follows:

$$(-m_d\Omega^2 + k_d)\hat{u}_d - m_d\Omega^2\hat{u} = -m_d\hat{a}_g$$

$$k_d\hat{u}_g + (-m\Omega^2 + k)\hat{u} = -m\hat{a}_g + p$$

The solutions for $\hat{u}$ and $\hat{u}_d$ are given by

$$\hat{u} = \hat{p}/k((1 - \rho^2{}_d)/D_1) - m\hat{a}_g/k((1+\overline{m} - \rho^2{}_d)/D_1)$$

$$\hat{u}_d = \hat{p}/k_d\,(\overline{m}\rho^2/D_1) - m\hat{a}_g/k_d(\overline{m}/D_1)$$

Where

$$D_1 = [1-\rho^2][1-\rho^2{}_d] - \overline{m}\rho^2$$

and the ρ term is a dimension less quantity i.e a frequency ratio given by,

$$\rho = \frac{\Omega}{\omega} = \Omega/\sqrt{\frac{k}{m}}$$

$$\rho_d = \Omega/\ \omega_d = \Omega/\sqrt{k_{d/}m_d}$$

Selecting appropriate combination of the mass ratio and damper frequency ratio such that

$$\rho^2{}_d + \overline{m} = 0 \quad \text{...(5)}$$

reduces the solution to

$$\hat{u} = \hat{p}/k$$

$$\hat{u}_d = -(\hat{p}/k_d)\,\rho^2 + (m\hat{a}_g/k_d)$$

.A typical range for $\bar{m}$ is 0.01 to 0.1.Then the optimal damper frequency is very close to the forcing frequency. The exact relationship follows from Eq. (5)

$$\omega_d \Big|_{opt} = \Omega/\sqrt{1+\bar{m}}$$

the corresponding damper stiffness is determined as

$$k_d \Big|_{opt} = [\,\omega_{dopt}]^2 m_d = (\Omega^2 m\bar{m})/(1+\bar{m})$$

3.3.2) Undamped Structure: Damped TMD

The equations of motion for this case are

$$m_d\ddot{u}_d + c_d\dot{u}_d + k_d u_d + m_d\ddot{u} = -m_d a_g \quad \text{......(6)}$$

$$m\ddot{u} + ku - c_d\dot{u}_d - k_d u_d = -ma_g + p \quad \text{......(7)}$$

The inclusion of the damping terms in Eqn (6)and (7) produces a phase shift

between the periodic excitation and the response. So, it is convenient to consider the solution expressed in terms of complex quantities. Then the excitation is expressed as

$$a_g = \hat{a}_g e^{i\Omega t}$$

$$p = \hat{p} e^{i\Omega t}$$

where $\hat{a}_g$ and $\hat{p}$ are real quantities.

Then the response is given by

$$u = \bar{u} e^{i\Omega t} \quad \text{......(8)}$$

$$u_d = \bar{u}_d e^{i\Omega t} \quad \text{......(9)}$$

where the response amplitudes, u and u_d, are considered as complex quantities. Substituting Eqn (8) and (9) in the equations (6) and (7) and cancelling $e^{i\Omega t}$ from both sides results in the following equations

$$[- m_d\Omega^2 + ic_d\Omega + k_d]\bar{u}_d - m_d\Omega^2 \bar{u} = -m_d\hat{a}_g$$

$$-[ic_d\Omega + k_d]\bar{u}_d + [-m\Omega^2 + k]\bar{u} = -m\hat{a}_g + \hat{p}$$

The solution of the governing equations is

$$\bar{u} = \hat{p}/kD_2[f^2 - \rho^2 + i2\varepsilon_d\rho f] - \hat{a}_g m/k\, D_2[(1+\bar{m})f^2 - \rho^2 + i2\, \varepsilon_d\rho f(1+\bar{m})]$$

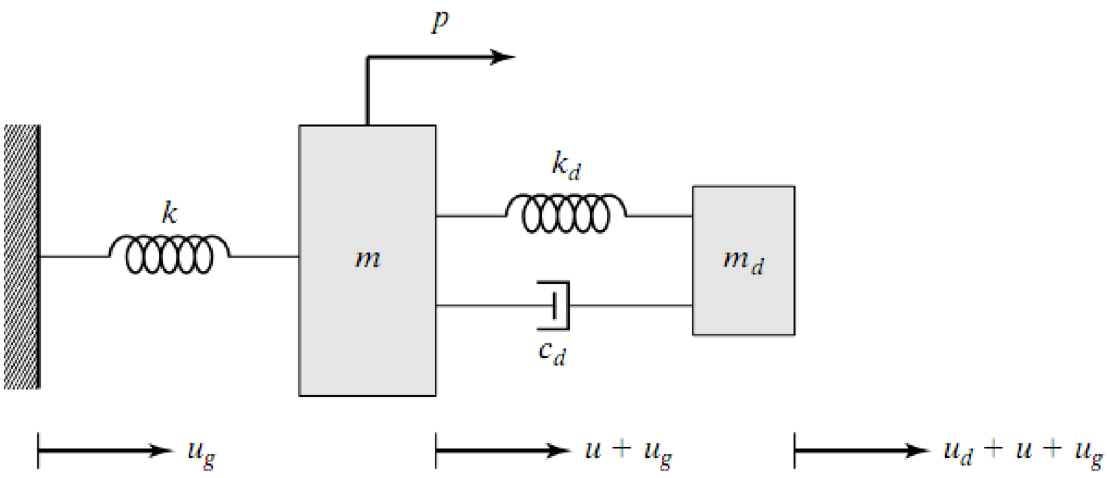


Fig 3.4 Undamped SDOF system coupled with a damped TMD system.

$$\bar{u}_d = (\hat{p}\rho^2/k\, D_2) – (\hat{a}_g m/ k\, D_2)$$

Where

$$D_2 = [1 - \rho^2][f^2 - \rho^2] - \bar{m}\rho^2 f^2 + i2\varepsilon_d\rho f[1 - \rho^2(1+\bar{m})]$$

$$f = \frac{\omega d}{\omega}$$

Converting the complex solutions to polar form leads to the following

expressions:

$$\bar{u} = \frac{\hat{p}}{k} H_1 e^{i\delta 1} - (\hat{a}_g m/k) H_2 e^{i\delta 2}$$

$$\bar{u}_d = \frac{\hat{p}}{k} H_3 e^{-i\delta 3} - (\hat{a}_g m/k) H_4 e^{-i\delta 3}$$

where the H factors define the amplification of the pseudo-static responses, and the δ's are the phase angles between the response and the excitation. The various H terms are as follows

$$H_1 = (\sqrt{[f^2 - \rho^2]^2 + [2\varepsilon_d \rho f]^2}) / |D2|$$

$$H_2 = (\sqrt{[(1+\bar{m})f^2 - \rho^2] + [2\varepsilon_d \rho f (1+\bar{m})]^2}$$

$$H_3 = \rho^2 / |D2|$$

$$H_4 = 1 / |D2|$$

$$|D2| = \sqrt{([1-\rho^2][f^2 - \rho^2] - \bar{m}\rho^2 f^2)^2 + (2\varepsilon_d \rho f [1 - \rho^2 (1+\bar{m})])^2}$$

3.3) Equation for forced vibration analysis of multistorey plane frame

The equation of motion for the frame structure subjected to external dynamic force p(t):

The dynamic response of the structure to this excitation is defined by the displacement x(t), velocity $\dot{x}$(t) , and acceleration $\ddot{x}$(t). The external force may be visualised as distributed among the three components of the structure , first is $f_s(t)$ to the stiffness components, second is $f_D(t)$ to the damping component and the third one is $f_I(t)$ to the mass component.

Thus $f_s + f_D + f_I = p(t)$……………………………………(10)

The force f_s is associated with displacement x such that

f_s= kx……………………………………………………..(11)

where k is the stiffness matrix of the structure; it is a symmetric matrix(i.e, $k_{ij} = k_{ji}$).

The force f_D is associated with velocity $\dot{x}$ such that

f_D= c$\dot{x}$………………………………………………………….(12)

where c is the damping matrix of the structure

f_I is associated with acceleration $\ddot{x}$ such that

f_I= m$\ddot{x}$………………………………………………………….(13)

Substituting eqns (11), (12) and (13) in eqn(10) gives

$[M]\{\ddot{X}\}+ [C]\{\dot{X}\} + [K]\{X\} = p(t)$

Where,

[M] = The global mass matrix of the 2D frame structure

[C] = The global damping matrix of the frame structure (Assumed to be a zeromatrix, as damping is neglected in the structure)

[K] = The global stiffness matrix of the 2D frame structure

{X} = The global nodal displacement vector

p(t) = External force

a) Element Matrices:

The element stiffness matrix for a frame structure is given by:

$$[k] = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12\mathrm{EI}}{\mathrm{L3}} & \frac{6EI}{L2} & 0 & -\frac{12EI}{L3} & \frac{6EI}{L2} \\ 0 & \frac{6EI}{L2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L3} & -\frac{6EI}{L2} & 0 & \frac{12EI}{L3} & \frac{6EI}{L2} \\ 0 & \frac{6EI}{L2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L2} & \frac{4EI}{L} \end{bmatrix}$$

Where

E = Young's Modulus of the frame element.

A = Cross sectional area of the element.

L = Length of the element.

The element mass matrix for a frame structure is given by:

$$[m]= \begin{bmatrix} 2ma & 0 & 0 & ma & 0 & 0 \\ 0 & 156mm & 22Lmm & 0 & 54mm & -13Lmm \\ 0 & 22Lmm & mm & 0 & 13Lmm & -3L^2mm \\ ma & 0 & 0 & 2ma & 0 & 0 \\ 0 & 54mm & 13Lmm & 0 & 156mm & -22Lmm \\ 0 & -13Lmm & -3L^2mm & 0 & -22Lmm & 4L^2mm \end{bmatrix}$$

Where mm = ρAL/420 and ma = ρAL/6

ρ = Density of the material.

b)Element Matrices Global Co-ordinate Systems:

The matrices formulated in the previous section are for a particular frame element in a specific orientation. A full frame structure usually comprises numerous frame elements of different orientations joined together. As such, their local coordinate system would vary from one orientation to another. To assemble the element matrices together, all the matrices must first be expressed in a common coordinate system, which is the global coordinate system.

