

DESIGN OF OPTIMUM PARAMETERS OF TUNED MASS DAMPER FOR A G+8 STORY RESIDENTIAL BUILDING

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ABSTRACT- In recent days, the numbers of taller and lighter structures are endlessly increasing within the construction industries that are versatile and having a very low damping value. Those structures will simply fail under structural vibrations induced by earthquake and wind. Therefore many techniques are available nowadays to minimize the vibration of the structure, out of that idea of using TMD could be oldest one.

This thesis summarizes the results of a parametric study performed to enhance the understanding of some important characteristics of tuned mass dampers (TMD). To identify the behavior of frame elements in the structure, Time history analysis is performed using ETABS 2015 ULTIMATE 15.2.2 software. The compatibility between the designs of a TMD for initial three modes of a MDOF structure is drawn to simplify TMD design to control a single mode of a multimodal structure. An example is given to illustrate the design procedure. Comparative study is also to be done for different mass ratio for particular mode. Keep the stiffness and damping value constant the TMD is tuned to the structural frequency of the structure. Various parameters such as mass ratio, frequency ratio, damping ratio etc. are considered to observe the effectiveness and robustness of the TMD in terms of percentage reduction in peak response of the structure.

Key Words: Tuned Mass Damper (TMD), Pendulum TMD, Residential Building, Mass ratio, Damping ratio, Frequency Ratio, ETABS, Time History Analysis (THA), Load Combination.

1. INTRODUCTION

A tuned mass damper (TMD) is a passive control device consisting of a mass, a spring, and a damper that is attached to a structure in order to reduce the dynamic response of the structure [1,7]. Energy is dissipated by the damper inertia force acting on the structure. It has been widely used for vibration control in many mechanical engineering systems. Recently many theories have been adopted to reduce vibration in civil engineering structures because of its easy and simple mechanism. To obtain optimum response the natural frequency of the secondary mass damper is always tuned to that of primary structure such that when that particular frequency of the structure get excited, the TMD will resonate out of phase with the structural motion. The excess amount of energy built up in the structure is transformed to the secondary mass and dissipated due to relative motion developed between them at a later stage.

1.1 PENDULUM TUNED MASS DAMPER:

The problems associated with the bearings can be eliminated by supporting the mass with cables which allow the system to behave as a pendulum [1,2]. Figure 1 (a) shows a simple pendulum attached to a floor. Movement of the floor excites the pendulum. The relative motion of the pendulum produces a horizontal force that opposes the floor motion. This action can be represented by an equivalent SDOF system that is attached to the floor, as indicated in Figure 1(b).

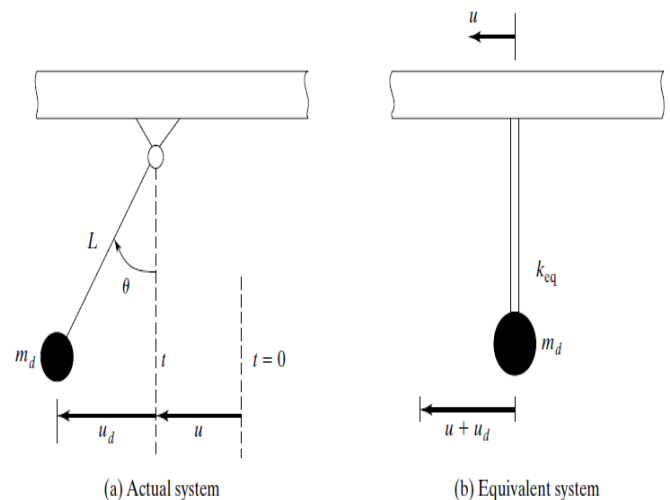


Fig. 1: A simple pendulum tuned mass damper [1,2]

natural period of the pendulum is given by

$$T_d = 2\pi \sqrt{\frac{L}{g}}$$

Above equations are taken from book "introduction to structural motion control" by Jerome J. Cannon [1].

2. METHOD OF ANALYSIS

There are different methods of analysis which provide different degrees of accuracy [3]. The analysis process can be categorized on the basis of three factors: the type of the externally applied loads, the behaviour of structure or structural materials, and type of structural model selected. Based on the type of external action and behaviour of structure, the analysis can be further classified as given below [3]:

(1) Linear static analysis

1.1 Equivalent static method

(2) Linear dynamic analysis

2.1 Response spectrum method

2.2 Elastic time history method

(3) Nonlinear static analysis

3.1 Push over analysis

(4) Nonlinear dynamic analysis

4.1 Inelastic time history method

Linear static analysis can only be used for regular structure with limited height [3]. Linear dynamic analysis can be performed in two ways either by response spectrum method and elastic time history method. This analysis will produce the higher modes of vibration and actual distribution of forces in the elastic range in a better way. Nonlinear static analysis is an improvement over the linear static or dynamic analysis in the sense that it allows the inelastic behaviour of the structure. Inelastic time history analysis is the only method to describe the actual behaviour of the structure during an earthquake. Among all these methods we use Time history method for analysis.

