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Control of Earthquake Induced Motions of Tall Buildings by Tuned Mass Damper

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ABSTRACT
Innumerable high rise building has been constructed all over the world and the number is increasing day by day. This is not only due to concerned over high density of population in the cities, commercial zones and space saving but also to establish land marks. As the seismic load acting on a structure is a function of the self-weight of the structure these structures are made comparatively light and flexible which have relatively low natural damping, and thus the structures become more vibration prone under wind and earthquake loading. To ensure the functional performance of tall buildings, various design modifications are possible, ranging from alternative structural systems to the utilization of passive and active control devices. This paper presents an overview of state-of-the-art measures to reduce structural response of tall buildings, including a discussion of auxiliary damping devices for mitigating the earthquake and wind induced motion of structures. To ensure the functional performance of tall buildings, various design modifications are possible, ranging from alternative structural systems to the utilization of passive and active control devices. Passive tuned mass damper (TMD) is widely used to control structural vibration under wind load but its effectiveness to reduce earthquake induced vibration is an emerging technique.
Here an analytical study is proposed to study the effectiveness of TMD to reduce structural vibration in Tall Buildings. For this study a 60m tall building having 15 storeys with a square plan of 20x20m has been modelled. The effectiveness of single TMD to reduce structural vibrations, is studied for a variation of TMD mass ratios

KEYWORDS: Structural Motions, Damping, Passive and Active control devices, TMD, AMD

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I. INTRODUCTION

Vibration may be caused by environmental forces acting on a structure, such as wind or earthquake, or by a seemingly innocuous vibration source causing resonance that may be destructive, unpleasant or simply inconvenient. The seismic waves will make buildings sway and oscillate in various ways depending on the frequency and direction of ground motion, and the height and construction of the building. Seismic activity can cause excessive oscillations of the building which may lead to even structural failure. The force of wind against tall buildings can cause the top of skyscrapers to move even more than a meter. This motion can be in the form of swaying or twisting. Certain angles of wind and aerodynamic properties of a building can accentuate the movement and cause motion sickness in occupants and pose serious serviceability issues. To enhance the functional performance of the building against seismic and wind forces, a proper building design is performed using alternative structural systems and by utilization of various vibration control devices.

1.1 Structural Systems

Beside the basic design of structural systems to efficiently carry lateral loads acting on the structure, certain features can be engineered into the structure to improve its performance under the action of wind and earthquake. The appropriate selection of an efficient structural system can provide the most effective means of controlling structural response to wind in the lateral and torsional directions. This may be accomplished through any number of systems including space frames, mega frame systems, tube systems, and the addition of vierendeel frames, belt trusses, super columns, vierendeel-type bandages and outrigger trusses. A structural system can also benefit from concrete or composite steel/concrete construction with higher internal damping. For example, the Petronas Towers in Kuala Lumpur utilized a concrete structural system which aided in improving the performance of the buildings from a serviceability standpoint.

1.2 TUNED MASS DAMPERS (Tmds)

Typically, a TMD consists of an inertial mass attached to the building location with maximum motion, generally near the top, through a spring and damping mechanism, typically viscous and viscoelastic dampers. 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. Energy is dissipated by the damper inertia force acting on the structure. The effectiveness of TMDs is determined by their dynamic characteristics, stroke and the amount of the added mass. Additional damping introduced by the system is also dependent on the ratio of the damper mass to the effective mass of the building in the mode of interest, typically resulting in TMDs, which weigh 0.25-1.0% of the building's weight in the fundamental mode (typically around one third). Often, spacing restrictions will not permit traditional TMD configurations, requiring the installation of alternative configurations including multi-stage pendulums, inverted pendulums, and systems with mechanically guided slide tables, hydrostatic bearings, and laminated rubber bearings. Coil springs or variable stiffness pneumatic springs typically provide the stiffness for the tuning of TMDs.

Although TMDs are often effective, even better responses have been noted through the use of multiple-damper configurations (MDCs) which consist of several dampers placed in parallel with distributed natural frequencies around the control tuning frequency (Kareem & Kline 1995). For the same total mass, a multiple mass damper can significantly increase the equivalent damping introduced to the system. There are several types of TMDs in use, typically oil dampers, viscous and viscoelastic dampers.