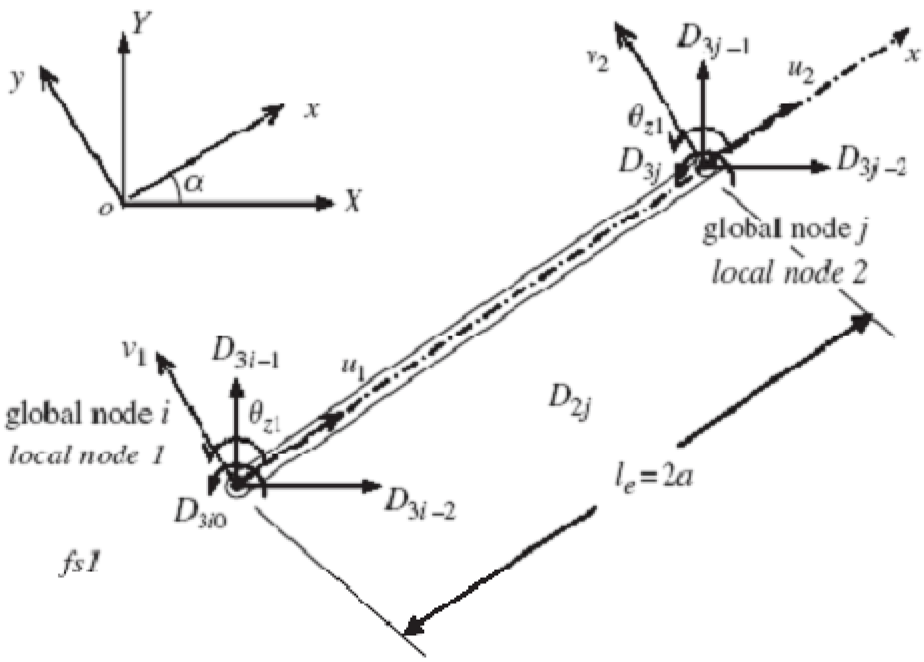


Fig 3.5 Co-ordinate transformation for 2D frame elements

Assuming that the local nodes 1 and 2 correspond to the global nodes i and j,respectively. Let T be the transformation matrix for the frame element given by

[T] =

$$\begin{bmatrix} l_x & m_x & 0 & a & 0 & 0 \\ l_y & m_y & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & l_x & m_x & 0 \\ 0 & 0 & 0 & l_y & m_y & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Where,

l_x = cos(x , X) = cos∝ = $\frac{X_j - X_i}{l_e}$

m_x = cos(x , Y) = sin∝ = $\frac{Y_j - Y_i}{l_e}$

l_y = cos(y , X) = cos(90+∝) = -sin∝ = - $\frac{Y_j - Y_i}{l_e}$

m_y = cos(y , Y) = cos∝ = $\frac{X_j - X_i}{l_e}$

Here, ∝ = the angle between x-axis and the X-X axis as shown in the figure

$l_e = \sqrt{(X_j - X_i)^2 - (Y_j - Y_i)^2}$

Using the transformation matrix, T, the matrices for the frame element in the global coordinate system become

$K_e = T^T kT$

$M_e = T^T mT$

c) Application of Boundary Conditions:

The boundary conditions are imposed on the structure by cancellation of the corresponding rows and columns in the stiffness as well as in mass matrix.

3.4) Forced Vibration analysis of TMD-Structure interaction problem

For frame structure the equation of motion of this tmd–structure system can

be written in the following form.

$[M]\{\ddot{X}\} + [C]\{\dot{X}\} + K\{X\} = -[M]\{\ddot{X}_g\} + \{F_{tmd}\}$

where,

[M] = The global mass matrix of the 2D frame structure

[C] = The global damping matrix of the frame structure (Assumed to be a zeromatrix, as damping is neglected in the structure)

[K] = The global stiffness matrix of the 2D frame structure

{X} = The global nodal displacement vector

$\{\ddot{X}_g\}$= Ground Acceleration

$\{F_{tmd}\}$=Resisting force to the structure at corresponding nodes due to TMD

3.5.1)Solution of Forced vibration problem using Newmark BetaMethod:

The forced vibration problem can be solved by Newmark Beta Method, alsoknown as the constant average acceleration method.

For the solution of the displacement, velocity and acceleration at time t+Δt are also considered,

$[M_f]^{t+\Delta t}\{\ddot{P}\} + [K_f]^{t+\Delta t}\{P\} = \{F_{el}\}^{t+\Delta t}$

The algorithm of the scheme is highlighted below:

Initial calculations:

1)Formulation of Global Stiffness matrix K and Mass matrix M

2)Initialization of P and $\ddot{P}$

3)Selection of time step Δt and parameters β' and α'

$\alpha' \geq 0.5$ and $\beta' \geq 0.25(0.5+\alpha')^2$

$\alpha'=0.5$ and $\beta'=0.25$ are taken in the present analysis

4)Calculation of coefficients for the time integration

$a_0 = 1/\beta'\Delta t^2$; $a_1 = \alpha'/\beta'\Delta t$; $a_2 = 1/\beta'\Delta t$; $a_3 = (1/2\beta')-1$

$a_4 = (\alpha'/\beta')-1$; $a_5 = (\Delta t/2)((\alpha'/\beta')-2)$; $a_6 = \Delta t(1-\alpha')$; $a_7 = \alpha'\Delta t$

5) Computation of effective stiffness matrix $\hat{K}$

$\hat{K} = K + a_0 M$

For each time step:

1) Calculation of effective load vector

$\hat{F}_{t+\Delta t} = F_{t+\Delta t} + M(a_0 P_t + a_2 \dot{P}_t + a_3 \ddot{P}_t)$

2) Solution for pressure at time $t+\Delta t$

$\widehat{K}P_{t+\Delta t} = \widehat{F}_{t+\Delta t}$

3) Calculation of time derivatives of pressure (P) at time t+Δt

$\ddot{P}_{t+\Delta t} = a_0(P_{t+\Delta t} - P_t) - a_2\dot{P}_t - a_3\ddot{P}_t$

and

$\dot{P}_{t+\Delta t} = \dot{P}_t + a_6\ddot{P}_t + a_7\ddot{P}_{t+\Delta t}$

CHAPTER-4

RESULTS AND DISCUSSIONS

4.1) Shear building:

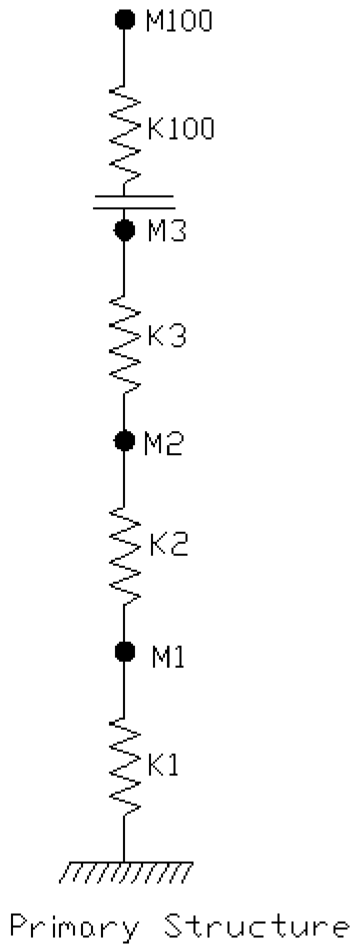


Fig 4.1 Shear Building

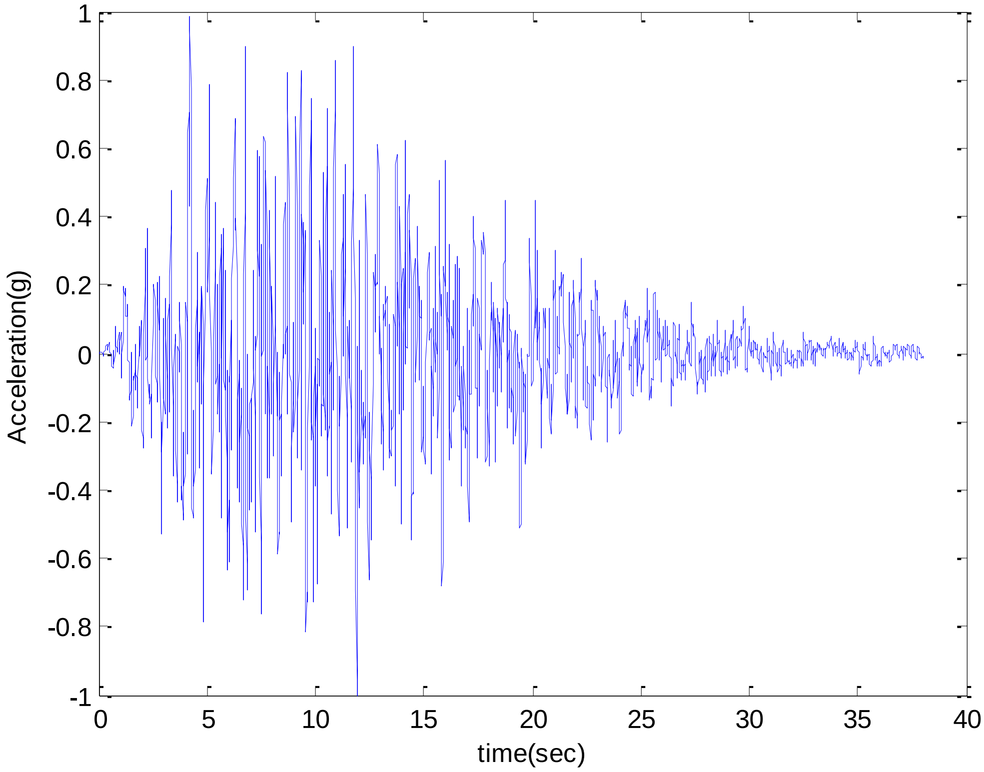
Assumed preliminary data for fig 4.1

1) Modulus of elasticity $22360.6\times10^{6}N/m^{2}$
2) Height of each storey 3500 mm
3) Area of each storey 250 mm × 450 mm

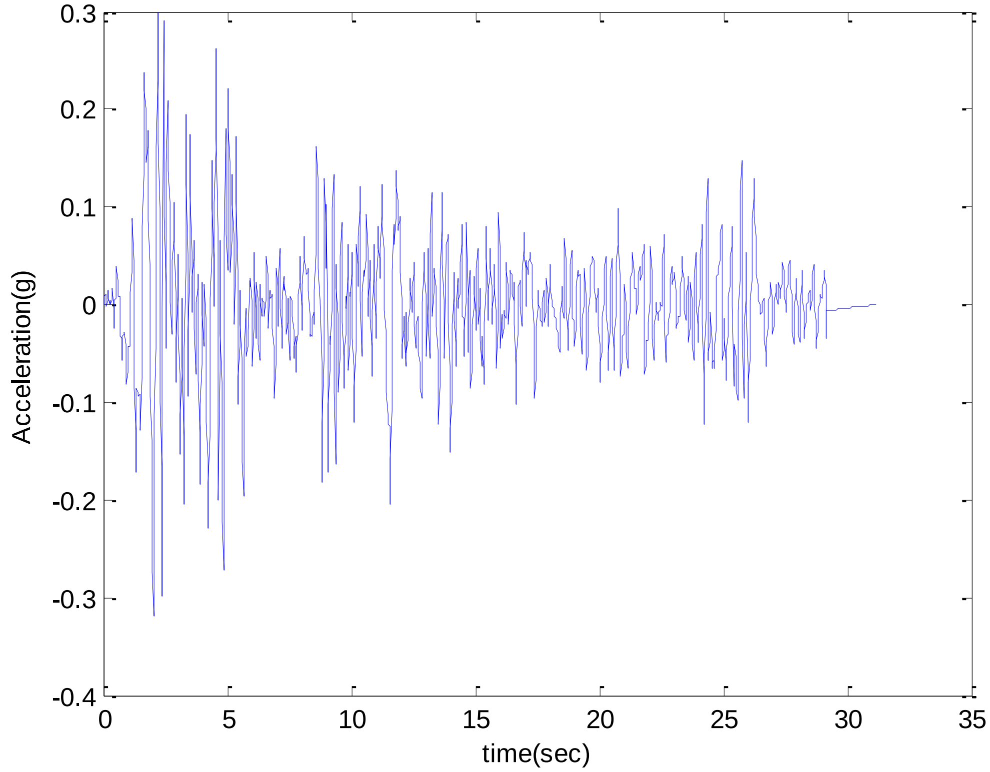
4.2) Forced vibration analysis of shear building

4.2.1) Time Histories of Random Ground Acceleration:

A total of two random ground acceleration cases are considered for the analysis. The first is the compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil. (PGA = 1.0g). The second is the 1940 El Centro Earthquake record (PGA = 0.313g).



a) Compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil



b) 1940 El Centro EQ Time History

Fig 4.2) Acceleration Time histories of past earth quakes

4.2.2 Response of shear building to Random Ground Acceleration

The above mentioned time histories are applied on the structure. The response of the structure is measured in terms of amplitude of displacement of the 100th storey.