2.1 TIME HISTORY ANALYSIS

The time history analysis technique represents the most sophisticated method of dynamic analysis for buildings [4]. In this method, the mathematical model of building is subjected to accelerations from earthquake records that represent the expected earthquake at the base of the structure. This method consists of a step-by-step direct integration over a time interval the equation of motion are solved with the displacements, velocities and accelerations of the previous step serving as initial function.

3. DESIGN OF TUNED MASS DAMPER:

3.1 PROBLEM STATEMENT:

G+8 storied buildings are modeled using concrete beams, columns, slabs, infill wall and stairs. These buildings were given H shape geometry with plan dimensions of 21.2 m × 28.4 m. They are loaded with Dead, Live and Seismic Forces [according to IS: 1893(Part I):2002]. These models are then analyzed using time history method for earthquake zone V of India (Zone Factor = 0.36). The details of the modeled building are listed below. Modal damping of 5% is considered with SMRF (Response Reduction Factor, R=5) and Importance Factor (I) =1. The performance of the models is recorded through ETABS to present a brief idea about the role of tuned mass damper in protecting the structure against earthquake hazards.

3.2 Design steps for tuned mass damper:

Steps involved in design of TMD are follows:

STEP 1: Lumped mass calculation of various floor levels

At roof level:

$$\text{Weight of slab} = 4(25 \times 8.6 \times 14.2 \times 0.15) + 25 \times 4 \times 3 \times 0.15 = 1876.8 \text{ kN}$$

$$\text{Weight of beam} = 25 \times 0.3 \times 0.5 [4(4 \times 8.4 + 14.1 \times 2 + 6.8) + 4.2 \times 2 + 3.2] = 1072.5 \text{ kN}$$

$$\text{Weight of column} = 50(25 \times 0.3 \times 0.6 \times \frac{3.5}{2}) = 393.75 \text{ kN}$$

$$\text{Weight of wall} = 4[18 \times 0.2(3 \times 8.4 + 2 \times 12.5 + 1.6 + 6.8 + 2 \times 4.2) \times \frac{3.5}{2}] = 1688.4 \text{ kN}$$

$$\text{Weight of staircase} = 25(0.5 \times 0.3 \times 0.14 \times 1.5 \times 24 + 1 \times 3 \times 0.14 + 7 \times 1.5 \times 0.15) \times 0.5 = 34.38 \text{ kN}$$

Weight of structure at roof level (W_g)

$$\begin{aligned} &= \text{weight of slab} + \text{weight of beam} + \text{weight of column} + \text{weight of wall} + \text{weight of staircase} \\ &= 1876.8 + 1072.25 + 393.75 + 1688.4 + 34.38 = 5065.58 \text{ kN} \end{aligned}$$

Mass of structure at floor level (M_g)

$$M_g = \frac{W_g}{9.81} = 516.36 \text{ tonne}$$

For other stories

$$M_1 = M_2 = M_3 = M_4 = M_5 = M_6 = M_7 = M_8$$

$$\text{Weight of slab} = 4(25 \times 8.6 \times 14.2 \times 0.15) + 25 \times 4 \times 3 \times 0.15 = 1876.8 \text{ kN}$$

$$\text{Weight of beam} = 25 \times 0.3 \times 0.5 [4(4 \times 8.4 + 14.1 \times 2 + 6.8) + 4.2 \times 2 + 3.2] = 1072.5 \text{ kN}$$

$$\text{Weight of column} = 50(25 \times 0.3 \times 0.6 \times 3.5) = 787.5 \text{ kN}$$

$$\text{Weight of wall} = 4[18 \times 0.2(3 \times 8.4 + 2 \times 12.5 + 1.6 + 6.8 + 2 \times 4.2) \times 3.5] = 3376.8 \text{ kN}$$

$$\begin{aligned} \text{Weight of staircase} &= 25(0.5 \times 0.3 \times 0.14 \times 1.5 \times 24 + 1 \times 3 \times 0.14 + 7 \times 1.5 \times 0.15) \\ &= 68.76 \text{ kN} \end{aligned}$$

$$\text{Weight of floor finish} = (8.6 \times 14.2 \times 4 + 4 \times 3) = 500.48 \text{ kN}$$

$$\text{Live load} = 3(8.6 \times 14.2 \times 4 + 4 \times 3) \times 0.25 = 375.36 \text{ kN}$$

