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Experimental and Numerical Study Ontuned Mass Damper in Controlling Vibration of Frame Structures

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ABSTRACT

Day by day, the numbers of taller and lighter structures are continuously increasing in the construction industries which are flexible and having a very low damping value.Those structures can easily fail under structural vibrations induced by earthquake and wind. Therefore several techniques are available today to minimize the vibration of the structure, out of which concept of using TMD is a newer one. There are large numbers of studies on theoretical investigation of behaviour of buildings with tuned mass dampers under various impacts. However, the experimental studies in this area are quite limited. In this thesis, a one-storey and a two-storey building frame models are developed for shake table experiment under sinusoidal excitation to observe the response of the structure with and without TMD. The TMD is tuned to the structural frequency of the structure keeping the stiffness and damping constant. Various parameters such as frequency ratio, mass ratio, tuning ratio etc. are considered to observe the effectiveness and robustness of the TMD in terms of percentage reduction in amplitude of the structure. Then the responses obtained are validated numerically using finite element method. From the study it is observed that, TMD can be effectively used for vibration control of structures.

INTRODUCTION

Earthquake is a compartment of structural analysis which involves the computation of the response of a structure subjected to earthquake excitation. This is required for carrying out the structural design, structural assessment and retrofitting of the structures in the regions where earthquakes are prevalent.

Now a day number of tall buildings are going on increasing which are quite flexible and having very low damping value to minimize increasing space problems in urban areas. These structures should be designed to oppose dynamic forces through a combination of strength, flexibility and energy absorption such that it may deform beyond elastic limit when subjected to severe earthquake motion. To make these structures free from earthquake and wind induced structural vibration, various techniques has been adopted which can be broadly classified into 4 categories.

(i)Active control,(ii) Passive control,(iii) Semi-active control and (iv) Hybrid control.

Active control devices:
These devices use an external power source which operates control actuators to apply forces to the structures. Some signals are sent to the actuators which are a function of responses of the structure. Requirement of equipments are more in active control strategies than passive control thereby increasing the cost and maintenance of such systems. Active tuned mass damper, active tuned liquid column damper and active variable stiffness damper are some of the examples of active control devices. Applications - AMD on Kyobashi Seiwa Building, Duox on ANDO Nighikicho, Trigon on Shinjuku Tower.

Passive control devices:
It is a device which imparts forces that are developed in response to the motion of the structures. By absorbing some of the input energy, it reduces the energy dissipation demand on the structure. Therefore no external power source is required to add energy to the structural system. Base isolation, tuned mass dampers (TMD),tuned liquid dampers (TLD),metallic yield dampers, viscous fluid dampers are some of the examples of passive control devices.

Applications - city halls of Oakland, US Court of Appeals in San Francisco (Friction Pendulum), NZ Parliament Building and the associated Assembly Library and the new Museum of NZ, USC Teaching Hospital in Los Angeles, Matu mura Research Institute building in Kobe.

Semi-active control devices:
It is a controllable passive control system where the external energy requirement is less than that of active devices. It unites the optimistic aspects of passive and active control devices. These devices generate forces as a result of the motion of the structure and cannot add energy to the structural system. Variable orifice dampers, variable friction dampers, variable stiffness damper, and controllable fluid dampers are some of the examples of semi active control devices.

Applications-Kajima Shizuoka Building in Shizuoka, Japan, Walnut Creek Bridge in Oklahoma, 11-storey building CEPCO Gifu Japan, Keio University School of Science and Technology Tokyo in Japan. Dongting Lake Bridge in Hunan, China.

Hybrid control Devices:
These devices combine the passive, active or semi-active devices to achieve higher level of performance. Since a portion of the control objective is accomplished by the passive system, less active control effort, implying less power resource, is required. A side benefit of hybrid systems is that, in the case of a power failure, the passive components of the control still offer some degree of protection, unlike a fully active control system. Examples of hybrid control devices include hybrid mass damper and hybrid base isolation.

Applications-Sendagaya INTES building in Tokyo

Practical implementation:
Till date many tuned mass dampers has been installed worldwide. Centre point Tower in Sydney, Australia is the first structure in which TMD was installed.

TMD is also installed in two buildings of United state. One is Citicorp Centre in New York City with its height 279 m, placed on the 63rd floor in topmost point of the structure, having a mass of 366 mg, with a linear damping from 8-14%, reducing the amplitude of the building by 50%.

Two dampers are installed in 60th floor of John Hancock Tower , Boston to reduce wind reduce structural vibration, each having its weight 2700 kN with a lead filled steel box 5.2msquare and 1m deep sliding on a 9m long steel plate.

Berlin TV tower, one of the tallest structures of Germany constructed between 1965 and 1969 with its height of 368 meters is installed with a tuned mass damper. The entrance of observation deck is 6.25 m above ground with 2 kone lifts for transport of visitors. Weight of the sphere is 4800 tones and diameter is 32 m. There is a steel stairway with 986 steps.