Table 1. Mass support mechanisms and dampers for TMDs

Mass Supporting Mechanism	**Damper Attached to TMD**
Pendulum, including multiple type	Oil Dampers
Roller Bearings & Coil Springs	Viscous Dampers
Laminated Rubber Bearings	Visco-Elastic Dampers

Figure 1. Tuned Mass Damper in Taipei 101.

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1.3 Real Life Structures Equipped With Tmds

Tuned mass dampers have been used to improve the response of building structures under wind and seismic excitation. A short description of the several building structures that are equipped with Tuned Mass Dampers (TMDs) follows.

I. John Hancock Tower, Boston

One of the earliest applications of this type was installed in June 1977 in the 244 m (60 storey) John Hancock Tower in Boston. Two TMDs were installed at opposite ends of the 58th floor, at a spacing of 67 m, in order to counteract sway as well as the torsional motion due to the shape of the building. Each damper measured about 5.2x5.2x1 m and was essentially a steel box filled with lead, weighing 300 tons, attached to the frame of the building by stiff springs. The lead-filled weight slides back and forth on a hydrostatic bearing consisting of a thin layer of oil forced through holes in the steel plate. Whenever the horizontal acceleration exceeds 0.003g for two consecutive cycles, the system is automatically activated. This system is expected to reduce the sway of the building by 40 to 50%.

II. **Citicrop Centre, New York**

Another pioneering application of TMDs has been in the Citicorp Building in New York. The height of the building is 278 m with fundamental period of around 6.5 s and damping ratio of 1% along both axes. The system, measuring 9.14 x 9.14 x 3.05 m, consists of a 410-ton concrete block supported on a series of twelve 60-cm diameter hydraulic pressure-balanced bearings with two spring damping mechanisms, one for the north-south motion and one for the east-west motion, was installed in the 63rd floor in 1978. The system reduces the wind induced response of the Citicorp building by 40% in both the north-south and east-west directions, simultaneously (Wiesner 1979). The damper system is activated automatically whenever the horizontal acceleration exceeds 0.003g for two consecutive cycles and will automatically shut itself down when the building acceleration does not exceed 0.00075g in either axis over a 30-minute interval.

Iv. Chiba Port Tower, Japan

Chiba Port Tower, a steel structure of 125 m in height and having a rhombus-shaped plan with a side length of 15 m (completed in 1986) was the first tower in Japan to be equipped with a TMD. The time period in the first and second mode of vibrations are 2.25 s and 0.51 s, respectively for the x direction and 2.7 s and 0.57 s for the y direction respectively. Damping for the fundamental mode was computed at 0.5%. For higher mode of vibration damping ratios proportional to frequencies were assumed in the analysis. The use of the TMD was to increase damping of the first mode for both the x and y directions. The mass ratio of the damper with respect to the modal mass of the first mode was 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%. Reductions of around 30 to 40% in the displacement of the top floor and 30% in the peak bending moments are expected.

V. Taipei 101, Taiwan

Taipei 101, a steel braced building is the 3rd tallest building in the world. A sphere shaped TMD of weight 660 ton and diameter 5.5 m has been installed between 88^{th} to 92^{nd} floor of the building as shown in Figure 1.5. This is an example of a pendulum type Tuned Mass Damper. The enormous sphere was suspended by four set of cables, and the dynamic energy is dissipated by eight hydraulic pistons each having length of 2 m. The damper can reduce 40% of the tower movement. Another two tuned mass dampers, each weighing 6 metric tons sit at the tip of the spire. These prevent damage to the structure due to strong wind loads.

Vi. Burj Al Arab, Dubai

In the world's tallest hotel Burj Al Arab is equipped with 11 TMDs have been installed at different locations to control the wind induced vibration.

Vii. Atc Tower in New Delhi, India

A 50-ton Tuned Mass damper has been installed just beneath the ATC floor at 90m level.

Viii. Statue of Unity, India

In the world's tallest statue, the Statue of Unity (182m high) two 200-ton Tuned mass dampers has been installed at the shoulder level.