Weight of structure for other stories (W_1)

= weight of slab + weight of beam + weight of column + weight of wall + weight of staircase

$$= 1876.8 + 1072.25 + 787.5 + 3376.8 + 68.76 + 500.48 + 375.36 = 8058.2 \text{ kN}$$

Mass of structure for other stories (M_1)

$$M_1 = \frac{W_1}{9.81} = 821.42 \text{ tonne}$$

Note^[8]: The earthquake forces shall be calculated for the full dead load plus the percentage of imposed load as given in Table 8 of IS 1893 (Part 1): 2002. The imposed load on Roof is assumed to be zero. 25 % of imposed load, if imposed load is upto 3 kN/m^2 .

STEP 2: Determination of fundamental natural period

The approximate fundamental natural period of a vibration (T_a) in seconds, of all other buildings, including moment resisting frame building with brick infill panels, may be estimated by the empirical expression^[8]:

$$T_a = 0.09 \times \frac{h}{\sqrt{d}}$$

Where h = height of building in meter and

d = base dimension of the building at the plinth/ground level in meter along the considered direction of the lateral force. so

$$T_{ax} = 0.09 \times \frac{31.5}{\sqrt{21.2}} = 0.616 \text{ second and}$$

$$T_{ay} = 0.09 \times \frac{31.5}{\sqrt{28.4}} = 0.532 \text{ second.}$$

STEP 3: Total Lateral stiffness of each story ^[3]

$$k_1 = k_2 = k_3 = k_4 = k_5 = k_6 = k_7 = k_8 = k_9$$

$$= 30 \times \left(\frac{12E}{L^3}\right) I_1 + 20 \times \left(\frac{12E}{L^3}\right) I_2 = k$$

$$E = 5000 \sqrt{f_{ck}} = 5000 \sqrt{25} = 25000 \text{ mpa}$$

$$I_1 = \frac{0.3 \times 0.6^3}{12} = 0.0054 \text{ m}^4 \text{ and } I_2 = \frac{0.6 \times 0.3^3}{12} = 0.00135 \text{ m}^4 \text{ and } L = 3.5 \text{ m, So value of } k = 1.32 \times 10^6 \text{ kN/m}$$

STEP 4: Calculation of Eigen values and Eigen vectors ^[3]

Mass matrix (M)

$$\begin{bmatrix} 821.42 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 821.42 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 821.42 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 821.42 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 821.42 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 821.42 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 821.42 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 821.42 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 516.36 \end{bmatrix}$$

Stiffness matrix(K)

$$k_1 + k_2 = k_2 + k_3 = k_3 + k_4 = k_4 + k_5 = k_5 + k_6 = k_6 + k_7 = k_7 + k_8 = k_8 + k_9 = 2k = 2.64 \times 10^6 \text{ kN/m}$$

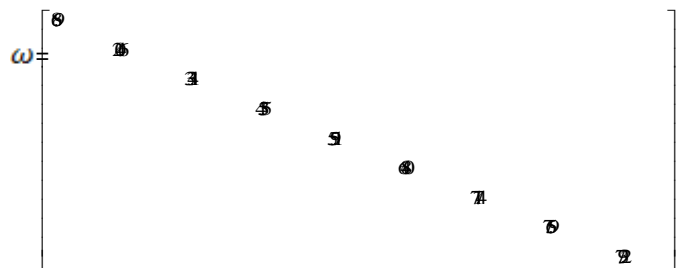
$$\begin{bmatrix} 2.64 & -1.32 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ -1.32 & 2.64 & -1.32 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -1.32 & 2.64 & -1.32 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1.32 & 2.64 & 1.32 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & -1.32 & 2.64 & -1.32 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -1.32 & 2.64 & -1.32 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & -1.32 & 2.64 & -1.32 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & -1.32 & 2.64 & -1.32 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & -1.32 & 1.32 \end{bmatrix}$$

All values are in 10^6 kN/m

For the above mass and stiffness matrices, Eigen values and Eigen vectors are worked out as follows:

$$[K - \omega^2 M] = 0$$

By solving the above equation, Eigen values and natural frequencies of various modes are ^[23]



The quantity of ω_i^2 , is called the i^{th} Eigen value of the matrix ^[3]. Each natural frequency (ω_i) of the system has a corresponding Eigen vector (mode shape), which is denoted by (ϕ_i) the mode shape corresponding to each natural frequency is determined from the equations.

$$[-M\omega_i^2 + K] \phi_i = 0$$

Solving the above equation, modal vector (Eigen vectors), mode shapes and natural periods under different modes are ^[23]

