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Vibration control of framed structure under seismic excitations

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ABSTRACT: The necessity of designing structure is that they must be put in real world;a world full of uncertainty. Vibration is becoming the most important factor for framed structures, subjected to seismic excitation. Structural design codes and standards historically have been based strictly on professional experience, judgment and intuition. The code development process has been and tends to be evolutionary in nature. As various analytical and experimental methods were developed; new design procedure and code requirements were introduced or modified to incorporate & implement new knowledge. However Scientific Knowledge & engineering methodologies are still not sufficient to predict with confidence the value of structural load capacities & the complex inter relationships between them. In the present context an attempt is made to study the effectiveness of using tuned mass dampers for controlling vibration of structures by developing a numerical algorithm using MATLAB for one dimensional and two dimensional framed model fitted with tuned mass damper.

KEYWORDS: Tuned Mass Damper, vibration control, damping ratio

1 INTRODUCTION

Tuned mass damper is attached to a vibrating structure to reduce undesirable vibrations. Tuned mass damper is a passive energy absorbing device consisting of a mass, a spring and a viscous damper. The mass is usually attached to the building via a spring-dashpot system and energy is dissipated by the dashpot as relative motiondevelops between the mass and the structure. The frequency of the damper is tuned to a particular structural frequency sothat when that frequency is excited, the damper will resonate out of phase with the structural motion. Till date tuned mass damperhave been installed in large number of structures all around the globe. The first structure in which tuned mass damperwas installed is the Centrepoint Tower in Sydney, Australia. There are two buildings in the United States equipped with tuned mass dampers; one is the Citicorp Centre in New York City and the other is the John Hancock Tower in Boston.Chiba Port Tower (completed in 1986) was the first tower in Japan to be equipped with a tuned mass damper.In Japan, countermeasures against traffic-induced vibration were carried out for two two-story steel buildings under an urban expressway viaduct by means of tuned mass dampers (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 tuned mass dampers with the mass ratio about 1%.

The tuned mass damperconcept 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 tuned mass damperwas 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). The initial theory was applicable for an un-damped 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 this technology in the context of structural control against earthquakes are unavoidable and need to be overcome. A new control approach-semi-active control device, which combines 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 Semi-Active tuned mass dampers can be traced to 1983. Hrovat et al.(1983) presented Semi-Active tuned mass damper, a tuned mass damperwith time varying controllable damping. Under identical conditions, the behavior of a structure equipped with Semi-Active tuned mass damperinstead of tuned mass damperis significantly improved. The control design of Semi-Active tuned mass damperis 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 tuned mass dampertuned 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 tuned mass dampers, more than one tuned mass damper in a given structure, each tuned to a different dominant frequency, can be used. The concept of multiple tuned mass dampers together with an optimization procedure was proposed by Clark (1988). Numerical and experimental studies have been carried out on the effectiveness of TUNED MASS DAMPERs in reducing seismic response of structures [for instance, Villaverde (1994)].

2 MULTISTOREY BUILDING MODELLED AS 1D SINGLE DEGREE OF FREEDOM MODEL

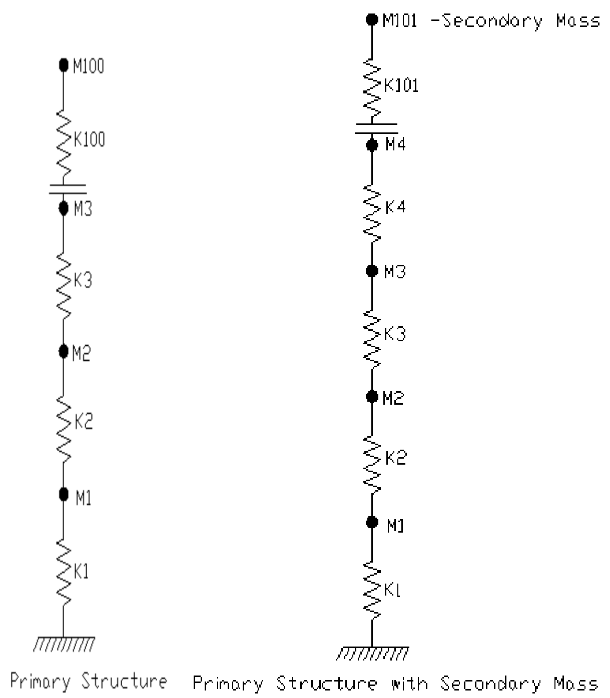

Figure 1. One dimensional single degree of freedom building and the building with tuned mass damper located at the top storey.