1.4 EQUATIONS OF MOTION

In this section, the concept of the tuned mass damper is illustrated using
the two mass system shown in Figure 5. Here, the subscript d refers to the tuned mass damper; the structure is idealized as a single degree of freedom system. Hence the following notation can be defined as,

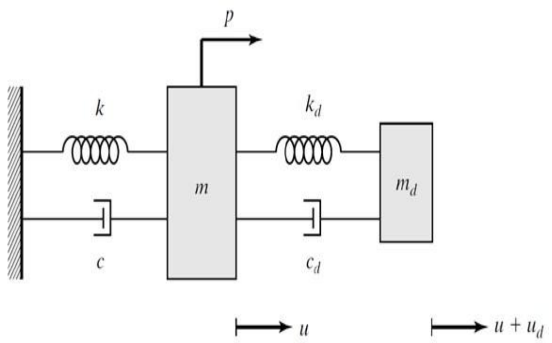


Figure 2 SDOF-TMD System.

and the equation of motion for the tuned mass is given by

$$u\ddot{u}_{dd} + 2\xi\xi_{dd}\omega\omega_{dd}u\dot{u}_{dd} + \omega\omega_{dd}{}^{2}uu_{dd} = -u\ddot{u} \quad (1)$$

The purpose of adding the mass damper is to limit the motion of the structure when it is subjected to a particular excitation. The design of the mass damper involves specifying the mass mm_{dd}, stiffness kk_{dd}, and damping coefficient cc_{dd}. The damper is tuned to the fundamental frequency of the structure such that

$$\omega\omega_{dd} = \omega\omega \quad (2)$$

The stiffness's for this frequency combination are related by

$$kk_{dd} = mmkk \quad (3)$$

Considering the primary mass is subjected to the following periodic sinusoidal excitation,

$$pp = p\hat{p} \sin \Omega\Omega\Omega\Omega \quad (4)$$

then the response is given by

$$uu = uu \sin(\Omega t + \delta_1) \quad (5)$$

$$uu_{dd} = uu_{dd} \sin(\Omega t + \delta_1 + \delta_2) \quad (6)$$

where uu and δ denote the displacement amplitude and phase shift, respectively. The critical loading scenario is the resonant condition, $\Omega = \omega$.

1.5 DESIGN OF A TUNED MASS DAMPER

The design of a damped TMD for an un-damped structure involves the
following steps:

- Establish the allowable values of displacement of the primary mass and the TMD for
the design loading.
- Determine the mass ratios required to satisfy these motion constraints from Figure 3.8 and Figure 3.9. Select the largest value of mm.
- Determine $ffooppoo$:
- Compute $\omega\omega dd$:
- Compute $kkdd$.
- Compute $ccdd$
- Determine Pendulum Length (L):

1.6 Model Considered For Analysis

The models considered for analysis are sample models and case study models. It has been attempted to select models which are both representative and indicative of the actual behavior of the real life structures. The building is 60 m high and has a plan dimension of 20x20m. The structural system of the building is a "Reinforced concrete SMRF" system. Lateral load resisting elements are square RCC columns of size 500x500 mm spaced at 5m in both X and Y axes. The floor is a 125 mm thick RCC slab supported by RCC beams of size 300x500 mm. The grade of concrete for slabs, beams and columns is M30.

Four numbers of 3-D FEM model of the 15 storey building has been made using the structural analysis and design software ETABS. The slabs have been modelled as a Membrane element just to transfer the floor loads to the beams. The beams and columns have been modeled as Frame elements. All the models are identical and have same loading and member properties. The first model is being used as a base model for calculating the fundamental natural period of the building and for comparison of the results. The Pendulum type TMDs have been added to the rest of three model, with mass ratios of 0.01, 0.02 and 0.04 as listed in Table 4.2. The TMDs has been modelled at the centre of the building at the roof level, with a linear spring attached to the structure at one end and at the free end the mass is assigned. The different parameters of the TMDs have been listed in Table 3.

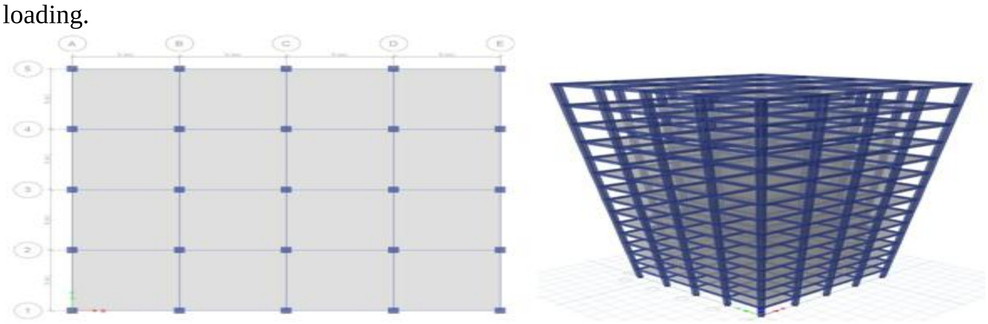


Figure 3 Base Model.