Burj Al Arab, a luxury hotel in Dubai, is the 3rd tallest hotel in the world is installed with 11 tuned mass damper. 39% of the total height is made up of non- occupiable space. There is helipad 210 m above sea level which provides an opportunity to arrive or depart from Burj Al Arab by helicopter and admire the city from a different perspective.

Taipai 101, Taiwan was the world's tallest building from 2004 to 2010, consists of 101 floors and 5 underground floor, equipped with a steel pendulum that serves as a tuned mass damper suspended from the 92nd to the 87th floor. The pendulum sways to offset movements in the building caused by strong gusts. It is the largest damper sphere in the world, consists of 41 circular steel plates of varying diameters, each 125 mm thick, welded together to form a 5.5 m diameter sphere. Two additional tuned mass dampers, each weighing 6 tonnes (7 short tons), are installed at the tip of the spire which help prevent damage to the structure due to strong wind loads. Spire of Dublin (Monument of Light), 121.2 m in height is the largest stainless steel monument located in Dublin, Ireland. It has an elongated cone of diameter 3m at the base and narrowing to15cm at the top. It is constructed from eight hollow tubes of stainless steel and equipped with a tuned mass damper to counteract sway.

The Akashi Kaikyo Bridge (Pearl Bridge) located in Japan has had the longest span of any suspension bridges in world. It has 3 spans with central span 1991 m and two others each 960 m long, designed to allow the structure to withstand winds of 286 km/Hr and earthquake of magnitude 8.5. A tuned mass damper is designed to operate at a resonance frequency of the bridge to increase the damping value.

LITERATURE REVIEW

Till date TMD has been studied by many researchers. The thought of TMD was first used by Frahm in 1909 to diminish the undulating motion of ships as well as ship hull vibrations. Later Hartog in 1940 developed analytical model for vibration controlling power of TMD. Later he optimized TMDs parameter for sinusoidal excitations. Fahim et al. (1997) considered different parameters like mass ratio, frequency ratio, damping ratio etc to obtain the optimum parameters which are used to compute the response of various single degree of freedom and multi degree of freedom structures with TMD at different earthquake excitation. The optimum parameters obtained are helpful in reducing the displacement and acceleration response significantly.

Wu et al. (1999) considered soil structure interaction in seismic response of tuned mass damper when fixed on a flexible based structure. A frequency independent model is used which covers a wide range of soil and structural characteristics. A stationary excitation is given to the model structure and the responses are used to measure the performance of TMD. It was observed that strong soil structure interaction considerably defeats the seismic effectiveness of TMD systems. Reduction in maximum response of the structure reduces with decrease in soil shear wave velocity. For any structure over soft soil, TMD structure is less effective in reducing the response due to high damping characteristics of soil structure system. The model is also subjected to NS component of the 1940 El Centro, California earthquake to observe the effectiveness of TMD in a realistic environment.

Nagashima et al. (2001) developed a hybrid mass damper (HMD) and applied it to a 36 storey high rise building with a bi-axial eccentricity, located in Tokyo. The system uses a gear type pendulum which make the natural period of the auxiliary mass relatively long minimizing the height of the device and a linear actuator which ensures smooth and noiseless operation of the system. Transverse torsion coupled vibration is controlled by two HMD systems and various feedbackcontroltechniqueshasbeendevelopedtoconsumethecapacityofHMDsystemforany external excitation. Free vibration tests as well as control of wind vibrations of the building induced by the Typhoon 9810 in 1998 were used to verify the performance of the control system. It was observed that the acceleration responses of the building were reduced to 63 %and47 % of the corresponding uncontrolled accelerations. Setareh (2001) studied the application of semi- active tuned mass dampers to base excitation systems. A single-degree-of-freedom system was subjected to sinusoidal base excitations and a damper was used to reduce the vibration.

Li and Liu (2002) manufactured an active multiple tuned mass damper (AMTMD) for structures subjected to ground acceleration keeping the stiffness and damping constant and varying the mass. Vibration in the structure is controlled by mode reduced order method. A numerical searching technique is used to demarcate the effect of optimum dynamic parameters on the strength of AMTMD. The parameters include the frequency spacing, damping ratio and frequency ratio and acceleration feedback gain coefficient. They compare the results of MTMD and AMTMD and concluded that it can effectively reduce the vibration of the structure under ground acceleration. The AMTMD can also increase the performance of MTMD and more effective than ATMD.

Samali et al. (2003) described a five storey model using an active tuned mass damper by Fuzzy Logic Controller and linear quadratic regulator under earthquake excitation and a comparison is made. The effect of mass ratio and frequency ratio is conducted using fuzzy controller because of its ability to handle any non linear behavior of the structure. Chen and Wu (2003) numerically observed the effect of multiple tuned mass damper (MTMD) and compared the results with tuned mass damper (TMD). A three storey building frame was subjected to white noise excitation and tested in shake table. The results observed that multiple tuned mass dampers are more effective than tuned mass dampers in reducing the floor acceleration. The experimental and numerical results are compared and dynamic properties of the structure are validated successfully.