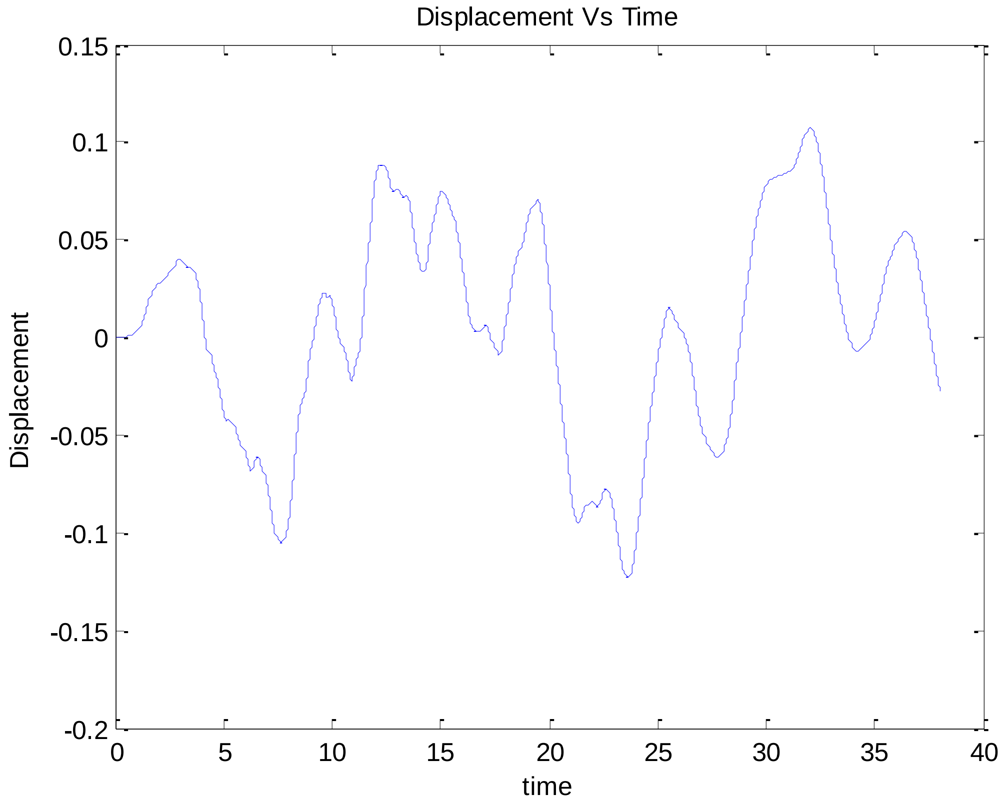


Fig 4.3a) Response of shear building to Compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil

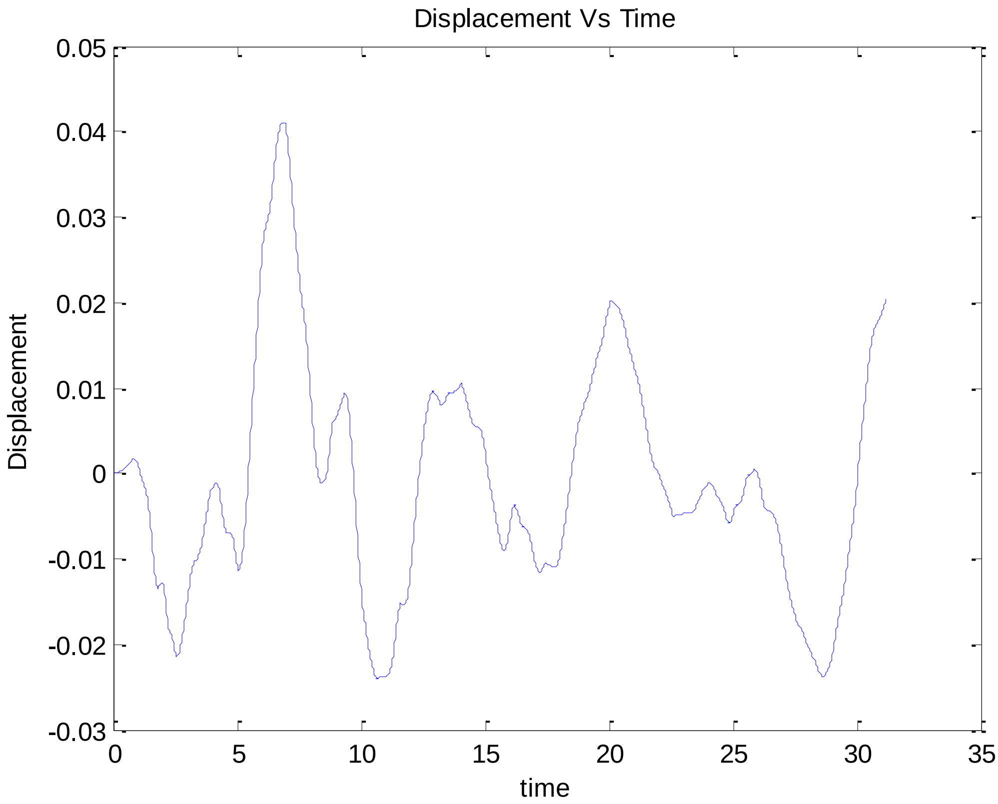


Fig 4.3 b) Response of shear building to the 1940 El Centro earthquake

4.3) TMD-Structure interaction

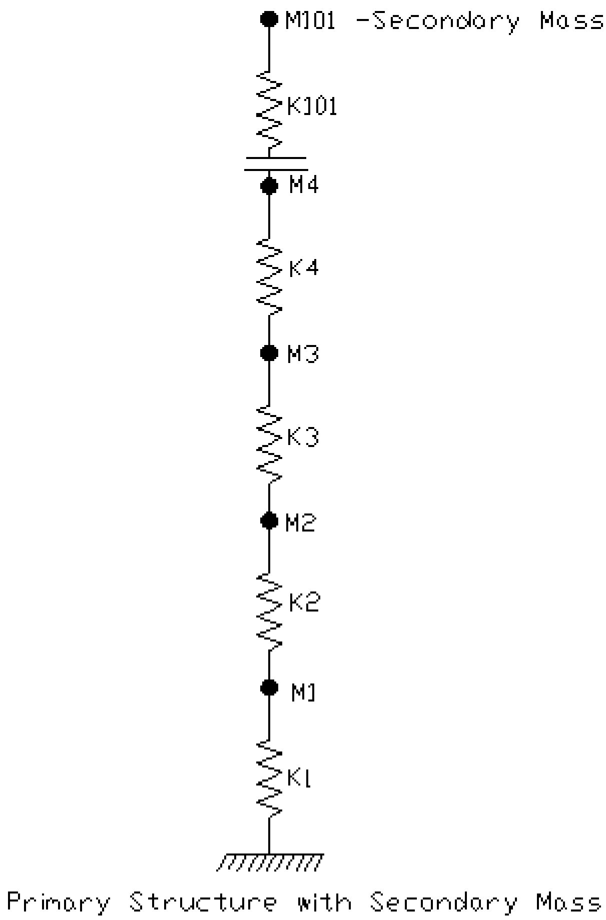


Fig 4.4 Damper-Structure Arrangement for shear building

4.3.1) Effect of TMD in structural damping when damping ratio of the structure is varied for shear building

To study the effect of TMD on reducing the response of the structure to seismic loading compatible time history as per spectra of IS 1894-2002(Part-1) and 1940 El Centro earth quakes are applied to the structure. The damping of the TMD is kept at 2% while the damping of the structure is varied from 2% to 5%.

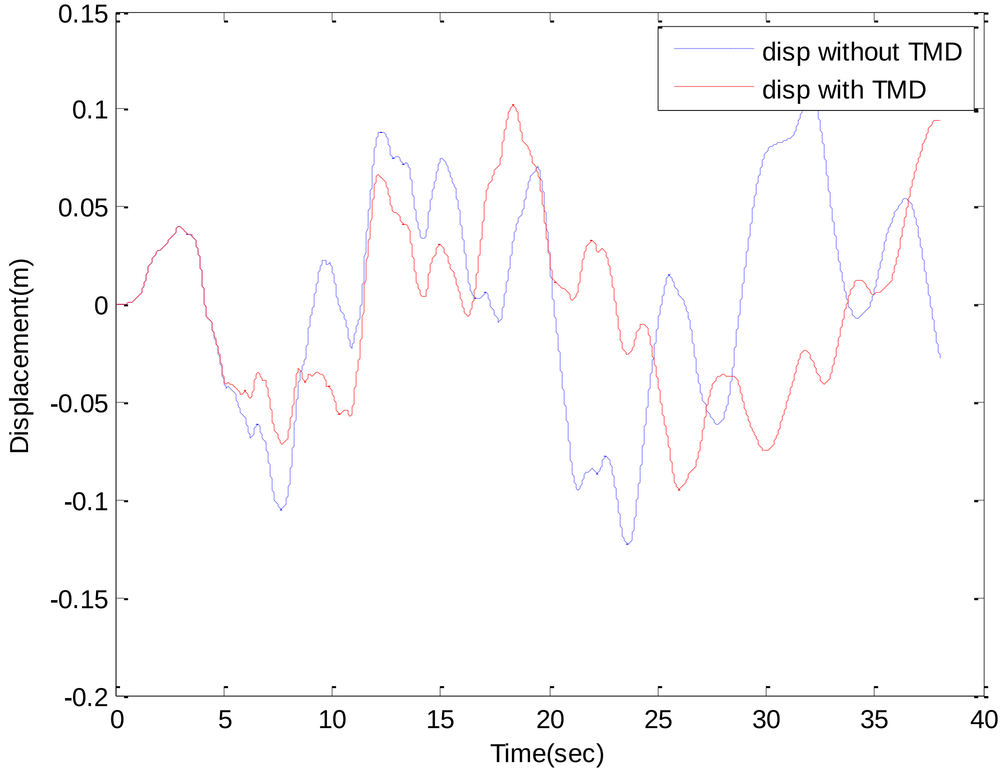


Fig 4.5 a) Response of the structure when damping ratio of the structure is 2%

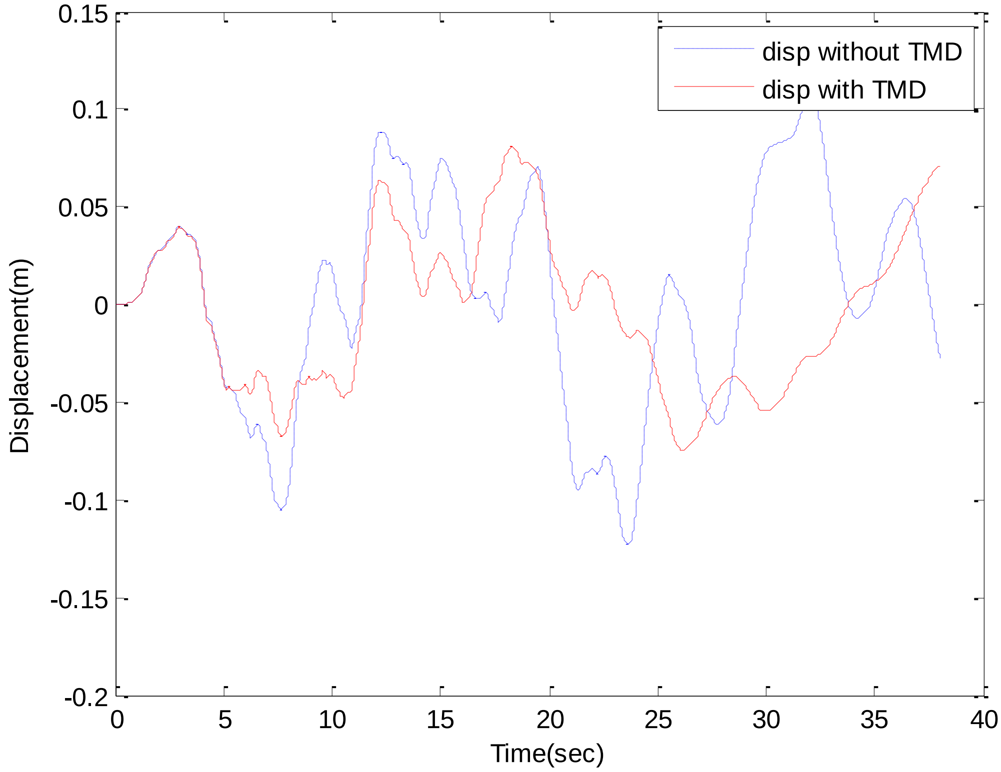


Fig 4.5 b) Response of the structure when damping ratio of the structure is 5%

Fig 4.5 Amplitude of vibration at top storey by placing TMD at top storey with variation of damping ratio of the structure when corresponding to compatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil acting on the structure