Considering a hypothetical multi-storey building modeled as a one dimensional single degree of freedom building. Figure 1 is a schematic diagram of the model and the model with tuned mass damper. The tuned mass damper is located at the top storey.

2.1 *Forced vibration analysis of onedimensional single degree of freedom model*

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).

2.1.1 *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.

The governing equation for the forced vibration analysis of the structure is given by

$$[M]^{t+\Delta t}\{\ddot{X}\} + [K]^{t+\Delta t}\{X\} = \{P\}^{t+\Delta t}$$

Where, [M] = The global mass matrix of the 2D frame structure; [K] = The global stiffness matrix of the 2D frame structure; {X} = The global nodal displacement vector; P = External force

The algorithm of the scheme is highlighted below:

1. Formulation of Global Stiffness matrix K and Mass matrix M
2. Initialization of X and $\ddot{X}$
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 $\hat{K}$

   $\hat{K} = K + a_0M$

For each time step:
Solution for displacement at time $t+\Delta t$

$$\hat{K}X_{t+\Delta t} = \hat{F}_{t+\Delta t}$$

Calculation of time derivatives of displacement(X) at time $t+\Delta t$

$$\ddot{X}_{t+\Delta t} = a_0(X_{t+\Delta t} - X_t) - a_2\dot{X}_t - a_3\ddot{X}_t$$

and

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

2.1.2 *Response of one dimensional model 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.

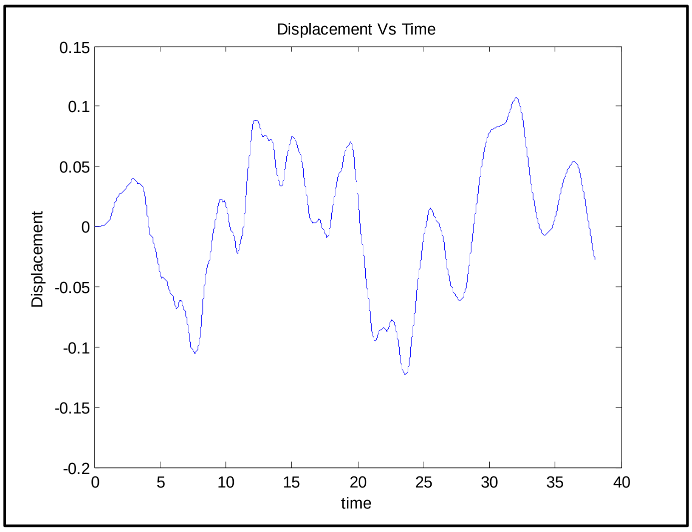

Figure 2a.Response of the one dimensional to Compatible time history as per spectra of IS-1894 (Part -1):2002 for 5% damping at rocky soil