Table 2 Model ID

Model ID	**Description**
Model-1	Base model without TMD
Model-2	Model with TMD having mass ratio mm = 0.01
Model-3	Model with TMD having mass ratio mm = 0.02
Model-4	Model with TMD having mass ratio mm = 0.04

1.7 Calculation of TMD Parameters

Based on the modal analysis results of the base model, different parameters of the TMDs have been calculated using the guidelines mentioned earlier in Section 3.2.3. The calculated parameters of the TMDs with mass ratios 0.01, 0.02 and 0.04 are listed below in Table 3.

Table 3 Tuned Mass Damper parameters

Description		**TMD Parameters for mass ratios**			**Units**
	Symbol	**1%**	**2%**	**4%**	
Fundamental period of the building	TT	3.552	3.552	3.552	sec
Angular frequency of building	$\omega\omega$	1.769	1.769	1.769	rad/s
Seismic weight of the building	WW	87025	87025	87025	kN
Modal participating mass ratio		0.75	0.75	0.75	
Participating mass in 1st mode	m	6656	6656	6656	Ton
Adopted mass ratio for TMD	mm	0.01	0.02	0.04	
Mass of the TMD	mm_{dd}	67	133	266	Ton
Weight of the TMD	WW_{dd}	657	1304	2609	kN
Optimal tuning frequency ratio	ff_{ooppoo}	0.988	0.975	0.952	
Optimal angular frequency of TMD	$\omega\omega_{dd}$	1.747	1.726	1.684	rad/s
Pendulum length required	L	3.213	3.294	3.459	m
Link stiffness in U1 direction	$kkdd1$	45440	87990	167585	kN/m
Link stiffness in U2 direction	$kkdd2$	204	396	754	kN/m
Link stiffness in U3 direction	$kkdd3$	204	396	754	kN/m
Optimal damping ratio for TMD	$\xi\xi ddooppoo$	0.060	0.085	0.118	
Linear damping coefficient	cc_{dd}	14.13	39.87	111.44	kN-s/m

- A linear 2-noded link is added to the structure at the roof level with start node attached to the roof framing and the end node hanging freely.
- Now the TMD mass mm_{dd} is assigned as special mass to the end node of link and the TMD weight is assigned to the same node as a gravity load in the Self weight load case. The Self-weight load case shall contain only the self-weight of the structure and the weight of the TMD.
- The link stiffness's $kkdd_1$, kk_{dd2} and kk_{dd3} are specified in the link property. The linear damping coefficient cc_{dd} is also specified in the link property.

- "Element Self Mass and Additional Mass" must be included in the Mass Source. Mass from "Specified Load Patterns" shall also be included but the Self-weight load pattern (which contains only the self-weight of the structure and the weight of the TMD) shall not be included as it has already been included through the "Element Self Mass and Additional Mass".

Table 4 Comparison of Response Spectrum Modal Information

Model	mm	Period		Effective Damping		U1/U2Acc	
		sec	change	%	change	m/sec^2	change
Model-1	0.00	3.552	-	5.00%	-	0.215	-
Model-2	0.01	3.814	7.38%	8.01%	60.2%	0.177	17.67%
Model-3	0.02	3.939	10.90%	9.27%	85.4%	0.164	23.72%
Model-4	0.04	4.143	16.64%	11.09%	121.8%	0.153	28.83%

1.7 Results and Discussions.

Comparison of Base Reactions

The base reactions from the models Model-1, Model-2, Model-3 and Model-4 are listed below in Tables 4.