Eigen vectors $\{\Phi\}$

$$\{\phi\} = \{\phi_1 \ \phi_2 \ \phi_3 \ \phi_4 \ \phi_5 \ \phi_6 \ \phi_7 \ \phi_8 \ \phi_9 \}$$

-0.077	-0.222	-0.341	-0.419	-0.449	-0.426	0.356	0.249	0.124
-0.152	-0.386	-0.445	-0.024	-0.024	0.261	-0.428	-0.411	-0.238
-0.222	-0.450	-0.240	0.202	0.447	0.266	0.159	0.429	0.332
-0.286	-0.396	0.132	0.447	0.049	-0.424	0.237	-0.297	-0.398
-0.341	-0.240	0.412	0.121	-0.445	-0.006	-0.443	0.061	0.430
-0.386	-0.020	0.406	-0.360	-0.073	0.428	0.297	0.196	-0.426
-0.420	0.204	0.118	-0.380	0.441	-0.256	0.087	-0.385	0.386
-0.442	0.375	-0.252	0.087	0.097	-0.271	-0.401	0.439	-0.313
-0.450	0.449	-0.447	0.443	-0.435	0.422	0.396	-0.339	0.214

Th

\tilde{c}_1	\tilde{c}_2	\tilde{c}_3	\tilde{c}_4	\tilde{c}_5
0.0004	0.0031	0.0083	0.0153	0.0234
\tilde{c}_6	\tilde{c}_7	\tilde{c}_8	\tilde{c}_9	
0.0315	0.0390	0.0452	0.0498	

Corresponding modal mass, stiffness and damping terms are

$$\text{Model mass: } \tilde{m}_i = \phi_i^T M \phi_i$$

All values are in tonne

\tilde{m}_1	\tilde{m}_2	\tilde{m}_3	\tilde{m}_4	\tilde{m}_5
759.65	759.93	760.55	761.67	763.63
\tilde{m}_6	\tilde{m}_7	\tilde{m}_8	\tilde{m}_9	
767.12	773.68	786.40	807.39	

Model stiffness:

All values are in 10^6 kN/m

\tilde{k}_1	\tilde{k}_2	\tilde{k}_3	\tilde{k}_4	\tilde{k}_5
0.0361	0.3181	0.8489	1.5662	2.3874
\tilde{k}_6	\tilde{k}_7	\tilde{k}_8	\tilde{k}_9	
3.2208	3.9820	4.6126	5.0804	

Model damping: $C = \alpha K$

Where $\alpha = \frac{2\xi}{\omega}$, ξ is structural damping ratio of primary

mass and ω is natural frequency corresponding to fundamental time period calculated in step 2 along a particular direction. we have [3]:

$$f = 1/T \text{ and } \omega = 2\pi f \text{ so } f_x = 1.623 \text{ and } f_y = 1.88,$$

$$\alpha_x = 0.0098 \text{ and } \alpha_y = 0.0085, \text{ We have } \tilde{c}_i = \phi_i^T C \phi_i$$

All values are in 10^6 kN - s/m

Along x direction

Along y direction

\tilde{c}_1	\tilde{c}_2	\tilde{c}_3	\tilde{c}_4	\tilde{c}_5
0.00035	0.0027	0.0072	0.0132	0.0203
\tilde{c}_6	\tilde{c}_7	\tilde{c}_8	\tilde{c}_9	
0.0274	0.0338	0.0392	0.0432	

From above expression it is clear that x direction is critical one so i want to calculate damping ratio in ratio along x direction,

Damping ratio for initial four modes along x direction

$$\text{For Mode 1: } \xi_1 = \frac{\tilde{c}_1}{2\omega_1 \tilde{m}_1} = 0.038, \text{ For Mode 2:}$$

$$\xi_2 = \frac{\tilde{c}_2}{2\omega_2 \tilde{m}_2} = 0.0996, \text{ For Mode 3: } \xi_3 = \frac{\tilde{c}_3}{2\omega_3 \tilde{m}_3} =$$

$$0.1633 \text{ and For Mode 4: } \xi_4 = \frac{\tilde{c}_4}{2\omega_4 \tilde{m}_4} = 0.2214$$

Assume mass ratio 3% f_{opt} and ξ_{dopt} are calculated from equation given below for mass ratio 3% and damping ratio taken from different modes calculated above.