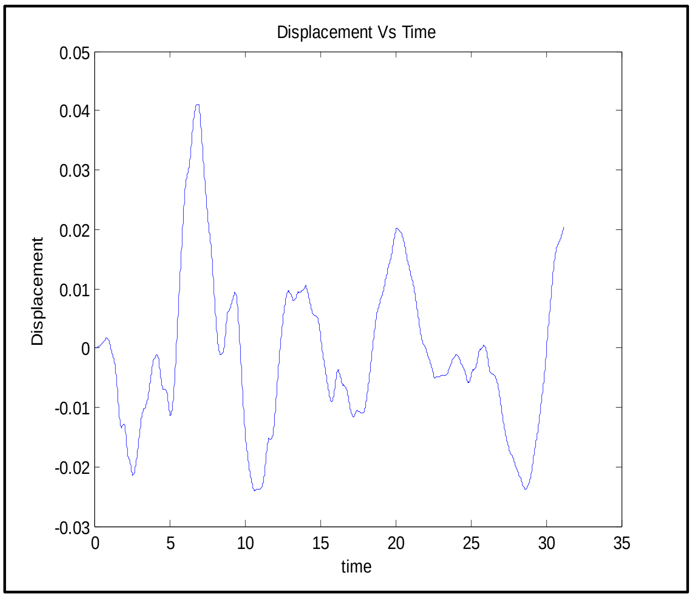

Figure 2b.Response of the one dimensional model to the 1940 El Centro earthquake

Figure 2 is showing the response of the model to random ground accelerations

To study the effect of tuned mass damperin structural damping when damping ratio of the structure is varied 1940 El Centro earth quakes having peak ground acceleration 0.313g was applied to the model. A study in such terms was carried by F Sadek et al(1997).The damping ratio of the tuned mass damper was kept at 2% while the damping ratio of the structure was varied from 2% to 5%.

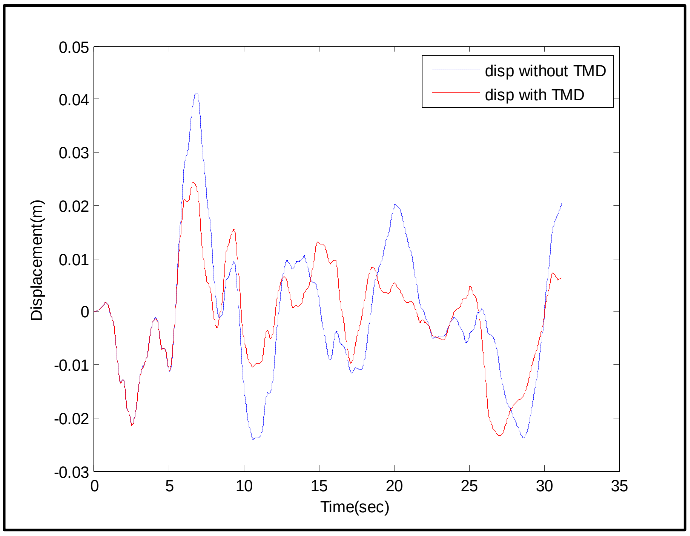

Figure 3a.Response of the structure when damping ratio of the structure is 2%

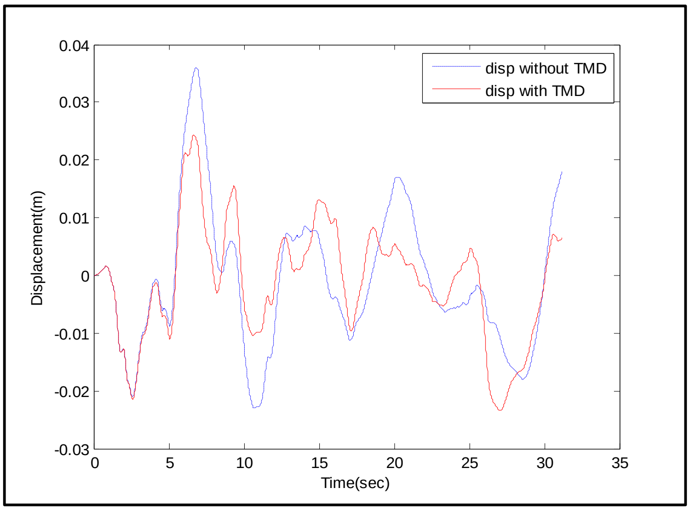

Figure 3b.Response of the structure when damping ratio of the structure is 5%

Figure 3 compares the displacement response at the top storey with variation of damping ratio of the structure when, El Centro (1940) earthquake loading is acting on the structure without and with tuned mass damper.

From figure 3 it can be concluded that tuned mass damper 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%).

2.2 *Effect of TUNED MASS DAMPER on structural damping with variation of mass ratio*

A study was carried out to see the effectiveness of tuned mass damperin controlling the response of the structure with variation of mass ratio and by keeping the damping ratio of tuned mass damperand building constant at 2% .Four mass ratios were considered and the building was subjected to two earthquake loads. One iscompatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil acting on the structure and the second one is El Centro (1940) earthquake.