Table 4 Comparison of Base Reactions

. Output Case	Model-1		Model-2 $mm = 00.0000$		Model-3 $mm = 00.0000$		Model-4 $mm = 00.0000$	
	FX	FY	FX	FY	FX	FY	FX	FY
	kN	kN	kN	kN	kN	kN	kN	kN
EQX	-1750	0	-1764	0	-1777	0	-1803	0
EQY	0	-1750	0	-1764	0	-1777	0	-1803
WLX	-2030	0	-2030	0	-2030	0	-2030	0
WLY	0	-2030	0	-2030	0	-2030	0	-2030
SPECX	1750	0	1504	367	1438	348	1388	305
SPECY	0	1750	367	1504	348	1438	305	1388
TH_Bhuj	1179	1521	1399	1513	1339	1450	1230	1327
TH_Bhuj	-1751	-1749	-1730	-1782	-1656	-1757	-1463	-1679

From the above results it can be seen that with the addition of the TMD, the Base Shears of the building have changed except for the Wind load case. Since the effect of the TMD is supposed to be captured in the Modal analysis cases only, there should not be any change in the Base Shear for EQX, EQY, WLX and WLY as these are Static load cases. But the increase in Base Shear for EQX and EQY with the increasing mass ratio of the TMD is because of the static weight of the TMD and the increment is AA_h times the weight of TMD, which is usual.

For the Response Spectrum load cases SPECX and SPECY, the Base Shear has reduced with the increasing mass ratio, compared to the base model. The reduction in Base Shear is 14.06%, 17.82% and 20.69% for the TMD mass ratios of 0.01, 0.02 and 0.04 respectively.

For the Time History load case the change in the Base shear is not substantial for the TMD mass ratios of 0.01 and 0.02, but the Base Shear has decreased by 16.45% in X direction and 4.0% in Y direction for the mass ratio 0.04.

Comparison of Storey Displacements

The storey displacements from the models Model-1, Model-2, Model-3 and Model-4 are shown below in Figures 4.18 and 4.19. The displacement responses of a joint at roof level for Time History load case are also shown in Figures 4.20, 4.21 and 4.22. From these results it can be seen that with the addition of the TMD, the storey displacements of the building have reduced significantly.

For the Response Spectrum load cases SPECX and SPECY, the reduction in storey displacements is 19.8%, 25.1% and 29.1% for the TMD mass ratios of 0.01, 0.02 and 0.04 respectively.

For the Time History load case the reduction in the storey displacements is 10.5%, 15.5% and 23.4% for the TMD mass ratios of 0.01, 0.02 and 0.04 respectively