Li (2003) numerically observed the performance of multiple active-passive tuned mass dampers (MAPTMD) to prevent vibration of single degree of freedom structures subjected to ground acceleration with a uniform distribution of natural frequency. The MAPTMD generates a controlling force by keeping the displacement and velocity response gain and changing the acceleration response gain. Conclusion has been made that maximum tuning frequency ratio of MAPTMD decreases with increasing mass ratio and the effectiveness increases with the increase in mass ratio.

Chen and Wu (2004) studied experimentally to reduce the seismic responses of a three-storey building structure by using multiple tuned mass dampers. They identified various dynamic properties of both structure and damper from free and forced vibration analysis. The structure was analyzed numerically with and without dampers and tested on shake table under white noise

excitation. Damper parameters are studied. Ghosha and Basu (2004) observed the effect of soil structure interaction and concluded that when the soil becomes stiff, it allows the foundation to move relative to the surrounding soil which changes the soil foundation system from that of the fixed base. In such a case a conventional TMD loses its effectiveness in controlling the response of the structure to base excitation. So to avoid the effect of SSI, it is necessary to tune the damper to the fundamental frequency of the structure–foundation system. It is also essential to provide damping in the TMD greater than the critical damping to ensure response reduction of the structure.

Kwok and Samali (2006) carried out some experimental verifications of both active and passive TMD and compare the results with parametric study which are very useful in selection of optimal TMD parameters. 40-50% reduction in wind induced response & an additional damping of 3-4% of critical damping by using passive system and 2/3rd reduction in wind induced response & an additional damping of 10% of critical damping by using active system was obtained from their experimental investigation.

Saidi et al. (2007) developed a Tuned mass damper using visco elastic material and concluded that TMD is effective when tuned to the natural frequency over a narrow band. They also describe the process of estimation of viscous damping of a damper made up of visco elastic material. For any given floor mass, damping and stiffness a damper can be an economical and simple solution for retrofitting floors with excessive vibrations. Uengetal.(2007)studied the Practical design issues of tuned mass dampers for torsionally coupled buildings under earthquake loadings and determined the optimal PTMD system parameters by minimizing the mean square displacement response ratio on the top floor of buildings with and without PTMDs.

Wong (2008) studied dissipation of seismic energy in inelastic structures with tuned mass dampers. By using the force analogy method, an inelastic structure is modeled which is chosen as the base of plastic energy dissipation analysis in the structure.

Alexander and Schilder (2009) proposed the performance of nonlinear tuned mass damper. A two degree of freedom system with a cubic nonlinearity is modeled. The nonlinearity is originated from geometric arrangement of two pairs of springs. One pair helps in providing linear stiffness whereas the other pair rotates as they extend and helps in hardening spring stiffness. In this paper a software AUTO has been used to study numerically the periodic response of a nonlinear tuned mass damper and the optimum design parameters has been observed. Lourenco et al.(2009) performed some experimental work taking a pendulum tuned mass damper with advantageous over conventional TMD.

Lin et al.(2010) studied the vibration control of seismic structures using semi-active friction multiple tuned mass dampers. In this paper a semi active friction type multiple tuned mass damper (SAF-MTMD)is developed to control vibration in seismic structures. Since a friction type mass damper is same as a conventional mass damper if the static frictional force inactivates the mass damper. Various friction mechanisms have been used to activate all the mass units of friction type multiple tuned mass damper during earthquake. A comparison study is made with a passive friction type multiple tuned mass dampers and concluded that SAF-MTMD effectively reduces the seismic motion particularly at a larger intensity. Islam and Ahsan (2012) optimized Tuned mass damper parameters using evolutionary operation algorithm and determined the optimum parameters of TMD in reducing the top storey response of the structure by using an evolutionary algorithm. They used El Centro NS earthquake to develop a computer program and found a higher percentage of reduction on the roof of a ten storey structure using TMD with the application of EVOP.

Objective and scope of present work:
The objective of the present work is to study experimentally and numerically the application of TMD to control vibration of both single and multi storey frame structure under sinusoidal excitation.

The scope of the work includes experimental modeling and analysis of single and multi-storey building frame under horizontal excitation with and without TMD considering different parameters like mass ratio, frequency ratio, tuning ratio. Linear time history analysis is carried out using finite element method and STAAD Pro with and without TMD under sinusoidal ground acceleration. The New mark Beta method is used to solve the dynamic equations for the structure-TMD system.

Forced Vibration Analysis Of A Frame Model:
Here the external force R(t) can be distributed into three mechanisms of the structure. First bythe stiffness component, second by damping component and third by mass component. So the dynamic response of the structure to the excitation can be expressed by the displacement $u(t)$, velocity $\dot{u}(t)$ and acceleration $\ddot{u}(t)$.