$$f_{opt} = \left(\frac{\sqrt{1-5\bar{m}}}{1+\bar{m}} + \sqrt{1-2\xi^2} - 1 \right) \cdot$$

$$(2.375-1.034\sqrt{\bar{m}}-0.426\bar{m}) \xi \sqrt{\bar{m}} -$$

$$(3.730-16.903\sqrt{\bar{m}}+20.496\bar{m}) \xi^2 \sqrt{\bar{m}}$$

$$\xi_{dopt} = \sqrt{\frac{3\bar{m}}{8(1+\bar{m})(1-5\bar{m})}} + (0.151 \xi - 0.170 \xi^2) +$$

$$(0.163 \xi + 4.980 \xi^2) \bar{m}$$

Table 2: Summary of above calculation for initial 4 modes with assumed mass ratio 3%

Mode	1	2	3	4
f_{opt}	0.947	0.913	0.871	0.819
ξ_{dopt}	0.11	0.121	0.130	0.138
m_d	112.5	112.58	114.192	114.36
k_d	4788.3	39270.5	96699	157737
c_d	161.47	508.93	864	1172.3

Similarly damping parameters are calculated for varying mass ratio 3.25, 3.5, 3.75, 4 and 4.5% for mode 1 so that we can show the effect of mass variation on analysis. Parameters obtain are as follows:

Table 3: Summary of above calculation of damping parameters for 1st mode with different mass ratio, that will be required as an input for modelling are as follows:

Mass ratio	3%	3.25%	3.50%	3.75%	4%	4.50%
f_{opt}	0.947	0.944	0.94	0.937	0.934	0.927
ξ_{dopt}	0.11	0.115	0.12	0.124	0.127	0.135
m_d	112.5	121.919	131.298	140.676	150.054	168.811
k_d	4788.3	5156.883	5513.276	5864.526	6210.716	6888.244
c_d	161.47	183.112	203.463	224.392	245.868	290.343

4. MODELLING IN ETABS:

4.1 Description of Models:

a) Model 1 = Time history Analysis without tuned mass damper

b) Model 2 = Time history Analysis with tuned mass damper for mass ratio 3%

(1) For mode 1 (2) For mode 2(3) For mode 3

c) Model 3 = Time history Analysis with tuned mass damper for mode 1 with mass ratio:

(1) 3% (2) 3.25% (3) 3.5% (4) 3.75% 5) 4% (6) 4.5%

All other parameters required for modelling are taken from table 1, 2 and 3.