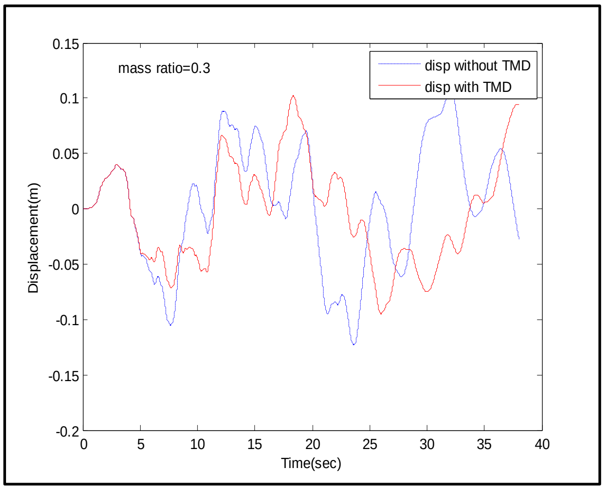

Figure 4a.Response of the structure when tuned mass damperis having mass ratio 0.3

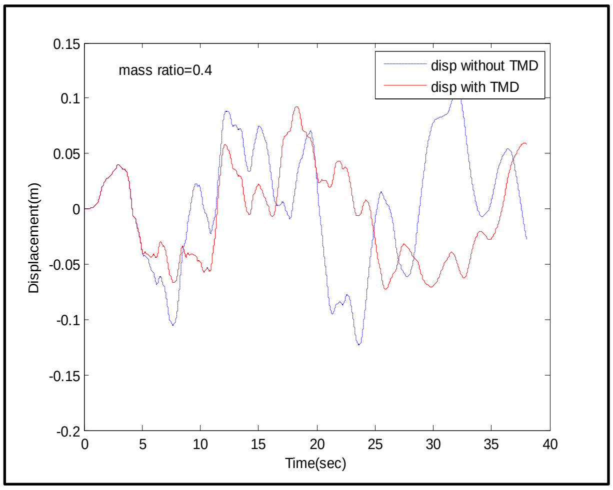

Figure 4b.Response of the structure when tuned mass damperis having mass ratio 0.4

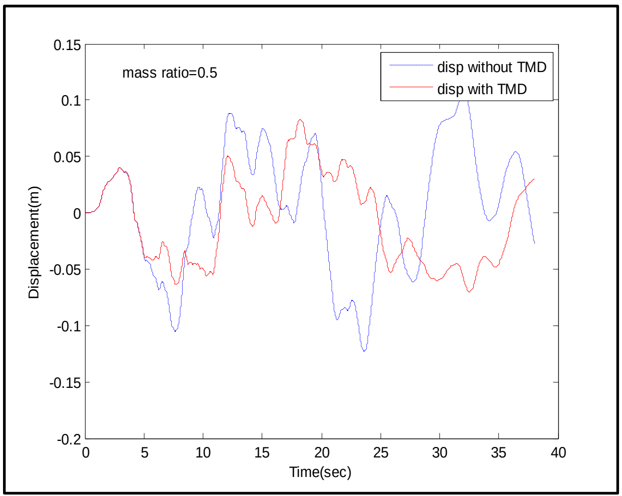

Figure 4c. Response of the structure when tuned mass damper is having mass ration 0.5

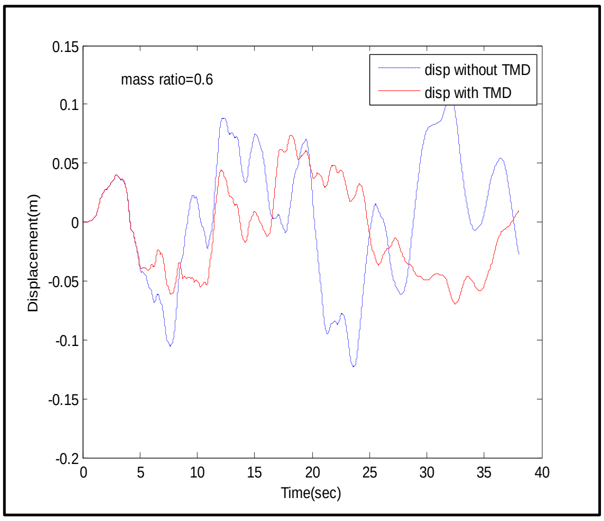

Figure 4d.Response of the structure when tuned mass damper is having mass ratio 0.6

Figure 4 compares the displacement response at the top storey with and without tuned mass damperwith variation of mass ratio of the tuned mass damper 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.