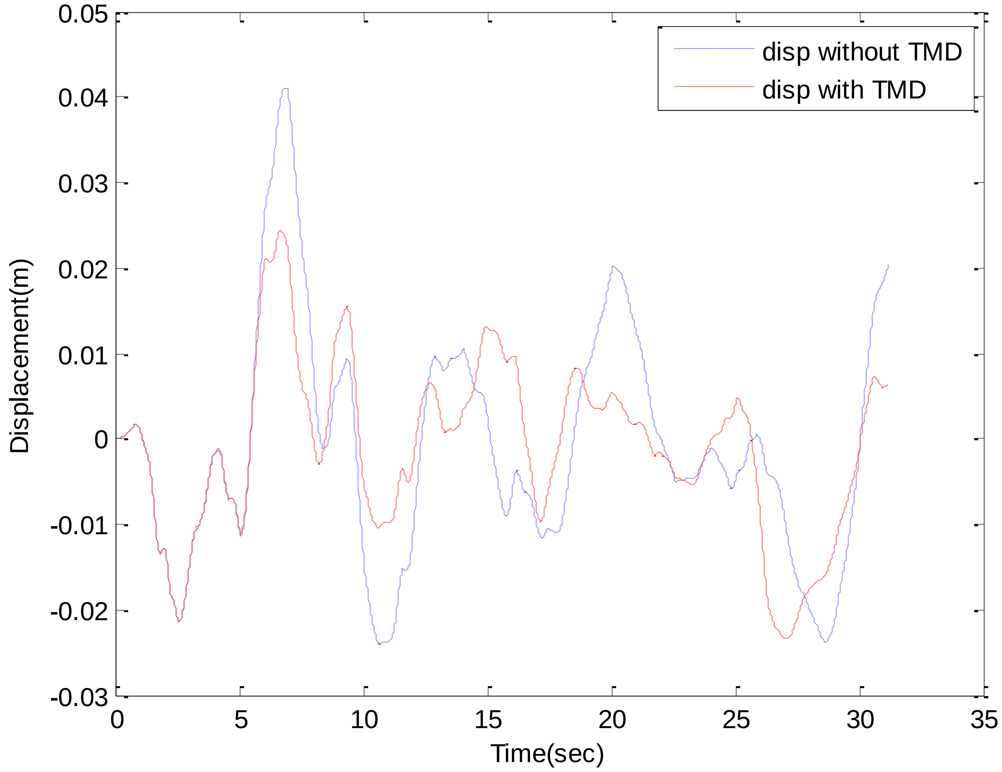


Fig 4.6 a) Response of the structure when damping ratio of the structure is 2%

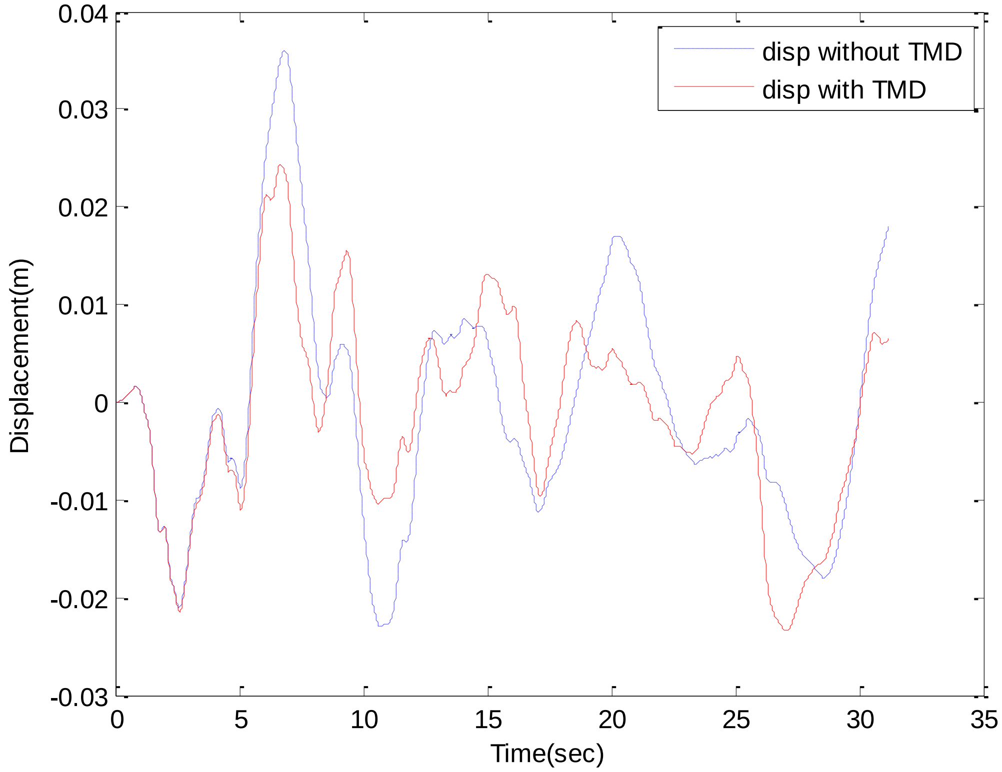


Fig 4.6 b) Response of the structure when damping ratio of the structure is 5%

Fig 4.6 Amplitude of vibration at top storey by placing TMD at top storey with variation of damping ratio of the structure when, El Centro (1940) earthquake loading acting on the structure.

From the above figures it can be concluded that TMD is more effective in reducing the displacement responses of structures with low damping ratios (2%). But, it is less effective for structures with high damping ratios(5%).

4.3.2)Effect of TMD on structural damping with variation of mass ratio:

A study has been carried out to see the effect of variation of mass ratio by keeping the damping of TMD and structure constant at 2% and considering four mass ratios and two earthquake loads.