[M] =Global mass matrix of the frame model
[C] =Global damping matrix of the frame model
[K]=Global stiffness matrix of the frame structure

$\{U\}$=Global nodal displacement vector
$\{\dot{U}\}$=Global nodal velocityvector

$\{\ddot{U}\}$=Global nodal acceleration vector R(t) = Applied external force
A static analysis can be done using a simple linear equation [A]{x}={B}. Because in such a analysis time does not play any role. But dynamic analysis follows a complex governing equation like$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=R(t)$which depends on time.

Therefore, when a single degree of freedom system is subjected to a ground acceleration which varies arbitrarily with time, analytical solution of the equation of motion is usually not possible. Such problems can be solved by numerical time steeping methods to integrate the differential equations. It can be done by two approaches.

IMPLICIT INTEGRATION

Explicit Integration
Implicit solution is one in which the calculation of current quantities in one time step are based on the quantities calculated in the previous time step. This is called Euler Time Intergration Scheme. In this scheme even if large time steps are taken, the solution remains stable. This is also called an unconditionally stable scheme. This algorithm requires the calculation of inverse of stiffness matrix, since in this method we are directly solving for {U} vector and calculation of an inverse is a computationally intensive step. It is used in New marks beta method.

In an explicit analysis, instead of solving for {U}, we go for solving {U"}. Thus we bypass the inversion of the complex stiffness matrix, and we just have to invert the mass matrix [M].Incase lower order elements are used, which an explicit analysis always prefers, the mass matrix is also a lumped matrix, or a diagonal matrix, whose inversion is a single step process of just making the diagonal elements reciprocal. Hence this is very easily done. But disadvantage is that the Euler Time integration scheme is not used in this, and hence it is not unconditionally stable. So we need to use very small time steps. It is used in Wilson-theta method

Step-by-step solution using New mark integration method:

1. Formation of stiffness matrix K, mass matrix M and damping matrix C whichever is required.
2. Selection of time steps Δt.
3. Calculation of constants

$a_0=\frac{1}{\square\Delta t2}$, $a_1=\frac{\square}{\square\Delta t}$, $a_2=\frac{1}{\square\Delta t}$, $a_3=\frac{1}{2\square}-1$

$a_4=\frac{\square}{\square}-1$, $a_5=\frac{\Delta t}{2}\left(\frac{\square}{\square}-2\right)$, $a_6=\Delta t\,(1-\delta)$, $a_7=\delta\Delta t$

4. Initialization of U_0, $\dot{U}_0$ and $\ddot{U}_0$
5. Formation of effective stiffness matrix.

$\hat{K}.\hat{K}=K+a_0M+a_1C$

$\hat{K}.\hat{K}=K+a_0M$,neglecting damping in the structure.

6. For each time step,

- Calculation of effective load vector at time t+Δt.
  $\hat{R}_{t+\Delta t}=R_{t+\Delta t}+M\,(a_0U_t+a_2\dot{U}_t+a_3\ddot{U}_t)+C\,(a_1U_t+a_4\dot{U}_t+a_5\ddot{U}_t)$
  $\hat{R}_{t+\Delta t}=R_{t+\Delta t}+M\,(a_0U_t+a_2\dot{U}_t+a_3\ddot{U}_t)$, neglecting damping in the structure.
- Solution for displacement response at time t+Δt.
  $(\hat{K}.\hat{K})U_{t+\Delta t}=\hat{R}_{t+\Delta t}$
- Calculation of velocity and acceleration response at time t+Δt. $\ddot{U}_{t+\Delta t}=a_0(U_{t+\Delta t}-U_t)-a_2\dot{U}_t-a_3\ddot{U}_t$
  $\dot{U}_{t+\Delta t}=\dot{U}_t+a_6\ddot{U}_t+a_7\ddot{U}_{t+\Delta t}$

EXPERIMENTAL STUDY

Introduction:

Tuned mass damper is a low cost seismic protection technique which is implemented in many tall building and tower in the world without interrupting the use of the building. Thus till now various research works have been conducted to discover the effect of TMD to reduce the seismic shaking of the structure numerically. But experimental works under this field is quite limited.

The motive of this study is to reduce the response by attaching a tuned mass damper to the structure under sinusoidal loading and also to obtain the effect of various parameters such as mass ratio, frequency ratio, tuning ratio etc. on response of the structure. Ratio of damper mass to the mass of the structure is known as mass ratio, ratio of excitation frequency to the fundamental frequency of the structure is known as frequency ratio and the ratio of damper tuning frequency to structural frequency is known as tuning ratio.

For this experiment, shaking table test is conducted to study the dynamic behavior of a single and a double frame structure with and without TMD where it is subjected to sinusoidal ground motion. The structure is rigidly attached to the shaking table platform. The weight of the structure maybe regarded as concentrated at the roof level. Since a sinusoidal motion consists of a single frequency, it will provide a better understanding of the behavior of TMD-structure system. The fundamental frequency of the structure is determined from free vibration analysis.