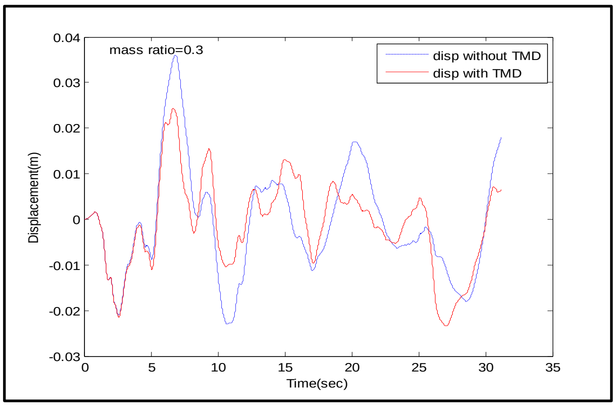

Figure 5a.Response of the structure when TUNED MASS DAMPER is having mass ratio 0.3

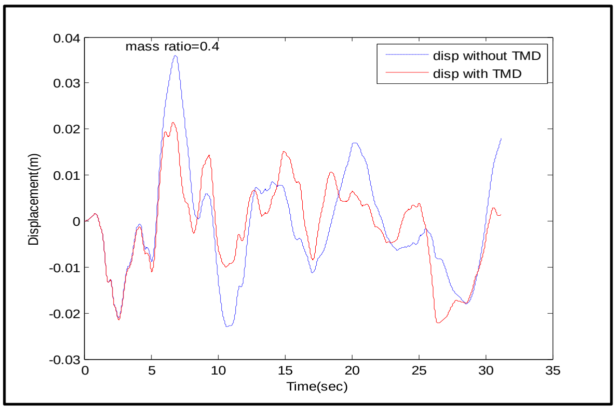

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Figure 5b.Response of the structure when tuned mass damper is having mass ratio 0.4

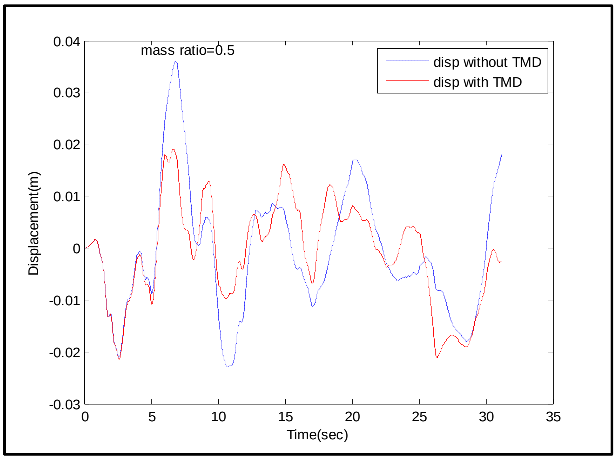

Figure 5c.Response of the structure when tuned mass damper is having mass ratio 0.5

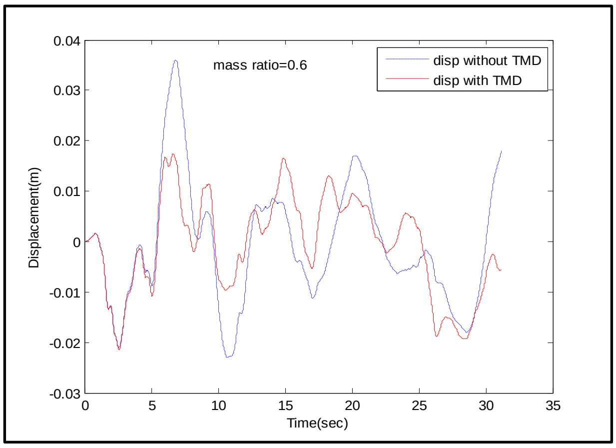

Figure 5d.Response of the structure when tuned mass damper is having mass ratio 0.6

Figure 5 compares the displacement response at the top storey with and without tuned mass damper,with variation of mass ratio of the tuned mass damper when, El Centro (1940) earthquake loading acting on the structure.