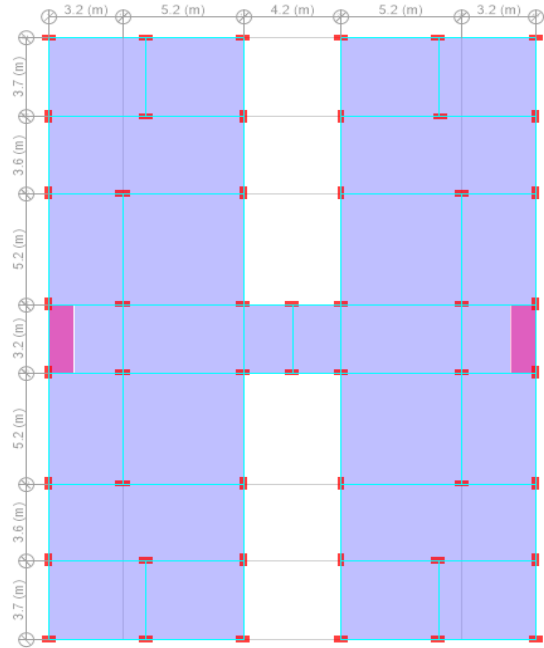


Fig. 4.1: Plan view of G+8 Residential building model

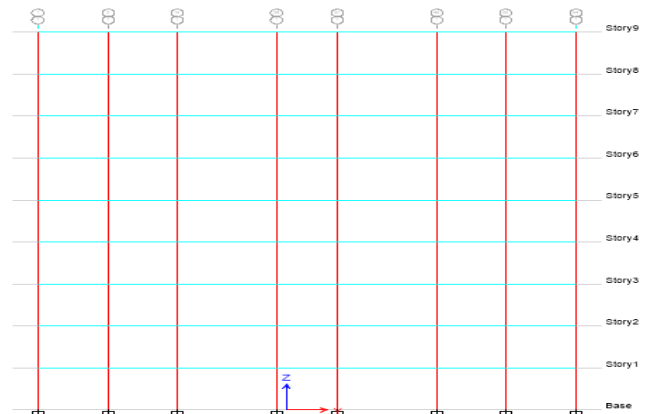


Fig. 4.2: Left elevation view of G+8 Residential building model

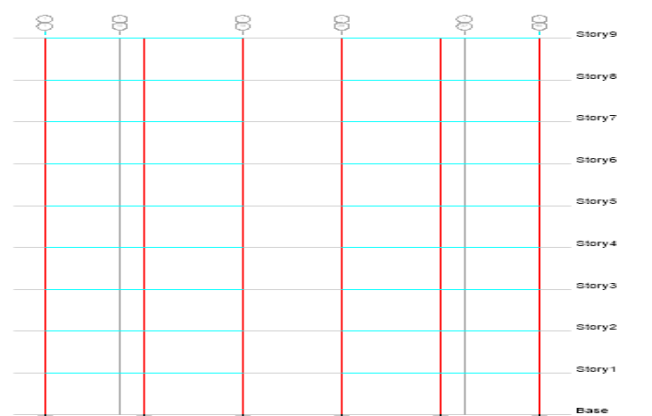


Fig. 4.3: Front elevation view of G+ 8 Residential building model

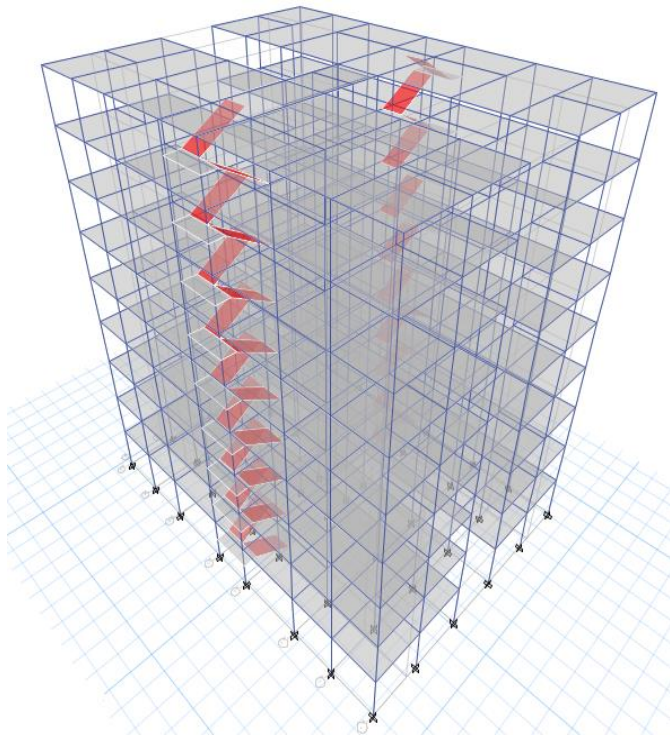


Fig. 4.4: Isometric view of G+ 8 Residential building model

5. RESULTS AND DISCUSSION:

5.1 STOREY DISPLACEMENT:

To determine story displacement time history analysis in x and y direction are to be done of a building without TMD and also with TMD for a particular mode and graph plotted to compare the reduction in responses for different modes also for different mass ratio peak story displacement are determine and graph also plotted for them to show the effect of mass variation.

Table 5.1: Displacement from time history analysis in x direction when mode 1 parameters are used from design of TMD.

Story	Elevation	Without TMD	With TMD
	m	mm	mm
Story 9	31.5	156.72	69.907
Story 8	28	149.77	25.288
Story 7	24.5	140.29	37.924
Story 6	21	127.37	45.517
Story 5	17.5	110.88	47.771
Story 4	14	93.5	46.604
Story 3	10.5	71.09	39.278

Story 2	7	44.63	26.677
Story 1	3.5	17.31	11.086

Table 5.1 and figure 5.1 shows that when TMD installed reduction in response occurs and it also shows that percentage response reduction is maximum nearer to the story where TMD is installed.

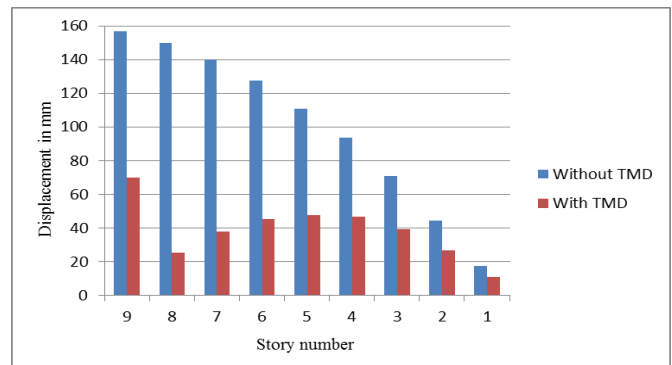


Fig. 5.1: Story displacement for THX using mode 1 parameters

Table 5.2: Displacement from time history analysis in x direction when mode 2 parameters are used from design of TMD.