Force vibration analysis is carried out by exciting the frame at various frequencies and the response is recorded.

Signal study is usually divided into time and frequency domains; each domain gives a different outlook and insight into the nature of the vibration.

Time domain analysis starts by analyzing the signal as a function of time. A signal analyzer can be used to develop the signal. The time history analysis plots give information that helps describe the behavior of the structure. Its behavior can be characterized by measuring the maximum vibration level.

Frequency analysis also provides valuable information about structural vibration. Any time history signal can be transformed into the frequency domain. The most common mathematical technique for transforming time signals into the frequency domain is called the Fourier Transform. Fourier Transform theory says that any periodic signal can be represented by a series of pure sine tones. In structural analysis, usually time waveforms are measured and their Fourier Transforms are computed. The Fast Fourier Transform (FFT) is a computationally optimized version of the Fourier Transform. With test experience, one can gain the ability to understand structural vibration by studying frequency data.

Experimental setup: The laboratory equipment consists of,

i. A unidirectional shaking table
ii. Vibrating analyzer
iii. Control panel
iv. Accelerometer
v. PC loaded with NVgate software
vi. Frame model with and without secondary mass

Unidirectional shake table:

The unidirectional shaking table is nothing but a 1m ×1m sliding stand regulated by an induction motor which operates a screw to impose horizontal motion. The screw sequentially operates a circular ball nut which is united to the sliding stand. The motor is driven by power supply. The sliding surface has 81 tie down points located on a 100x100 mm grid. The shake table can operate frequencies ranging from 0 to 20 Hz which can be control from the control panel. There is a system to fix up the maximum displacement of the shake table. It has the capacity to produce simple harmonic motion (sine wave forms). The maximum displacement of the table is 100 mm (± 50mm) with amplitude resolution of 5 mm. The maximum payload of the shake table is 100 kg.

Vibrating analyzer:
The vibrating analyzer provides a solution in the form of structural testing equipment for vibration measurement and analysis. It helps in converting the electrical signal to measured digital signal for analysis after which a variety of analysis and displays can be possible. The most common processing on dynamic data is to perform FFT analysis to convert the data to the frequency domain from where most of the data can be viewed. Thus it provides a fast, easy and accurate way to use time and frequency domain measurements for structural tests.

Accelerometer:
It is attached to the frame at the location where the acceleration needs to be measured. Further the data can be transformed to determine the displacement by a software known as NV gate loaded in PC. It has the provision to get the velocity and displacement response by integrating the acceleration data.

Control panel:
The control panel is managed by an input voltage of 440 volts. It can control the excitation frequency of the shake table ranging from 0- 20 Hz.

TMD structure model:
A one-storey and a two-storey building frame are developed for this experiment. The frame is supported by four columns of circular cross section of diameter 7.7 mm. The height of the column is 70 cm for single storey and 50 cm each for double storey. The roof of the frame is a rectangular iron plate of size 50 cm x 40 cm weighting 15.44 kg which is connected to the columns by nuts. An accelerometer is attached to the model to record the storey acceleration and displacement. The TMD is made up of various square iron plates of size 12.6 x 12.6 cm each having a weight of 0.707 kg, attached at the centre of roof plate by a circular rod. The frame is subjected to free vibration analysis to know the fundamental frequency of the structure. Then the damper is designed by tuning it to that frequency to obtain maximum response reduction at various mass ratios.

Weight of the plate=15.44 Kg Time-domain analysis for single storey frame:

In each case, displacement and acceleration response at the top of the frame is observed taking different mass ratios. From figure 4.3 and 4.4, it is observed that for frequency ratio of 0.8, optimum displacement and acceleration response of the frame without TMD are 26 mm and 2.1 m/s^2 which are reduced gradually with increase in mass ratio from 0.10 to 0.25. At mass ratio of 0.25, the value of maximum displacement and acceleration is found to be 5 mm and 0.5 m/s^{2}respectively.

Figure4.5and4.6explains the displacement and acceleration response at a frequency ratio of 1.0 (state of resonance). In this case, optimum displacement and acceleration response of the frame without TMD are found to be 70 mm and 8m/s^2 respectively which are reducing abruptly after attachment of TMD. At mass ratio of 0.25, the value of maximum displacement and acceleration is found to be 10 mm and 1 m/s^2 respectively.

From the above observations, it can be concluded that optimum reduction in peak displacement and acceleration for a particular frequency ratio is obtained at high mass ratio and the reduction is maximum when the frequency ratio is unity i.e, when the frame is subjected to a excitation frequency equal to the fundamental frequency of the structure.