Figure 4 and 5 demonstrates that the performance of the tuned mass damper improves when mass ratio increases.

3 TWO DIMENSIONAL MULTI DEGREE OF FREEDOM FRAME MODEL

Considering a multistorey rigid jointed plane frame having storeys of height ‘H’ and bays of length ‘L’. The 2D frame model is discretized into a number of elements by considering 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. Figure 5 is a schematic diagram of the model.

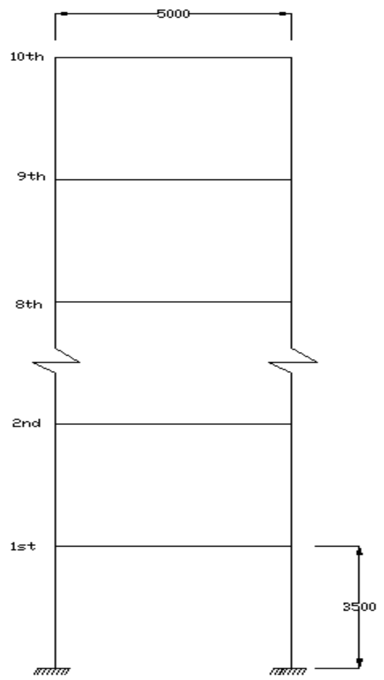

Figure 6.Elevation of 2D plane frame

3.1 *Free Vibration Analysis of the Multi Storey Frame*

3.1.1 *Convergent Study for Natural Frequency*

A convergent study has been carried out to find out the natural frequencies of the structural model. In total five equivalent models has been considered for this study by changing the number of elements. Number of elements has been increased by increasing the number of nodes in each beam and column between each floor.

Table 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				
	1	2	3	4	5
1st	13.424	13.424	13.424	13.424	13.424
2nd	43.513	43.511	43.510	43.510	43.510
3rd	81.725	81.709	81.706	81.704	81.704
4th	126.600	126.545	126.532	126.527	126.525
5th	167.534	167.015	166.924	166.899	166.889

It is observed from Table 1 that the fundamental natural frequencies of the structure are getting converged for any finer mesh division than 60 elements.

Therefore for further study, the structure model considered is 2-D frame model discretized to 60 elements, and the fundamental frequency of the structure is considered as 13.424 rad/sec.

3.1.2 *Variation of Natural frequencies with increase in number of storey*

Table 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 number of intermediate nodes per column=number intermediate nodes per beam = 5

Modes	Natural frequencies in(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	159.097	113.867	85.832
4th	592.899	257.833	207.109	171.342	132.918
5th	788.338	470.714	237.3117	196.733	175.321

From the table, it is clear that, the fundamental frequency of the structure decreases with increase in number of storey, keeping mass, stiffness of each storey unchanged.

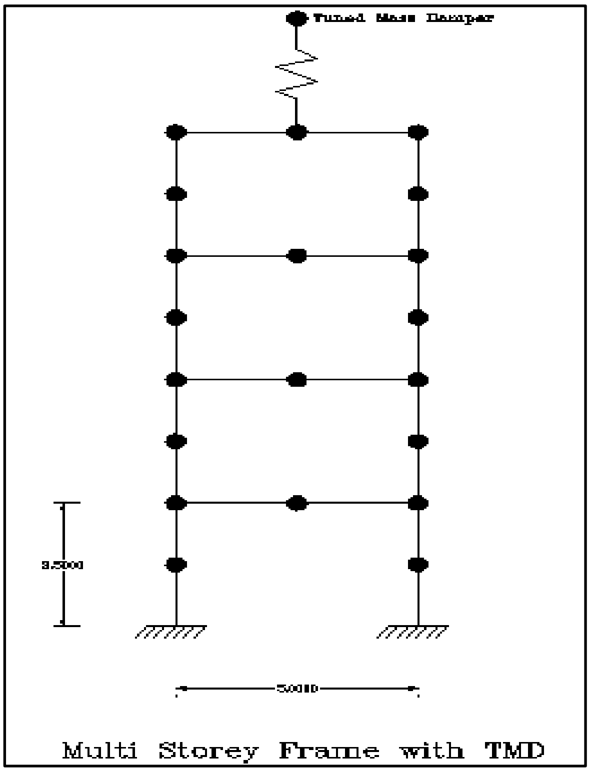

Figure 7.Damper Structure Arrangement for 2D frame

3.2 *Two Dimensional MDOF frame model with tuned mass damper*

The tuned mass damperis placed at the 10^{th} 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 10^{th} storey with tuned mass damper and without tuned mass damper. The tuned mass damper is having massratio=0.1 and tuning ratio=1.