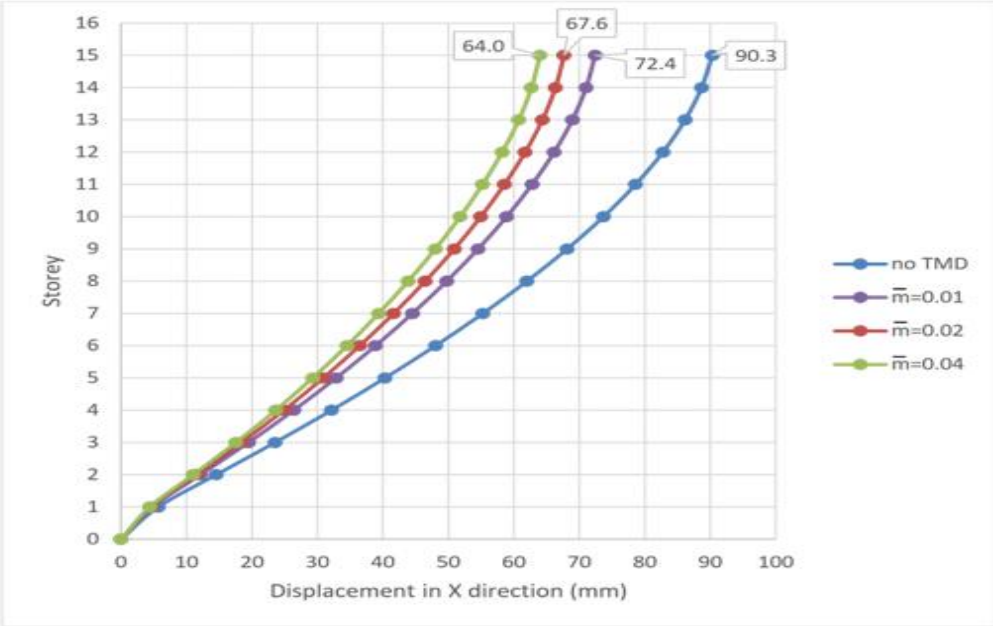


Figure 4 Storey Displacements for EQX and EQY

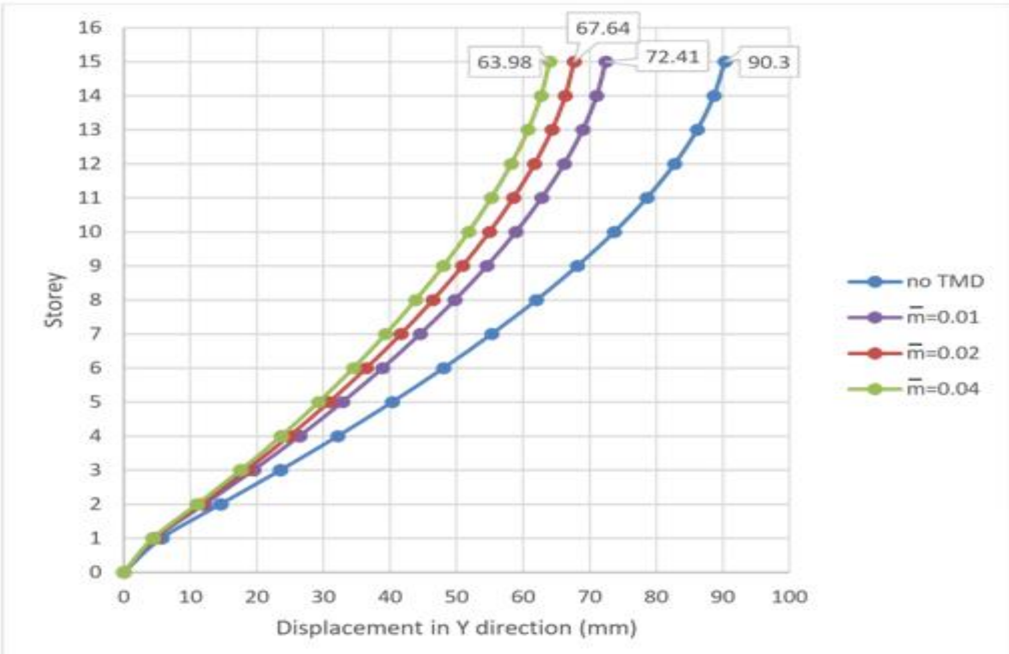


Figure 5 Storey Displacements for SPECX and SPECY