Beating effect in structure mass damper system:
When the damping is very high in the secondary system, the combined system basically behaves as a SDOF system and the transfer of energy takes place in the coupled system which can stimulate vibrations in the primary structure instead of suppressing them. However, beyond a certain level of damping in the TMD, this beat phenomenon ceases and the structural response resembles SDOF decay. It basically appears when the exciting frequency nears the fundamental frequency of the structure.

Mechanically, we know that beats occur when two frequencies are close together. The difference between the two modal frequencies is defined as beat frequency.

Beat frequency, $f_{beat}= |f_1-f_2|$

Where, f_1 =fundamental frequency of the structure. f_2=excitation frequency given to the structure.
Beat period is the reciprocal of beat frequency and can be expressed as,

$T_{beat} = 1/f_{beat}$

Some experimental results are obtained to explain the beat phenomenon in combined structure- mass damper system showing in figure 4.7 to 4.10.

Effect of mass ratio on structural response for single storey frame:

Figure4.16describes the effect of various mass ratios in reducing the response, when the damper is tuned to the fundamental frequency of the primary structure.

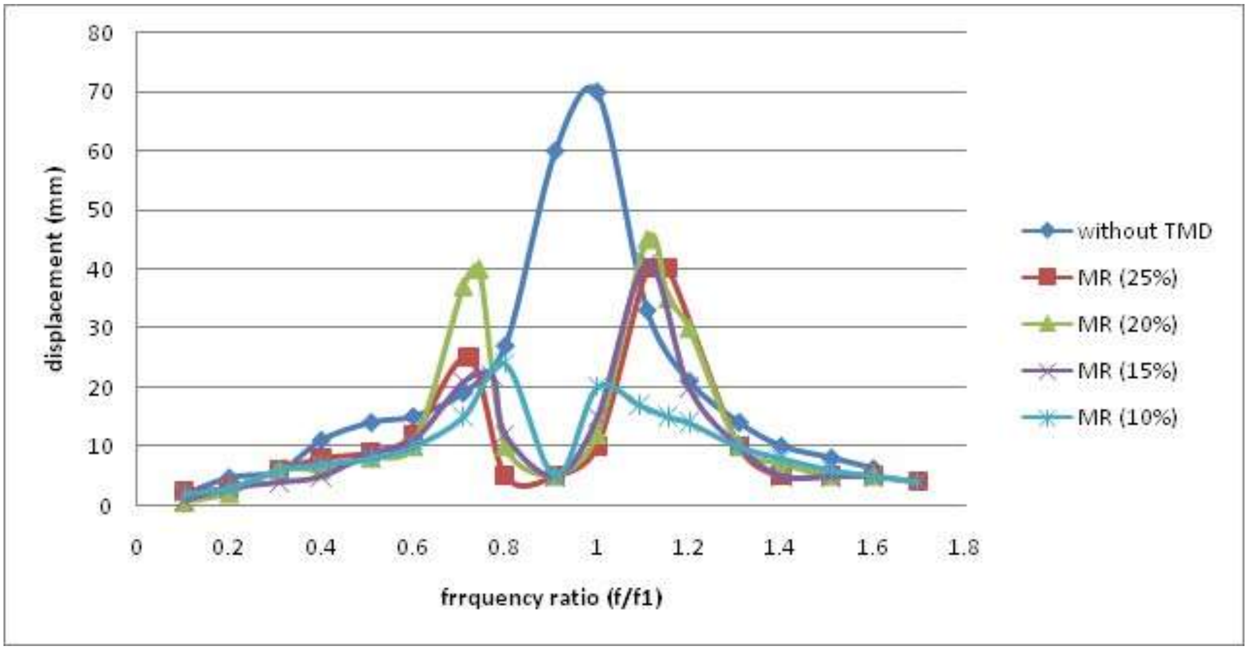


Figure 4.16 Displacement response of structure for varying frequency ratios with different mass ratio

It can be noticed that when the structure is subjected to sinusoidal excitation, maximum reduction is occurring at a frequency ratio of 0.9 which is nearer to the point of resonance. So the optimum TMD tuning frequency ratio obtained here is 0.9. 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. Here optimum reduction is occurring corresponding to a mass ratio of 0.15. Table-3 illustrates the percentage reduction in peak displacement as a function of mass ratio and frequency ratio.

Analysis of double storey frame:
The single storey frame model developed earlier is further extended to a double storey frame model by attaching another iron plate of same mass as previous one. Now the two storey frame as shown in figure 4.18 (a)is subjected to harmonic excitation defined by the expression,$x=x_0$
□^{2}sin(2πft)where,x_0 and f are the amplitude and frequency of excitation respectively which
Are the two varying parameters. Weight of each plate=15.44 Kg
Height of column in each floor(l)=50cm Diameter of column (D)= 0.77 cm
Fundamental frequency of frame (f) = 1.8 Hz (obtained from free vibration analysis) Displacement Amplitude =0.5 cm

Frequency Domain Analysis For Double Storey Frame:
The frame is excited under sinusoidal excitation containing various exciting frequency ranging from 0.18 Hz to 2.97 Hz and the signals obtained are studied both in time and frequency domain which gives two different out looks to examine the nature of vibration. The maximum amplitude response for each excitation frequency is obtained for each floor from corresponding time domain plots and presented graphically against the frequency ratio as shown in figure 4.19.