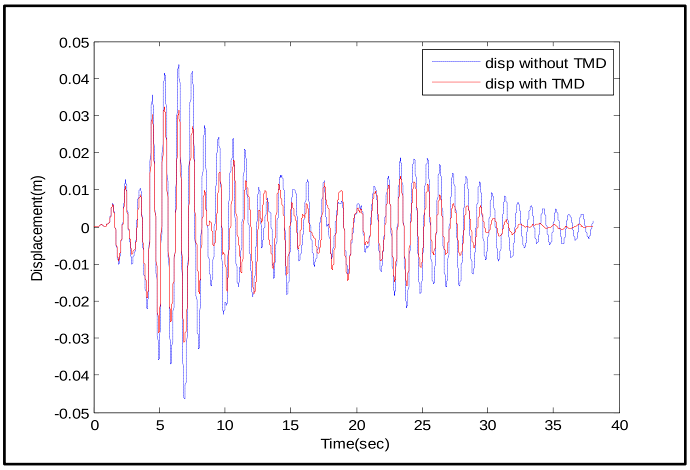

Figure 8.Responseof the structure when, El Centro(1940) earthquake loading acting on the structure.

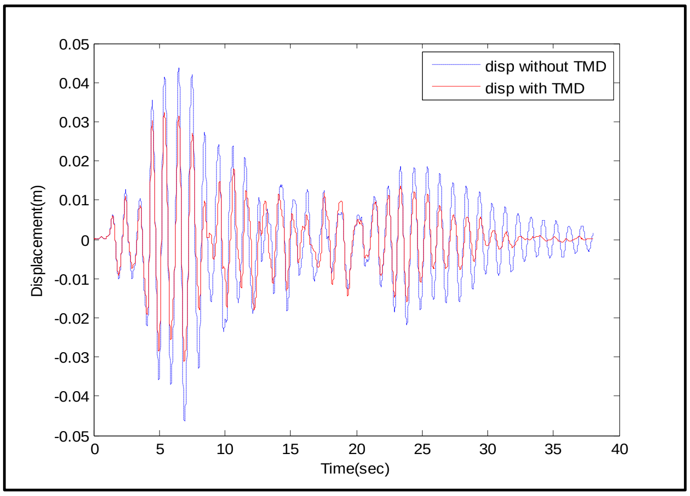

Figure 9.Response of the structure when, corresponding to compatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil earthquake loading acting on the structure.

Figure 8 and 9 compares the displacement response at the extreme right node of the 10th storey when subjected to both 1940 El Centro earthquakes and corresponding to compatible time history as per spectra of IS-1894(Part-1):2002 for 5% damping at rocky soil with tuned mass damper and without tuned mass damper. From the figures it can be concluded that the tuned mass damper is effective in reducing the displacement responses of the 2D multi-storey frame structure.

4 CONCLUSION

This study has been made to study the effectiveness of tuned mass damper for controlling vibration of structure. A numerical algorithm was developed to model the multi-storey multi-degree of freedom building frame structure as one dimensional single degree of freedom model with a tuned mass damper. Another numerical algorithm was also developed to analyze 2D-MDOF frame structure fitted with a tuned mass damper. A total of two loading conditions were applied at the base of the structure. The first one corresponding to compatible time history as per spectra of IS-1894(Part -1):2002 for 5%damping at rocky soil and the second one is 1940 El Centro Earthquake record (PGA = 0.313g).Following conclusions can be made from this study:

1. Tuned mass dampercan be successfully used to control vibration of the structure.
2. Tuned mass damperis 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 Tuned mass damperdecreases the displacement response of the structure.

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