DOI: http://dx.doi.org/10.12989/sss.2011.8.3.239

Seismic response control of elastic and inelastic structures by using passive and semi-active tuned mass dampers

Sung-Sik Woo, Sang-Hyun Lee* and Lan Chung

Department of Architectural Engineering, Dankook University, 126, Yongin-city, Korea

(Received November 26, 2009, Revised January 1, 2011, Accepted April 13, 2011)

Abstract. In this study, the performances of a passive tuned mass damper (TMD) and a semi-active TMD (STMD) were evaluated in terms of seismic response control of elastic and inelastic structures under seismic loads. First, elastic displacement spectra were obtained for damped structures with a passive TMD and with a STMD proposed in this study. The displacement spectra confirmed that the STMD provided much better control performance than passive TMD and the STMD had less stroke requirement. Also, the robustness of the TMD was evaluated by off-tuning the frequency of the TMD to that of the structure. Finally, numerical analyses were conducted for an inelastic structure of hysteresis described by the Bouc-Wen model. The results indicated that the performance of the passive TMD whose design parameters were optimized for an elastic structure considerably deteriorated when the hysteretic portion of the structural responses increased, and that the STMD showed about 15-40% more response reduction than the TMD.

Keywords: semi-active TMD; inelastic structure; displacement spectra; stroke; robustness; Bouc-Wen Model.

1. Introduction

In recent years, many academic and practical attempts have been made to control wind- or earthquake-induced structural responses by using a mass type damper which transfers the inertia force resulting from auxiliary moving mass to the main structure (Soong and Dargush 1996). Such a mass type damper can be classified into passive, active, hybrid, and semi-active ones according to the use of force delivery devices integrated with real-time processing evaluators and sensors. Passive tuned mass damper (TMD), which is one of the most traditional mass type dampers, generally consists of moving mass, spring, and viscous damper. Major design parameters of the TMD are mass ratio to the main structure, tuning frequency ratio, and the damping ratio of the TMD itself. Although the larger mass ratio brings about the better control performance, TMD having mass ratio less than 2% is generally applied due to the cost, installation space limit, and increase of gravity load. For tuning frequency and damping ratios, there exist optimal values to provide the best control performance. Because TMD with off-tuned frequency and non-optimal damping ratio does not control but may excite the main structure, the spring stiffness and damper viscosity should be carefully determined based on the accurate identification of the structural frequency and characteristics of the excitation load. Also, TMD is conventionally utilized in the field of wind engineering since it is effective in reducing the stationary first modal response.

*Corresponding Author, Associate Professor, E-mail: lshyun00@dankook.ac.kr

Seismic effectiveness of TMD has been investigated by many researches. Sadek *et al.* (1997) briefly reviewed previous studies on the seismic application of TMD and demonstrated through statistical analysis using 52 ground motions measured in the western parts of the United States that TMD could control earthquake-induced responses if the parameters of TMD for the structure-TMD system are set up such that the damping ratios of the first two complex modes become identical. Jer-Fu *et al*. (2009) proposed two-stage optimum design procedure for passive TMD with the stroke limitation of TMD. In the first stage, the frequency ratio and damping ratio of TMD are derived without considering the TMD stroke, and then the parameters are optimized by adopting weighting factor for the TMD stroke in the performance index. Leung *et al.* (2008) suggested optimum parameters including the optimum mass ratio, damper damping and tuning frequency of the non-stationary TMD system by using particle swarm optimization (PSO) algorithm. Luigi Petti *et al.* (2010) investigated the effectiveness of the base isolation and tuned mass damping combined control strategy.

Semi-active TMD(STMD) was proposed by a few studies in order to improve seismic effectiveness of the passive TMD. Setareh (2001) designed a STMD, which can modulate its viscosity, and verified the excellence of the STMD over TMD in terms of control of a structure response subject to sinusoidal base excitation. Setareh applied the so called ‘ground hook’ control algorithm, in which the force transferred through the viscous damper to the structure adds structural damping. Jiang and Hanagan (2006) designed a STMD with piezoelectric friction dampers to floor vibration control. Yang *et al.* (2002) presented a STMD of which damping was instantaneously determined based on an off-and-towards-equilibrium (OTE) algorithm and showed that the STMD was effective in reducing vibration of a five-storey and a ten-storey shear structures subjected to seismic excitations. Kim *et al.* (2006) proposed a STMD whose damping plays a role similar to the restoring force of the structure by adopting magneto-rheological (MR) damper. F. Amini and R. Doroudi (2010) showed that semi-active control logic could effectively mitigate the seismic response of the building complex which is formed of one main building and one podium structure connected through MR Dampers and TMD.

The results from previous researches indicate that structural damping considerably impairs the efficiency of the passive TMD and a larger mass ratio is required to maintain the performance of TMD. Also, most studies on seismic applications of TMD and STMD were mostly focused on elastic structures with constant small structural damping, but most structures subject to earthquake show inelastic behaviors and as a result, both structural period and damping increase. Therefore, in order to verify that TMD is effective in seismic application, the performance of TMD should be evaluated by considering the inelastic behavior of structures.