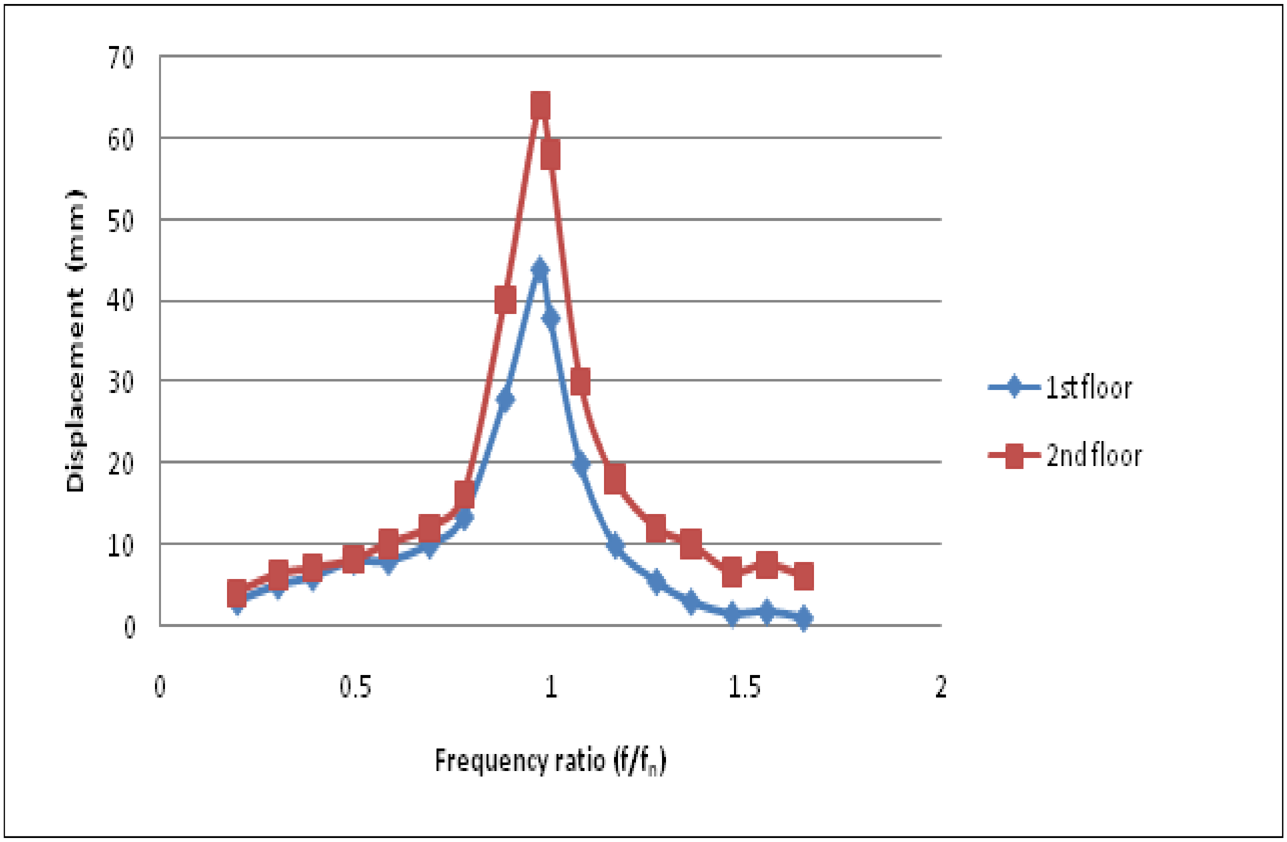


Figure 4.19 Displacement response of two storey frame structure forvarying frequency ratios without TMD

The nature of the frequency domain curve obtained for the two storey frame model in figure 4.19 resembles the curve in figure 4.11 obtained for single storey model. It can also be observed that the 1st floor response is always lower than the 2nd floor response.

Effect of frequency ratio on displacement response:

A mass damper is attached at top floor of the primary frame as shown in figure 4.18(b) to observe the reduction in response taking different mass ratio ranging from 0.025 to 0.126.In this study, various frequency ratios starting from 0.1to1.7 and mass ratio ranging from 0.025 to 0.126 are considered and the corresponding displacement responses are observed. At each mass ratio the damper is not tuned to the fundamental frequency of the primary structure.