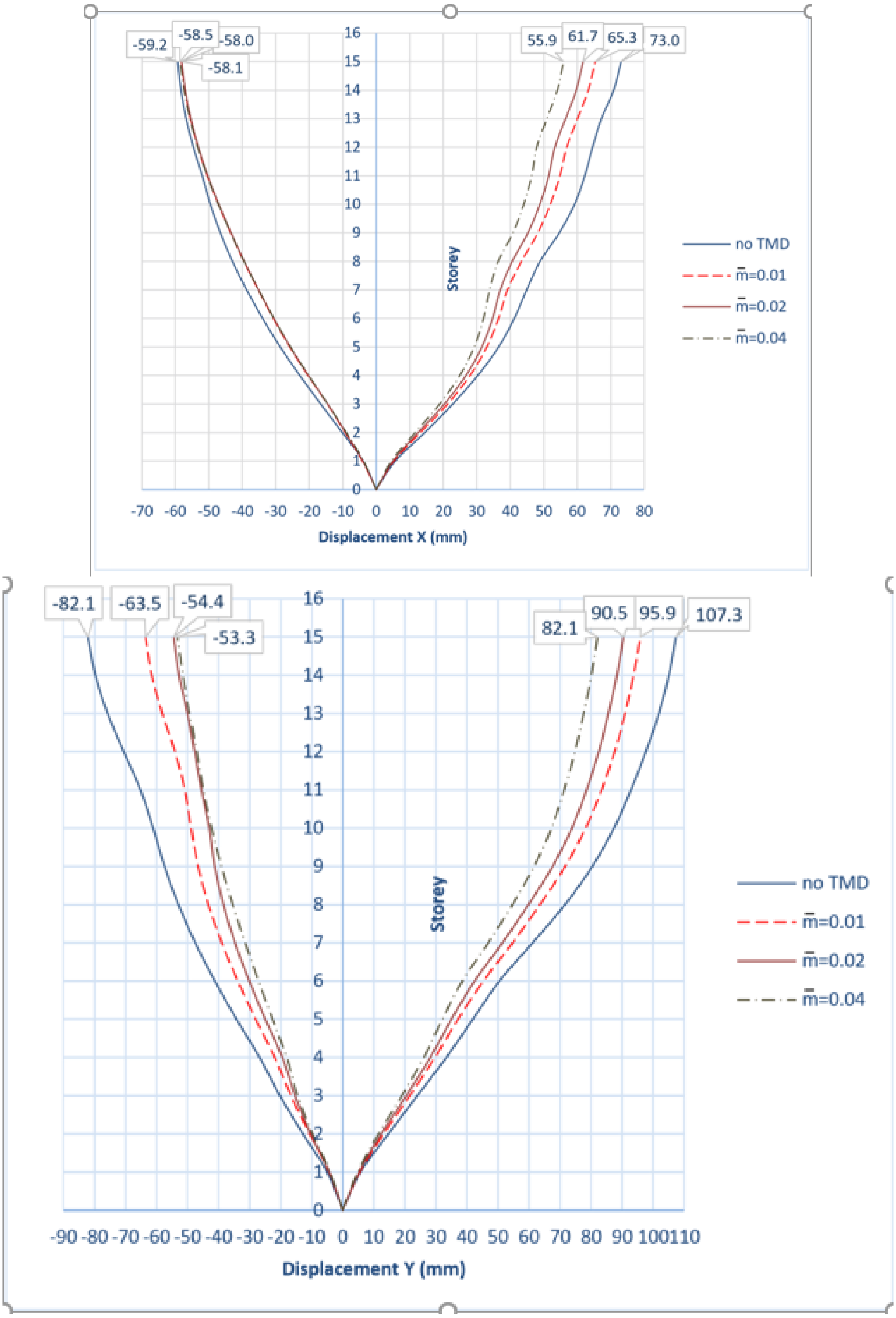


Figure 6 Storey Displacements for TH_Bhuj.

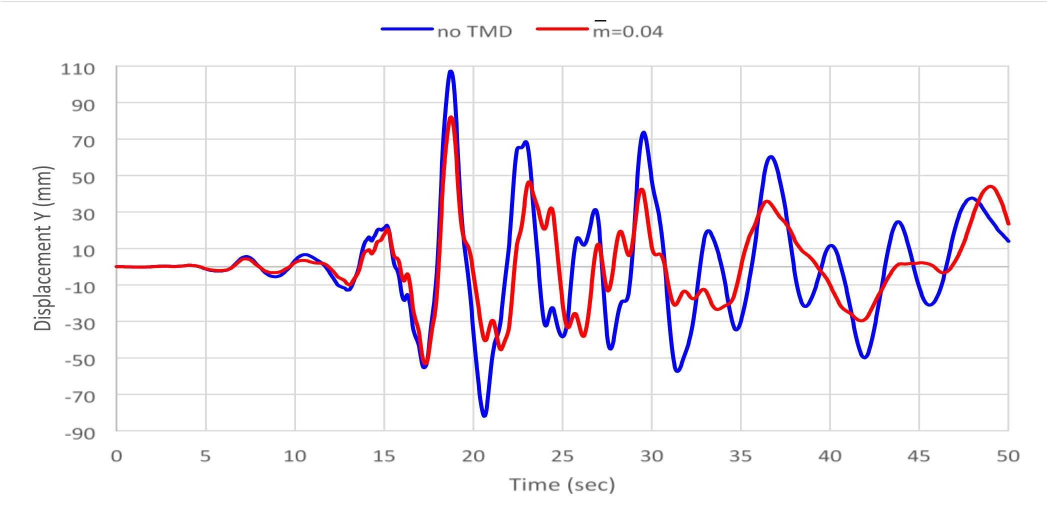


Figure 7 Joint Displacement at Roof Level for TH_Bhuj.