Story	Elevation	Without TMD	With TMD
	m	mm	mm
Story 9	31.5	156.72	79.702
Story 8	28	149.77	67.81
Story 7	24.5	140.29	51.459
Story 6	21	127.37	43.6
Story 5	17.5	110.88	33.221
Story 4	14	93.5	17.905
Story 3	10.5	71.09	54.458
Story 2	7	44.63	1.556
Story 1	3.5	17.31	0.966

Table 5.2 and figure 5.2 shows that when TMD installed reduction in response occurs and it also shows that percentage response reduction is maximum nearer to the story where TMD is installed.

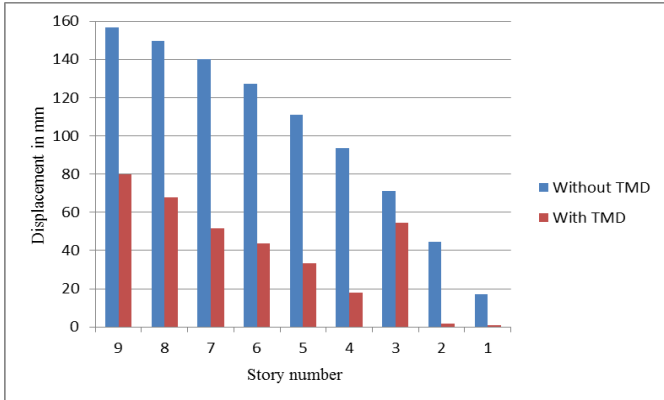


Fig. 5.2: Story displacement for THX using mode 2 parameters

Table 5.3: Displacement from time history analysis in x direction when mode 3 parameters are used from design of TMD

Story	Elevation	Without TMD	With TMD
	m	mm	mm
Story 9	31.5	156.72	48.194
Story 8	28	149.77	25.308
Story 7	24.5	140.29	37.94
Story 6	21	127.37	45.524
Story 5	17.5	110.88	47.775
Story 4	14	93.5	46.634
Story 3	10.5	71.09	39.259
Story 2	7	44.63	26.63
Story 1	3.5	17.31	11.06

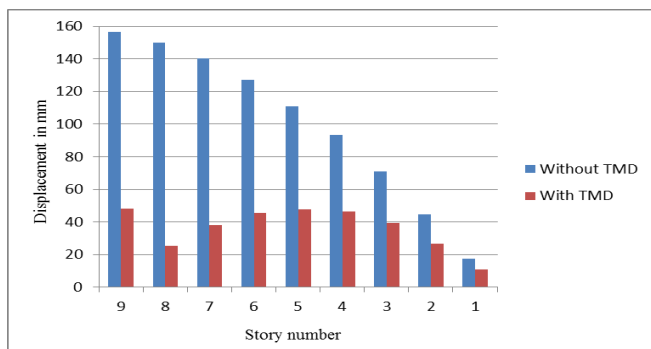


Fig. 5.3: Story displacement for THX using mode 3 parameters

Table 5.3 and figure 5.3 shows that when TMD installed reduction in response occurs and it also shows that

percentage response reduction is maximum nearer to the story where TMD is installed.

Similarly time history analysis in y direction also done to compare the above conclusion . From analysis we find similar results in y direction also.

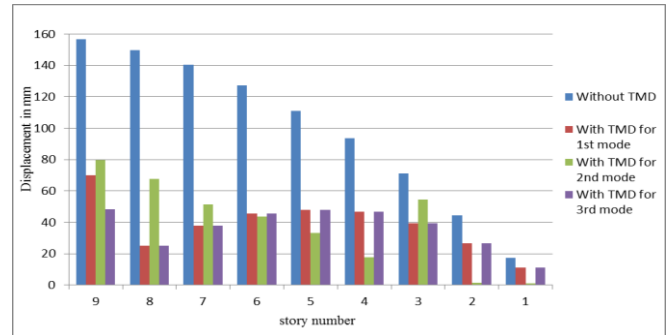


Fig. 5.7: Comparative story displacements for THX using different modes parameter

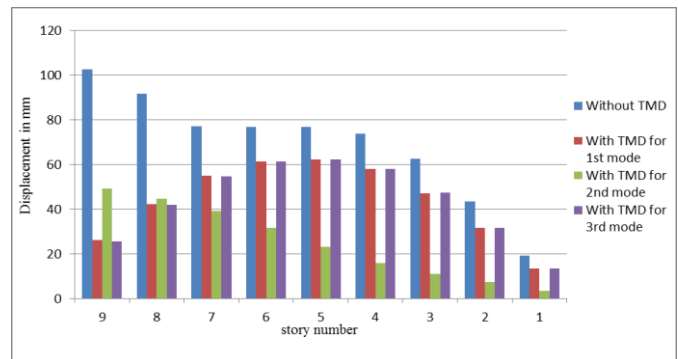


Fig. 5.8: Comparative story displacements for THY using different modes parameter

Summary of effect of TMD parameters used in initial three modes are also compared through figure 5.7 and 5.8 shown in x and y both direction respectively, its shows that when TMD installed reduction in response occurs and it also shows that percentage response reduction is maximum always nearer to the story where TMD is installed.