At mass ratio of 12.5%, the damper is tuned to the fundamental frequency of the frame (1.8Hz). Stiffness of damper (k_d) = $m_d w^2$ (in N/m)= 486.08 N/m

Damper length(L_d)= $\left(\frac{3EI}{k_d}\right)^{\frac{1}{3}}$ (in cm)=43.42 cm

With decreasing the mass ratio, the length of the damper is increasing after tuning the damper to the fundamental frequency of the primary model which is not feasible for the experiment. Therefore the damper in tuned at only 12.5% mass ratio. Table 5 shows the different modal frequencies at different mass ratio.

NUMERICAL STUDY

Problem statement-1

The structural response of the one storey frame analyzed experimentally with and without TMD in the previous chapter is validated numerically using present FEM and STAAD Pro by subjecting the frame model under sinusoidal acceleration. Figure 5.2 and 5.3 show the top floor displacement of the single storey frame obtained in STAAD Pro without and with TMD respectively.

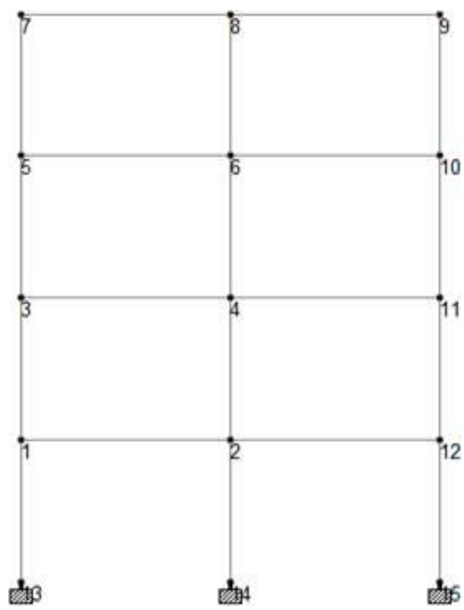
Problem statement-2

The dynamic amplitude response of a 4 and a 10 storey reinforced concrete frame model subjected to sinusoidal ground motion is analyzed with and without tuned mass damper.

A sinusoidal ground acceleration $\ddot{x}=x_0\square^2$ Sin (ω.t) is considered for time history analysis on

the 4-storey frame model with and without mass damper to show the dynamic structural response. Where, x_0 and ω are the maximum amplitude of acceleration and frequency of the sinusoidal acceleration respectively.
Following data are taken for the analysis.

Total height of the building=14m Height of each floor=3.5 m
Each bay width=5m No of bays =2
Size of beam= (0.25 x 0.35) m Size of column=(0.25x0.45)m Grade of concrete =M_{20}
Modulus of elasticity= 2.236 x10^6kN/m^2



Member load at roof level=23.75kN/m

Floor level=41.25kN/m Joint load at roof level=42.11 kN

floor level=73.28 kN Fundamental frequency of structure=1.373Hz Damping factor =0.05

Time step =0.01
Maximum acceleration amplitude=0.2 m/s^2

Effect of TMD on Displacement response with variation of damping ratio:
The effect of variation of damping ratio of TMD is studied through the amplitude response of the frame at a constant mass ratio of 0.01 under sinusoidal acceleration. From the figure 5.28 it is found that the response of the frame decreases with increase in damping ratio and then it will remains constant after a certain point.

CONCLUSION

The present study focuses on the capability of TMD in reducing the structural vibration induced due to earthquake. A single and a double storey frame model are examined experimentally with and without TMD to determine the structural response and presented in graphical and tabular forms. Effect of various parameters such as frequency ratio, mass ratio, damping ratio on the amplitude response has been studied with TMD. The results obtained are validated numerically using finite method. Further a four storey and a ten storey RC frame models are studied using Finite element method and STAAD Pro considering various parameters. The experimental and numerical investigation of various frame models under sinusoidal ground motion confirms that the structural response can be considerably reduced to a large extent by a properly designed TMD.

The Following Conclusions Are Made From The Study.

- When the frame is subjected to sinusoidal ground motion without TMD, amplitude becomes maximum at the point of resonance.
- From the experimental study, 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.
- 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.It is more effective in reducing the displacement responses of structures when tuned to fundamental (1st mode) frequency of the structure.
- It is more effective to use high damping ratio.
- At a higher beat frequency, beating effect is prominent which diminishes as the forcing frequency approaches to the fundamental frequency and no beating effect is observed at the state of resonance.
- From this study, it can be concluded that properly designed TMD with efficient design parameters such as tuning ratio, frequency ratio and mass ratio is considered to be a very effective device to reduce the structural response.

Future Scope For Study:

1. A linear model is considered in the experimental study. This can be studied by considering a non-linear model.
2. In current study the damper mass is placed at the top of the frame model and the reduction in response is observed. A further study can be carried out by placing the damper mass at different floors of the frame structure.
3. Effect of mass damper on response reduction can be studied by designing the damper mass in a pendulum shape.
4. The study can be further extended by using multiple tuned mass dampers (MTMD) tuned with various modal frequencies at different levels.
5. A future study can be done with active multiple tuned mass damp.

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