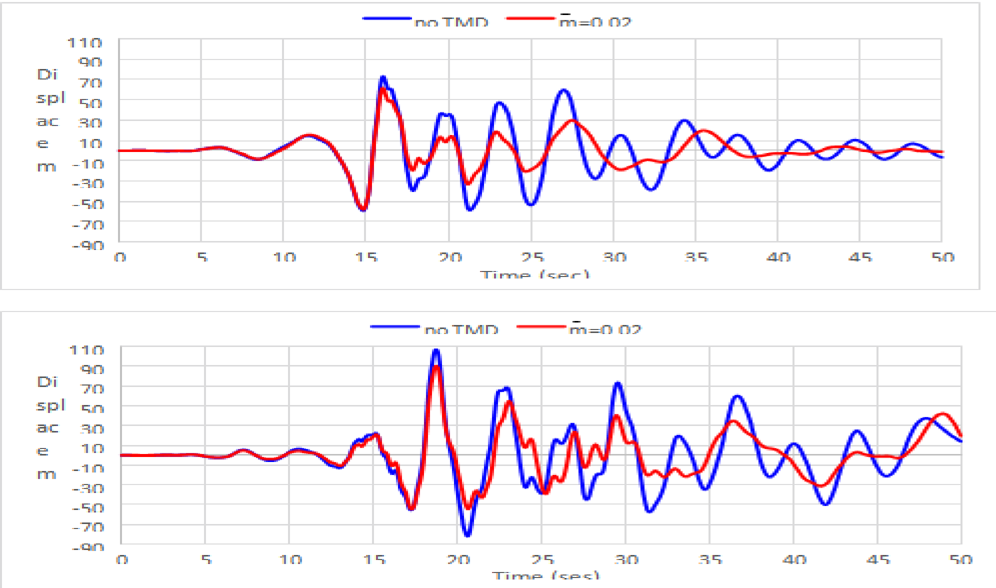
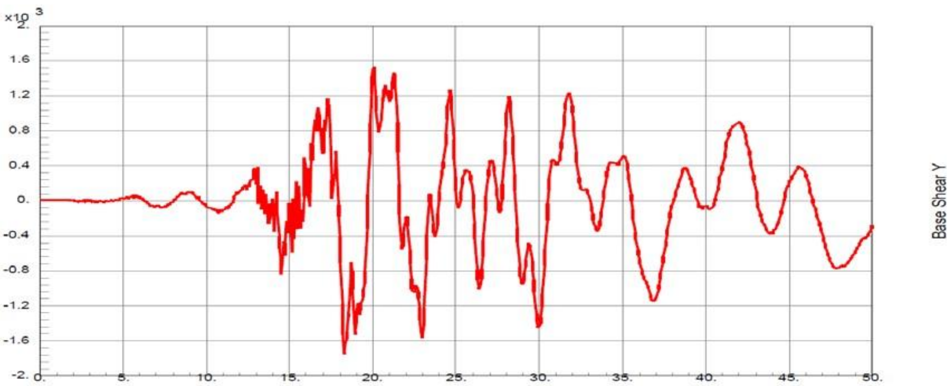


Figure 8 Joint Displacement at Roof Level for TH_Bhuj



VB_Ymin = -1749 kN at 18.25 sec, VB_Ymax = 1521 kN at 20.05 sec.

Figure 8 Base Shear for Time History load case

II. CONCLUSION

In the present study the 3D model considered is 60 m tall building having 15 storeys with floor to floor height of 4 m. The building has a square plan of 20x20 m. In this study both the building and the damper has been modelled as linear. Four numbers of identical models were created, first model is the base model (uncontrolled) and the remaining three models (controlled) have TMDs with mass ratios of 0.01, 0.02 and 0.04.

Linear time history analysis of the building has been done using the
acceleration data of Bhuj/Kutch 2001 earthquake. Present study focused on the ability of TMD to reduce earthquake induced structural vibration and to compare the building response with effect of variation in mass ratio and damping ratio of TMD. From this study it can be concluded that.

1) The acceleration of the building in the fundamental mode is reduced by 17.67%, 23.72% and 28.83% for the mass ratios of 0.01, 0.02 and 0.04 respectively.
2) The effective damping of the building in the fundamental mode is increased to 8.01%, 9.27% and 11.09% for the mass ratios of 0.01, 0.02 and 0.04 respectively.
3) The maximum storey displacement of the building is reduced by 19.8%, 25.1% and 29.1% for the mass ratios of 0.01, 0.02 and 0.04 respectively.
4) The effective damping of the building increases and the dynamic response of the building reduces as the mass ratio of the TMD is increased. The TMD becomes robust with increasing mass ratio. Hence an optimal mass ratio of the TMD can be found to reduce the building responses substantially there by giving a desired level of human comfort, safety and economy to the structure.

Compliance with ethical standards

Conflict of interest On behalf of all authors, the corresponding author states that there is no conflict of interest.

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