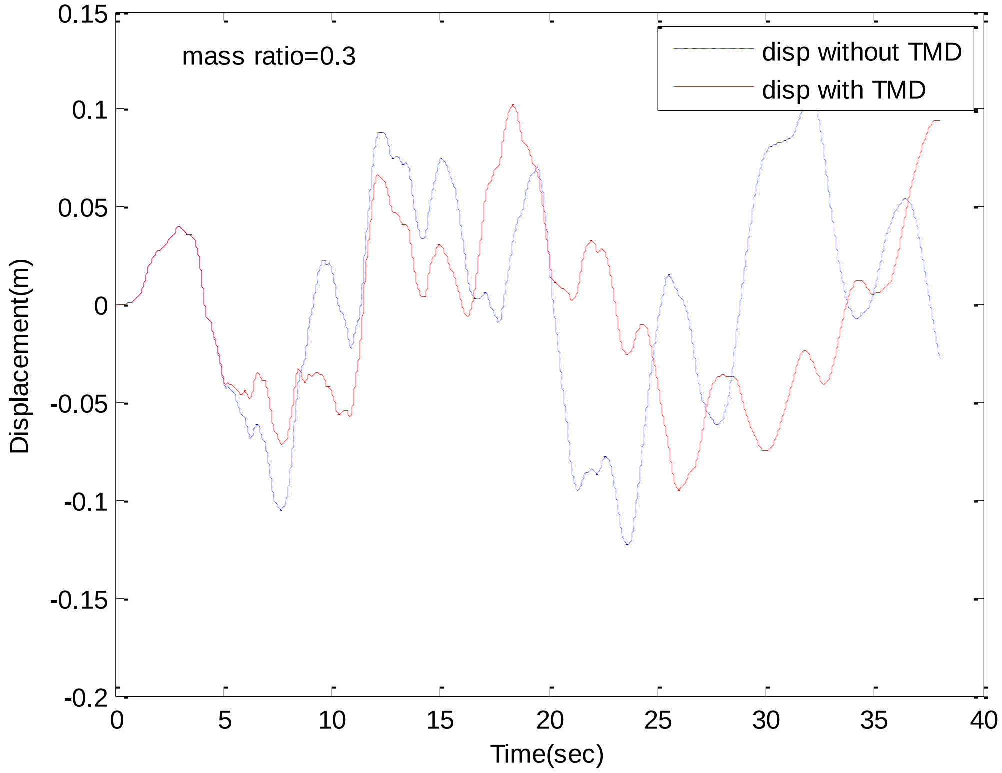


Fig 4.7 a) Response of the structure with TMD with 0.3 mass ratio

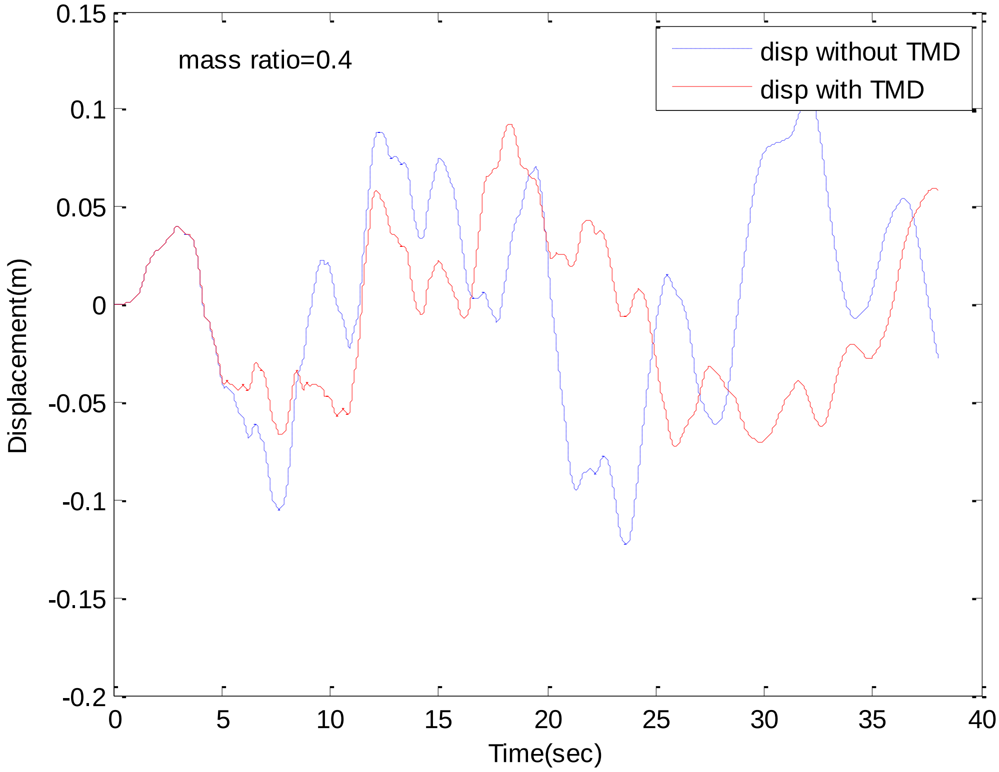


Fig 4.7 b) Response of the structure with TMD with 0.4 mass ratio

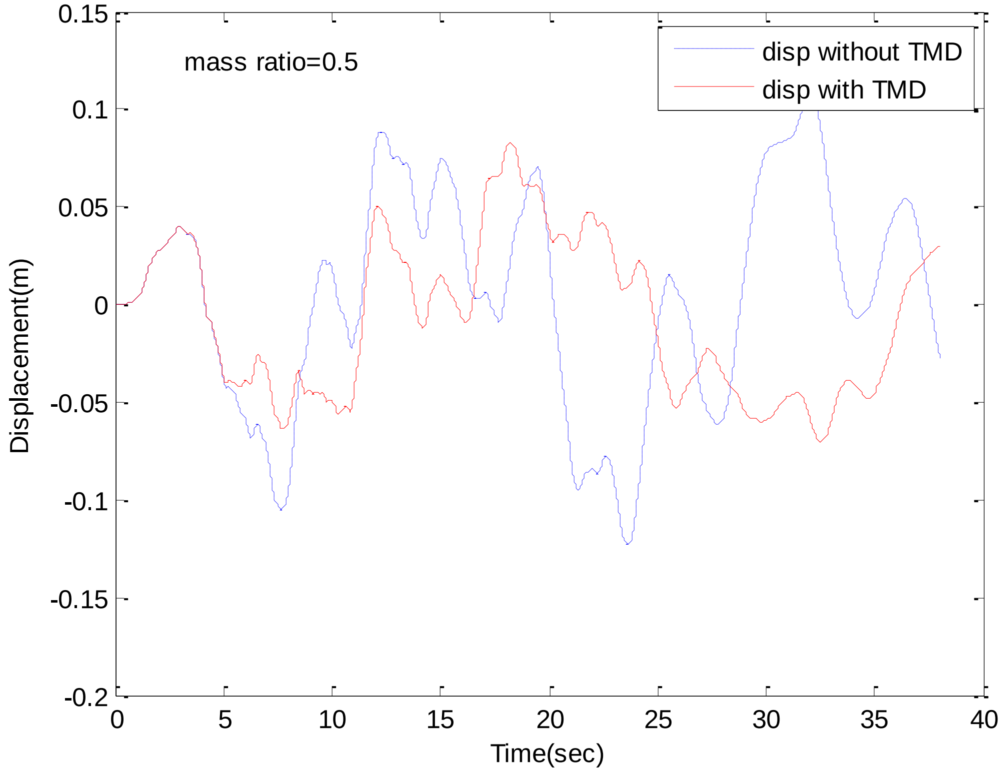


Fig 4.7 c) Response of the structure with TMD with 0.5 mass ratio

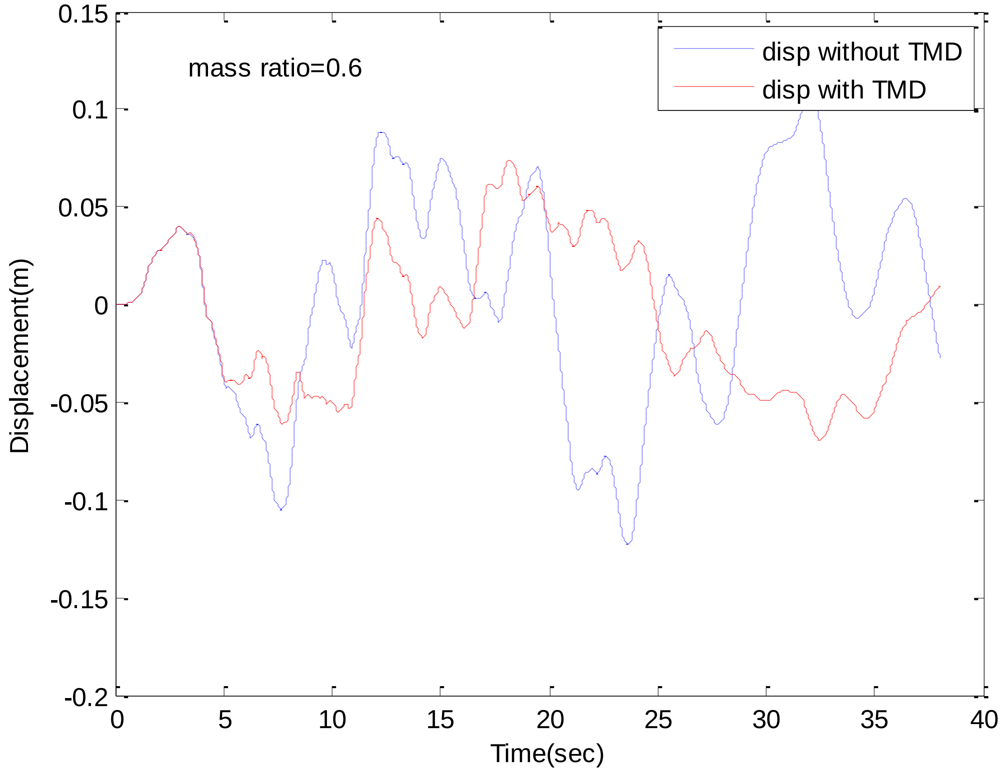


Fig 4.7 d) Response of the structure with TMD with 0.6 mass ratio

Fig 4.7) Amplitude of vibration at top storey by placing TMD at top storey with variation of mass ratio of the TMD when corresponding to compatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil acting on the structure.