Table 5.7: Time history analysis in x direction for different mass ratio when mode 1 parameters are used from design of TMD.

mass ratio	Peak Displacement	story number
%	mm	
3	81.168	9
3.25	71.937	9
3.5	57.54	9
3.75	53.394	9
4	56.495	9
4.5	90.896	9

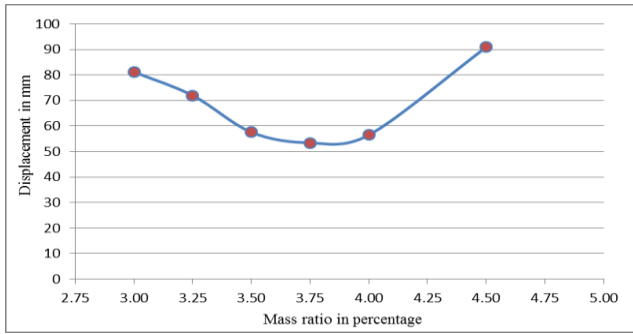


Fig. 5.9: Story displacement for different mass ratio

Above table and figure shows that as mass ratio increases percentage response reduction increases upto certain limit after then percentage response reduction decreases again and these variations are small in magnitude because time period of TMD independent on mass of damper but its causes small changes in overall structural time period by increasing mass of damper that's why small variation occurs.

Table 5.8: Damping parameters f_{opt} and ξ_{dopt} calculated for different mass ratio

Mass ratio	3%	3.25%	3.5%	3.75%	4%	4.5%
f_{opt}	0.947	0.944	0.940	0.937	0.934	0.927
ξ_{dopt}	0.11	0.115	0.120	0.124	0.127	0.135

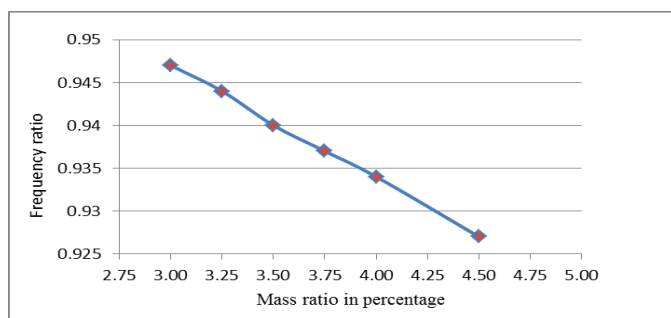


Fig. 5.10: Variation in frequency ratio for different mass ratio

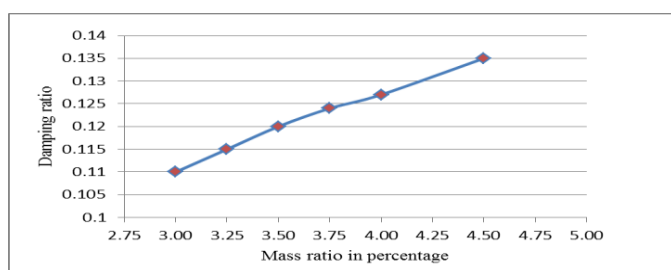


Fig. 5.11: Variation in damping ratio for different mass ratio

Above table and figure shows that when mass ratio increases corresponding frequency ratio decreases and damping ratio increases.

6. CONCLUSIONS:

- (1) Percentage response reduction is maximum to nearer stories where tuned mass damper is installed.
- (2) Conclusion has been made that maximum frequency ratio of PTMD decreases with increasing mass ratio and the effectiveness increases with the increase in mass ratio. With increase in mass ratio, the peak displacement is going on decreasing up to a particular mass ratio and again it is increasing on further increment of mass ratio.
- (3) In general, the optimal TMD has lower tuning frequency and higher damping ratio with increasing mass ratio.
- (4) It is observed that, after using damper optimum reduction is occurring at a frequency ratio nearer to the point of resonance. That is when the frequency ratio becomes nearer to unity. It is more effective in reducing the displacement responses of structures when tuned to fundamental frequency of the structure.
- (5) From this study, it can be concluded that properly designed TMD with efficient design parameters such as mass ratio, frequency ratio is considered to be a very effective device to reduce the structural response.

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