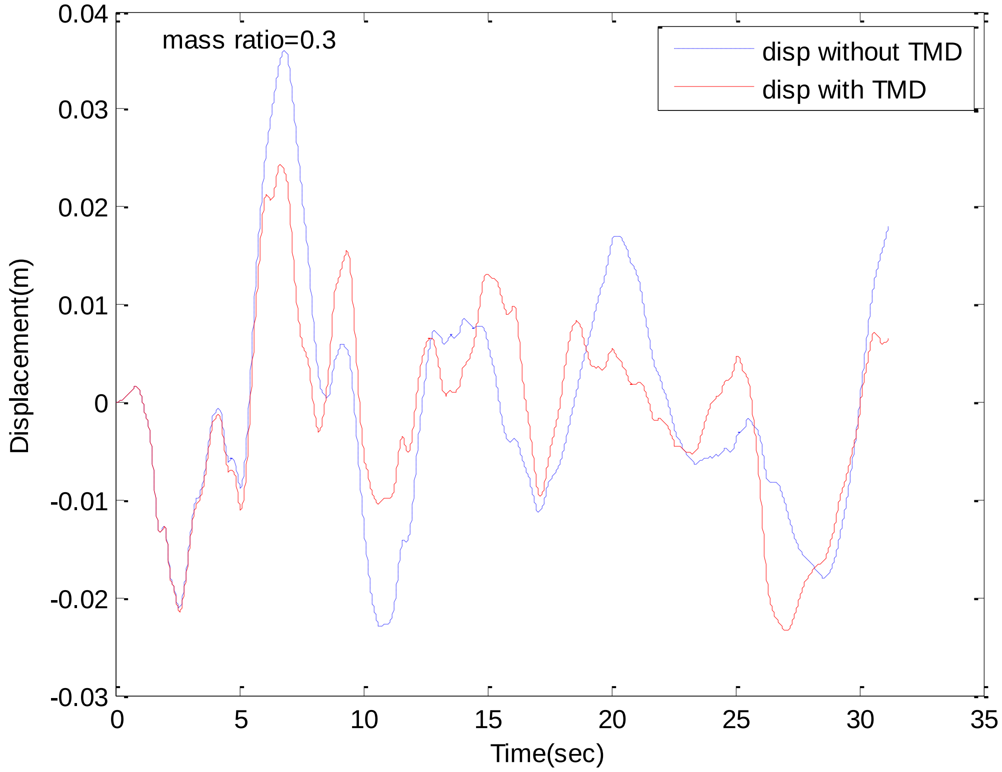


Fig 4.8 a) Response of the structure with TMD with 0.3 mass ratio

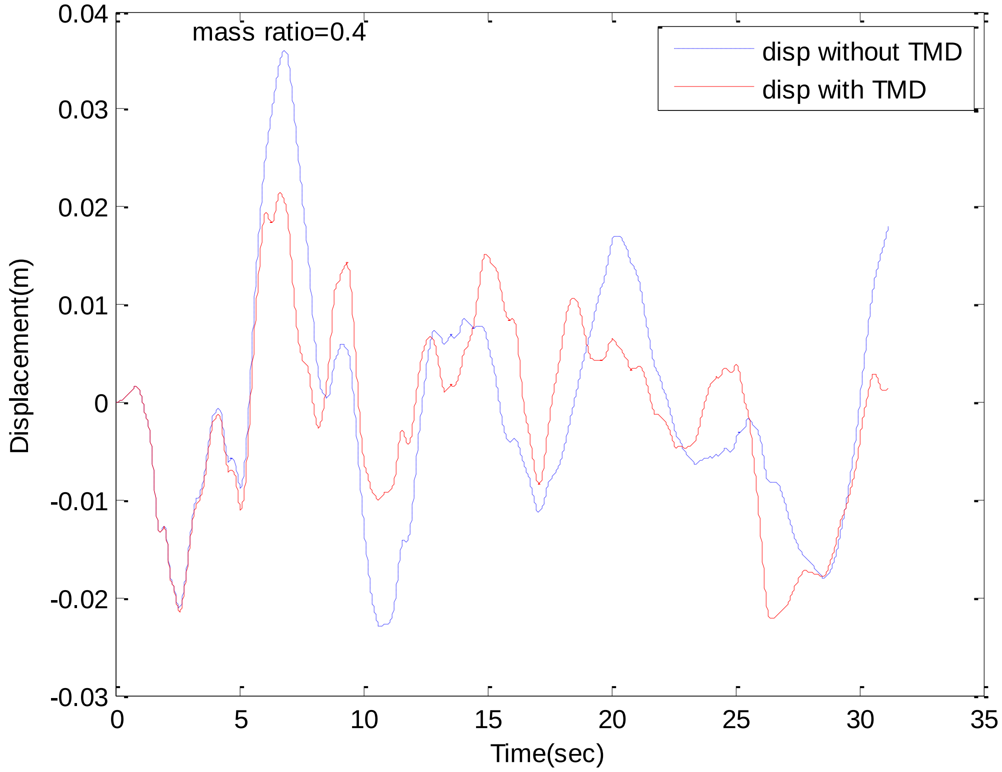


Fig 4.8 b) Response of the structure with TMD with 0.4 mass ratio

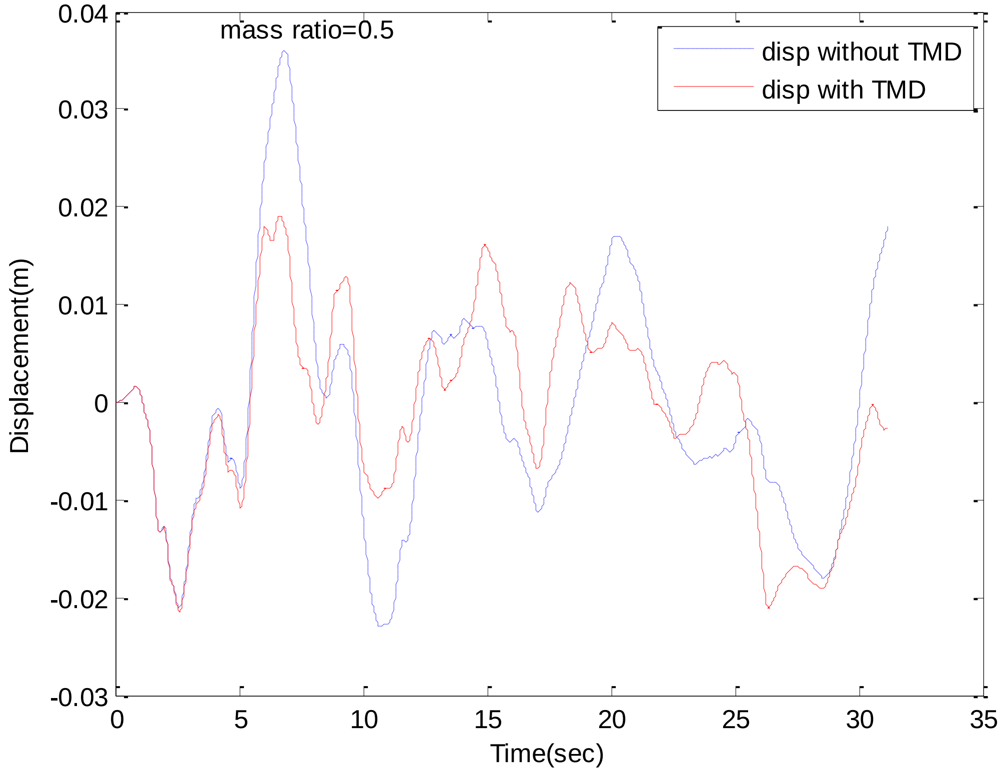


Fig 4.8 c) Response of the structure with TMD with 0.5 mass ratio

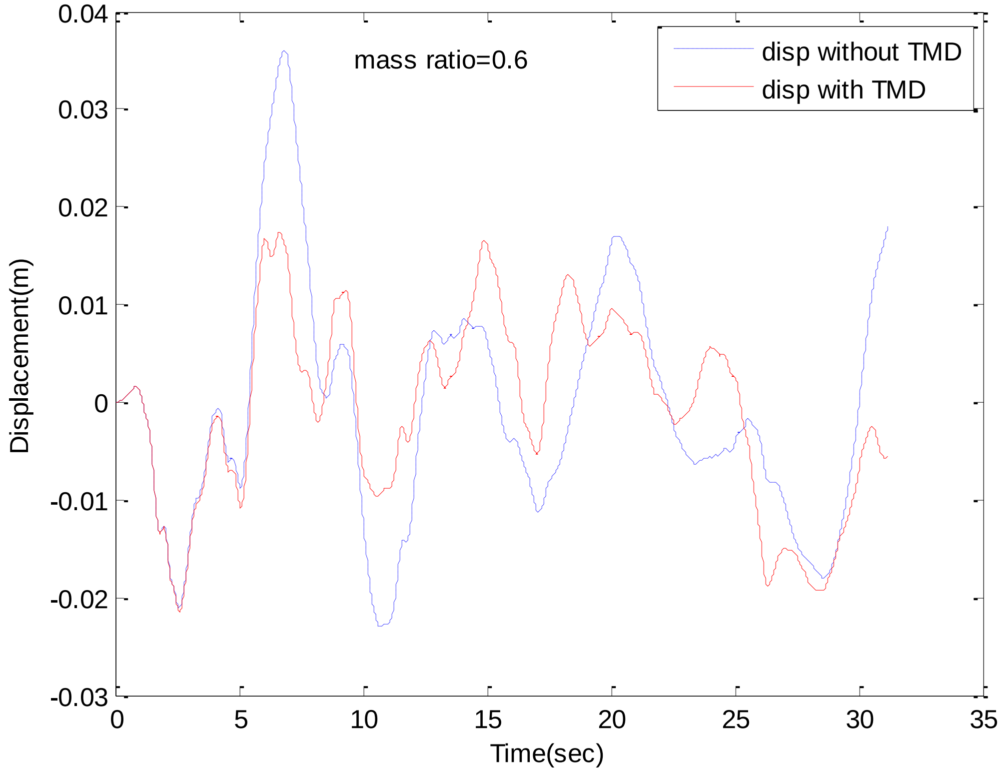


Fig 4.8 d) Response of the structure with TMD with 0.6 mass ratio

Fig 4.8 Amplitude of vibration at top storey by placing TMD at top storey with the variation of the mass ratio of the TMD when, El Centro(1940) earthquake loading acting on the structure.

It can be concluded from the above graphs that increasing the mass ratio of the TMD decreases the displacement response of the structure.

4.4) Two Dimensional MDOF frame model

4.4.1) Problem Statement:

A multistorey plane frame having storeys of height 'H' and bays of length 'L' is analysed. The 2D frame model is discretized into a number of elements, We can consider infinite numbers of nodes in each element such that inc = number of intermediate nodes per each column, inb = number of intermediate nodes per beam. Three degrees of freedom i.e, two translations and one rotation are associated with each node.

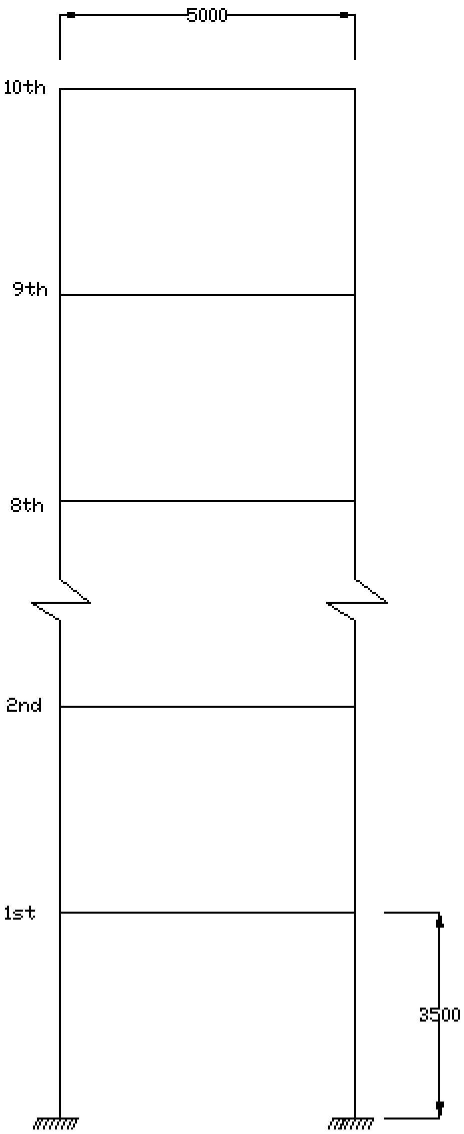


Fig 4.9 Elevation of 2D plane frame structure

The following data are taken for analysis of the above frame:

1)Type of the structure Multi-storey rigid jointed plane frame.

2) Size of columns 0.250 m × 0. 450m

3) Size of beams 0.250m× 0.400m in longitudinal direction

4) Depth of slab 0.100 mm

5) Modulus of elasticity $22360.6\times10^{6}N/m^{2}$

4.5) Preliminary Calculations:

1) MOI of Column = $0.25 \times 0.45^3/12 = 1.9 \times 10^{-3}\ m^4$

2) MOI of Beam = $0.25 \times 0.4^3/12 = 1.33 \times 10^{-3}\ m^4$

3) Loading on column per unit length

a)Self weight of column =$0.25 \times 0.45 \times 25 \times 1000$ N = 2812.5 N

4)Loading on column per unit length

a)Self weight of beam =$0.25 \times 0.4 \times 25 \times 1000$ N = 2500 N

b)Weight of slab =$0.1 \times 5 \times 25 \times 1000$ N = 12500 N

c)Live load on slab =$5 \times 3.5 \times 1000$ N = 17500 N

Total weight per metre length of beam =2500+12500+17500 N=32500 N

4.6) Free Vibration Analysis of the Multi-storey frame:

4.6.1) Convergent study for Natural frequencies of the structure

Table 4.1 Convergent study for Natural frequencies of the structure(No of storey = 5, No of bay = 1, Height of each storey = 3.5 m)

Modes	Natural frequencies in(rad/sec)				
	No of elements				
	30	45	60	75	90
1^{st}	13.424	13.424	13.424	13.424	13.424
2^{nd}	43.513	43.511	43.510	43.510	43.510
3^{rd}	81.725	81.709	81.706	81.704	81.704
4^{th}	126.600	126.545	126.532	126.527	126.525
5^{th}	167.534	167.015	166.924	166.899	166.889

4.6.2) Variation of Natural frequencies with increase in number of storey

Table 4.2 Variation of Natural frequencies with increase in number of storey (No of Bay = 1,Height of each storey=3.5 m and Width of each Bay = 5 m) when inc=inb = 5

Modes	Natural frequencies(rad/sec)				
	No of storeys				
	1	2	3	4	5
1st	86.433	39.970	25.113	18.125	14.102
2nd	230.385	135.935	85.117	59.917	45.708
3rd	552.9759	213.111	159.087	113.867	85.832
4th	592.899	257.433	207.109	171.342	132.918
5th	788.338	470.714	237.3117	196.733	175.321

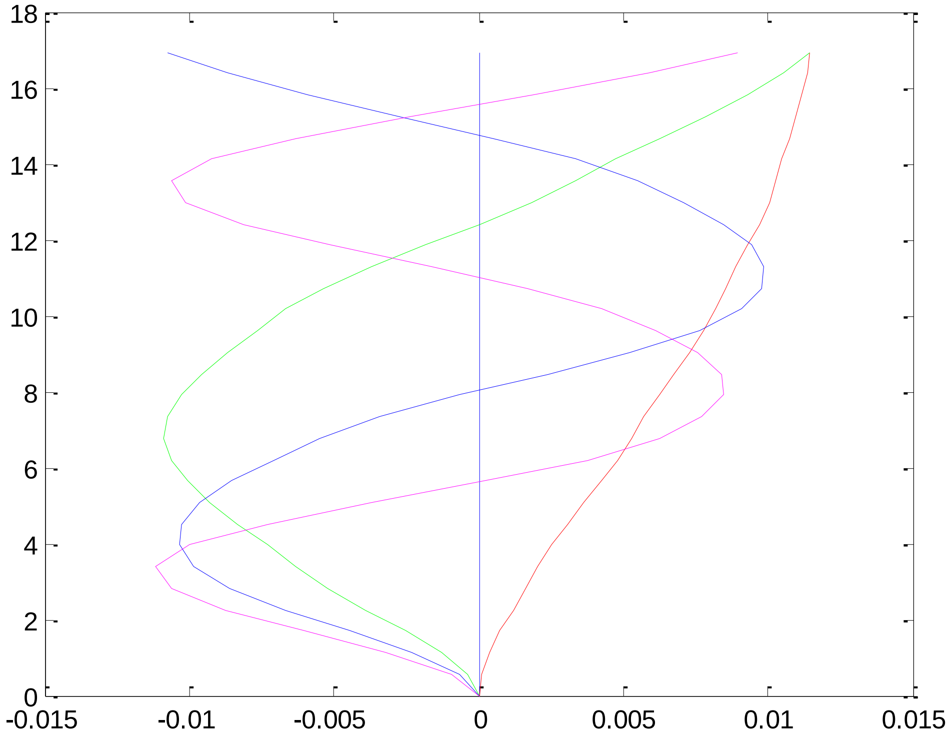


Fig 4.10 First four mode shapes for the frame structure

4.7)Forced vibration analysis of the Multi-storey frame:

4.7.1)Response of structure to Harmonic Ground Acceleration:

Forced Vibration analysis is carried out for the structure.The structure issubjected to a sinusoidal forced horizontal base acceleration given by:

$a_x(t) = X_0 \sin(\omega t)$

where , X_0 and ω are the amplitude and frequency of the sinusoidal excitation respectively. The structure is discretized into 60 elements. The response of the structure at 10th storey are measured in terms of displacement, velocity, acceleration as shown in figure below;

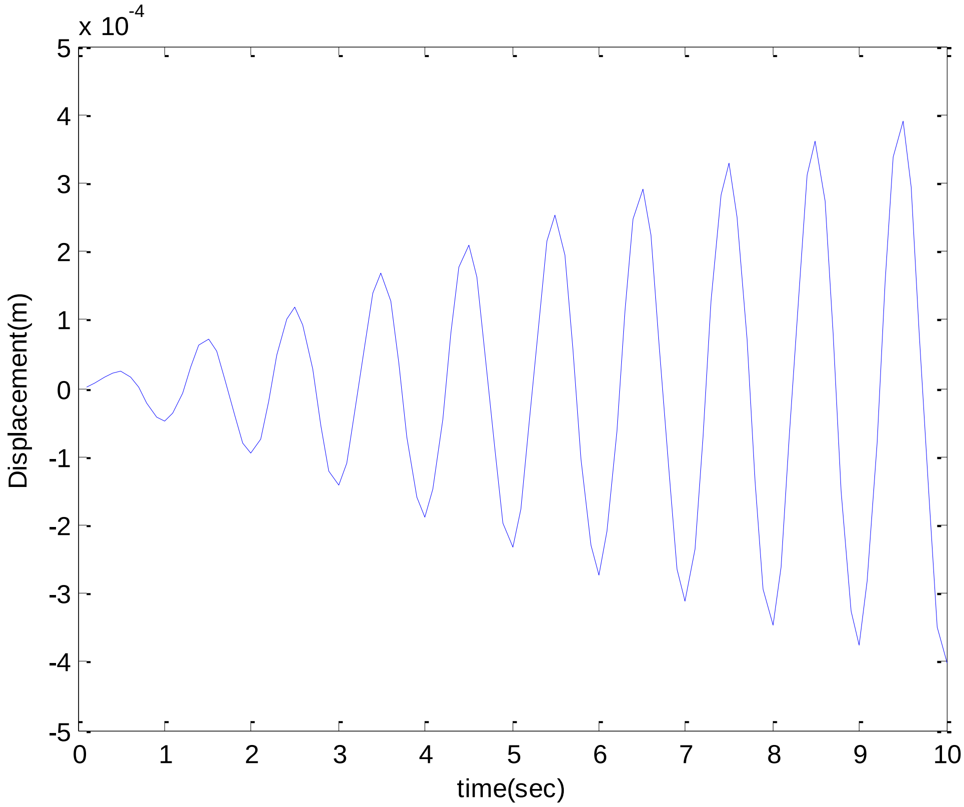


Fig 4.11 a) Displacement Vs Time

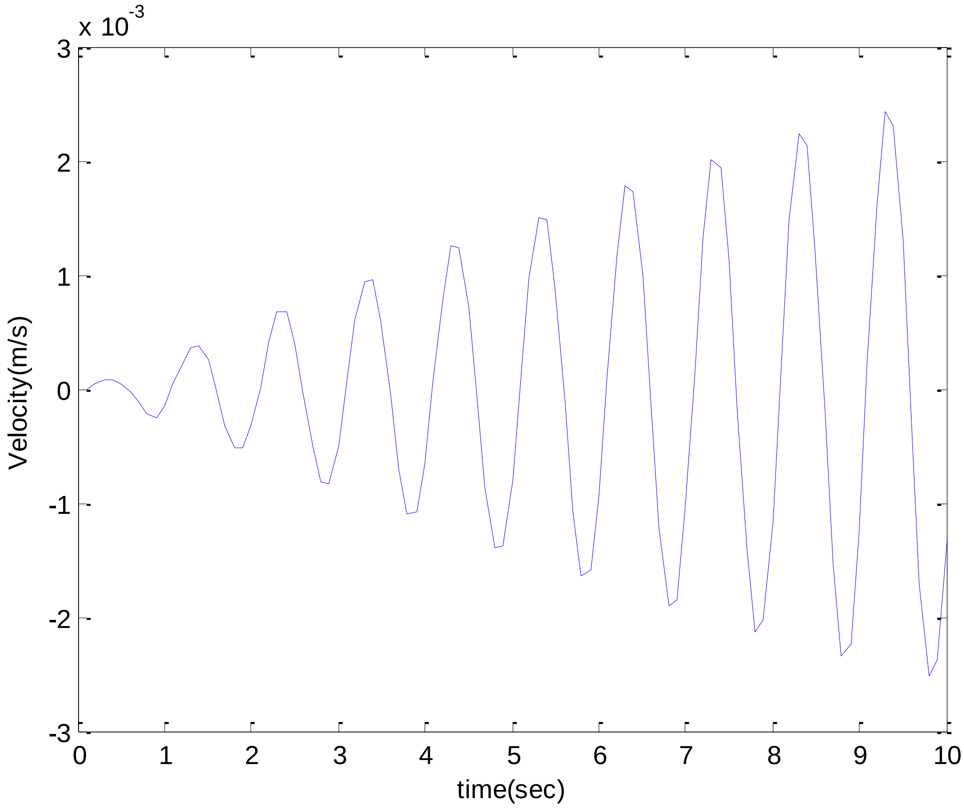


Fig 4.11 b) Velocity Vs Time

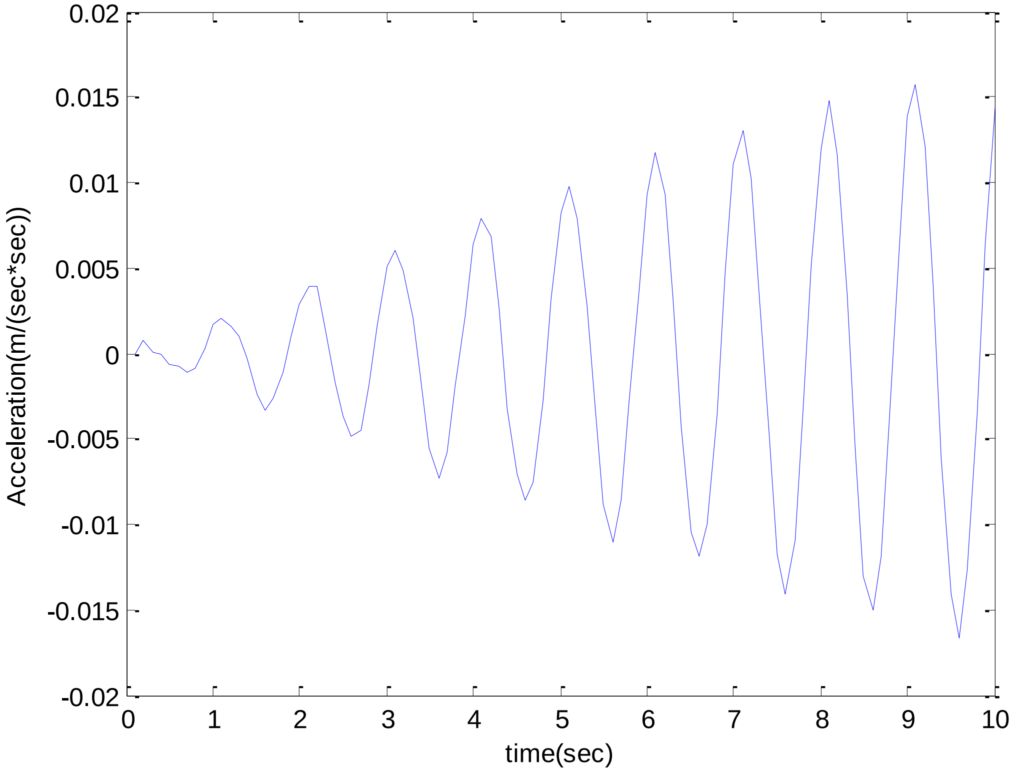


Fig 4.11 c) Acceleration Vs Time

Fig 4.11 Response of 10th storey of the structure to sinusoidal ground acceleration

4.7.2) Response of the 2D frame structure to Random Ground Acceleration:

The above mentioned time histories are applied on the multi-storey frame. The response of the structure is measured in terms of amplitude of displacement of extreme right node of the 10th storey.

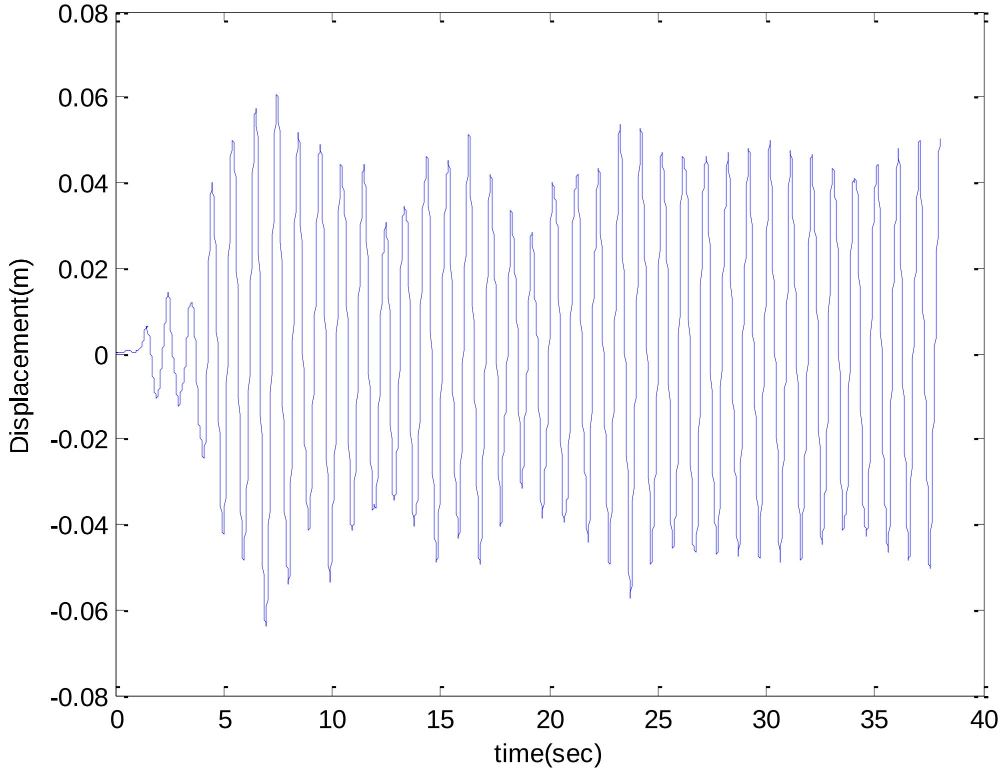


Fig 4.12 a) Response of the frame structure to Compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil.

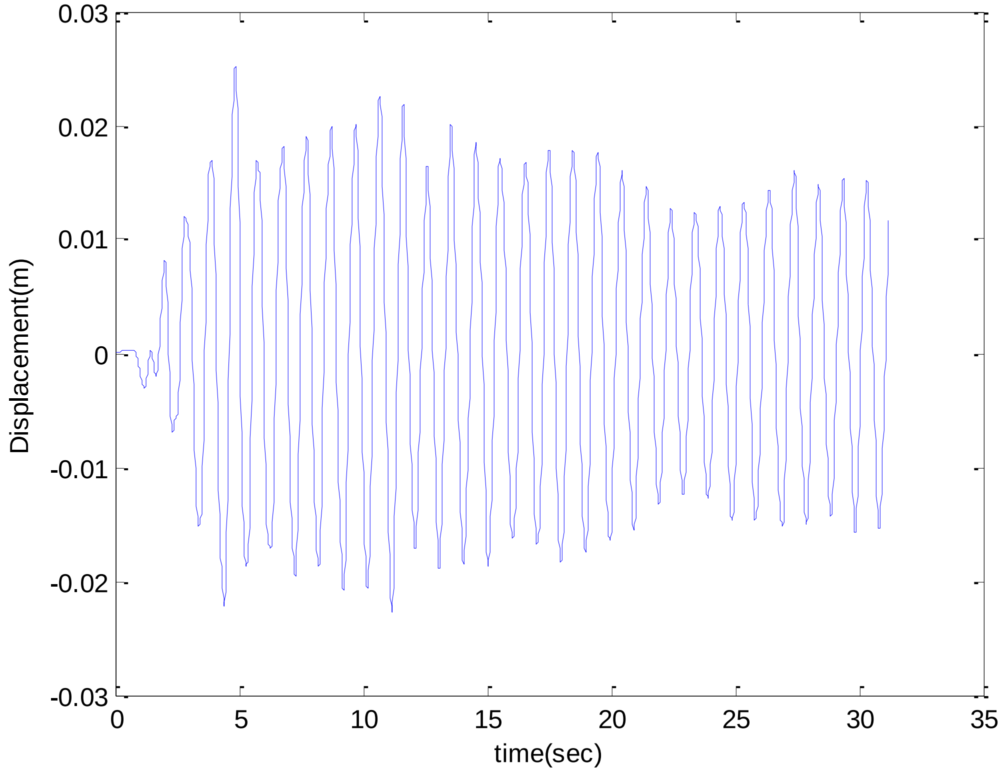


Fig 4.12 b) Response of the frame structure to 1940 El Centro earthquake

4.8) Two Dimensional MDOF frame model with TMD

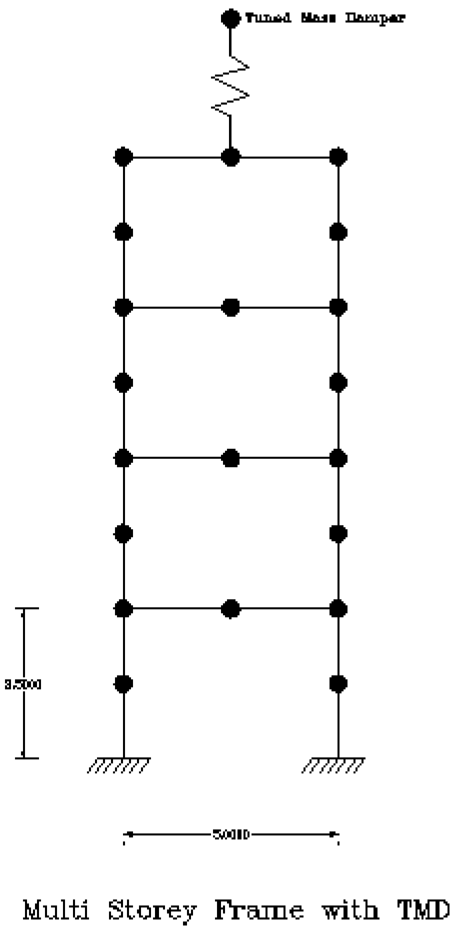


Fig 4.13 Damper Structure Arrangement for 2D frame

The TMD is placed at the 10th storey and the 2D frame structure is subjected to both corresponding to compatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil and 1940 El Centro earthquake load and the amplitudes of displacement is noted at the extreme right node of the 10th storey with TMD and without TMD. The TMD is having massratio=0.1 and tuning ratio=1

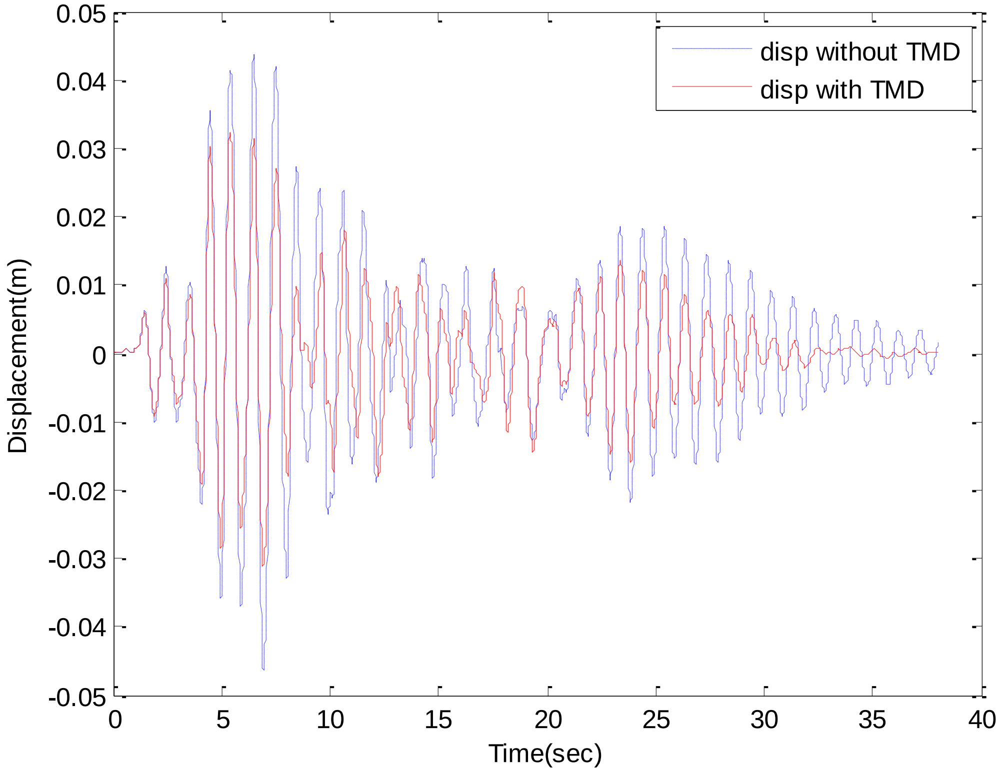


Fig 4.14 a) Amplitude of vibration at top storey of 2D frame by placing TMD at top storey when, corresponding to compatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil earthquake loading acting on the structure.

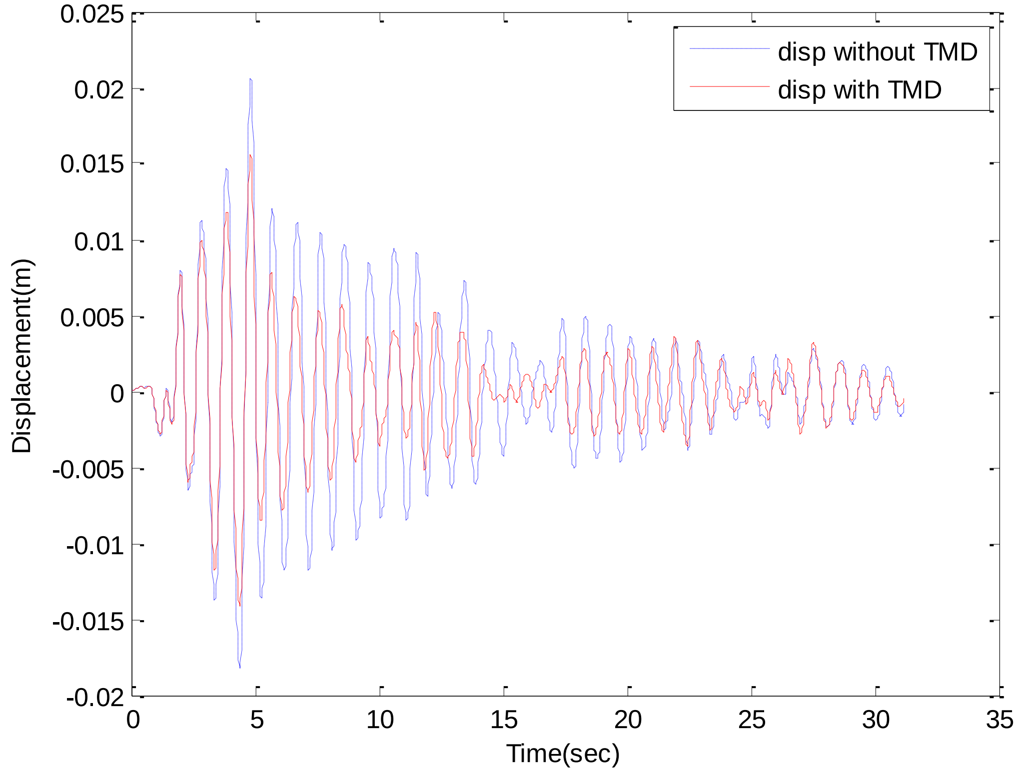


Fig 4.14 b) Amplitude of vibration at top storey of 2D frame by placing TMD at top storey when, El Centro(1940) earthquake loading acting on the structure.

Fig 4.14 Amplitude of vibration at top storey of 2D frame by placing TMD at top storey when subjected to different earthquake loadings.

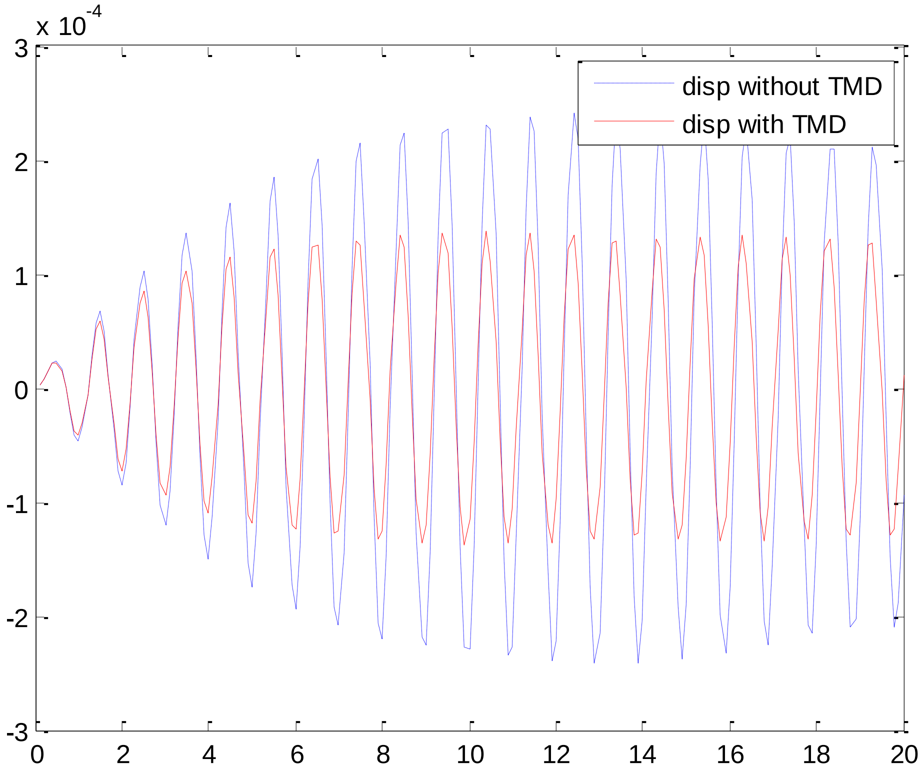


Fig 4.15 Amplitude of vibration at top storey of 2D frame by placing TMD at top storey when subjected to sinusoidal acceleration

CHAPTER-5

SUMMARY AND FURTHER SCOPE OF WORK

5.1) Summary:

Current trends in construction industry demands taller and lighter structures, which are also more flexible and having quite low damping value. This increases failure possibilities and also, problems from serviceability point of view. Several techniques are available today to minimize the vibration of the structure, out of which concept of using of TMD is one. This study is made to study the effectiveness of using TMD for controlling vibration of structure. A numerical algorithm was developed to model the multi-storey multi-degree of freedom building frame structure as shear building with a TMD. Another numerical algorithm is also developed to analyse 2D-MDOF frame structure fitted with a TMD. A total of three loading conditions are applied at the base of the structure. First one is a sinusoidal loading and the second one corresponding to compatible time history as per spectra of IS-1894(Part -1):2002 for 5% damping at rocky soil and the third one is 1940 El Centro Earthquake record (PGA = 0.313g).

Following conclusions can be made from this study:

1) It has been found that the TMD can be successfully used to control vibration of the structure.
2) TMD is more effective in reducing the displacement responses of structures with low damping ratios (2%). But, it is less effective for structures with high damping ratios (5%).
3) Applying the two earthquake loadings, first is the one corresponding to compatible time history as per spectra of IS-1894(Part -1):2002 for 5% damping at rocky soil

and second being the 1940 El Centro Earthquake it has been found that increasing the mass ratio of the TMD decreases the displacement response of the structure.

5.2) Further Scope for study

1) Both the structure and Damper model considered in this study are linear one; this provides a further scope to study this problem using a nonlinear model for TMD as well as for structure.

2) The frame model considered here is two-dimensional, which can be further studied to include 3-dimensional structure model.

3) Further scope, also includes studying the possibility of constructing Active TMD.

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