In this study, the performances of the passive TMD and STMD are investigated in terms of the seismic response control of elastic and inelastic structures. First, elastic displacement response spectra are obtained for the damped structures controlled by passive TMD and by STMD whose stiffness and viscosity are modulated to switch from their maximum values to minimum ones. Also, the robustness of TMD is evaluated by off-tuning the frequency of TMD to that of the structure. Finally, the effects of TMD and STMD are evaluated for an inelastic structure of which hysteretic characteristics are described by the Bouc-Wen model.

2. Equation of motion

The hysteretic characteristics of an inelastic structure can be described by using the well known Bouc-Wen model. The governing equation of the inelastic structure with TMD is

$$\begin{bmatrix} m_s & 0 \\ 0 & m_t \end{bmatrix}\begin{bmatrix} \ddot{x}_s \\ \ddot{x}_t \end{bmatrix} + \begin{bmatrix} c_s + c_t & -c_t \\ -c_t & c_t \end{bmatrix}\begin{bmatrix} \dot{x}_s \\ \dot{x}_t \end{bmatrix} + \begin{bmatrix} \alpha k_s + k_t & -k_t \\ -k_t & k_t \end{bmatrix}\begin{bmatrix} x_s \\ x_t \end{bmatrix} = \begin{bmatrix} -(1-\alpha)k_s D_y \\ 0 \end{bmatrix}\eta - \begin{bmatrix} m_s \\ m_t \end{bmatrix}\ddot{x}_g \tag{1}$$

where m_s, c_s, and k_s denote, respectively, the mass, damping, and initial stiffness of the structure, and m_t, c_t, and k_t denote, respectively, the mass, damping, and stiffness of TMD. x_s and x_t, are, respectively, the relative displacements of the structure and TMD to the ground and $\ddot{x}_g$ is the ground acceleration. α denotes the ratio of post-yielding stiffness to the initial one and D_y is the yielding displacement. $\alpha = 1$ means that the structure behaves elastically. η is an non-dimensional variable adopted to describe hysteresis, and it is governed by the following differential equation.

$$D_y \dot{\eta} + \gamma|\dot{x}_s||\eta|\eta^{n-1} + \beta \dot{x}_s \eta^{n-1} - A\dot{x}_s = 0 \tag{2}$$

where γ, β, n and A are parameters associated with the magnitude, shape and the smoothness of the hysteretic curve.

Whittaker *et al.* (1998) described the bi-linear behavior using the Bouc-Wen model with $\gamma = 0.5$, $\beta = 0.5$, $n = 5$, and $A = 1$. Fig. 1 shows various hysteretic curves of two SDOF systems of 1.0 and 0.3 natural periods according to α. The notation e denotes the ratio of yield displacement of an inelastic structure to peak displacement of corresponding elastic structure. It is observed in Fig. 1 that bi-linear behavior can be accurately modeled. Since bi-linear curve cannot consider pinching and stiffness degradation phenomenon in local inelastic elements, the hysteresis model used in this study is more appropriate for steel structures than concrete one. However, the motion of TMD is mainly correlated with the global structural behavior like relationship between the base-shear and top floor displacement which can be usually approximated by bi-linear curve, the Bouc-Wen model using parameters presented by Whittaker *et al.* are used for modeling inelastic behavior of general building structures in this study.

The mass, stiffness and damping of TMD were determined based on the following equation.

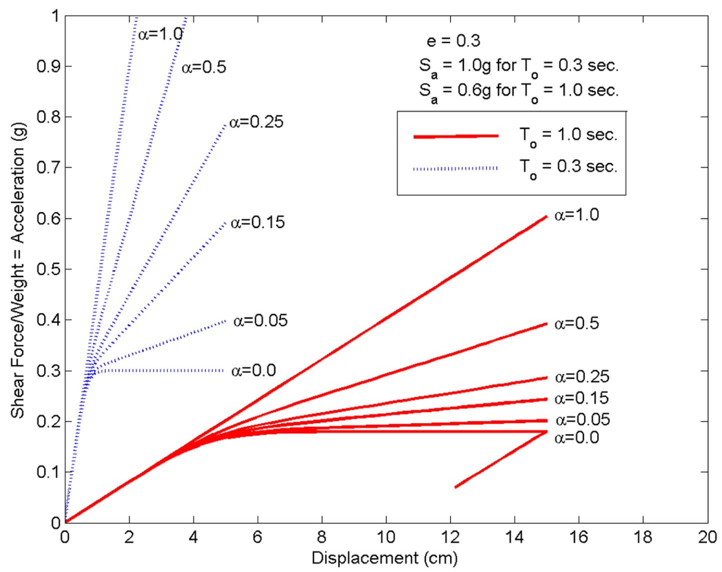
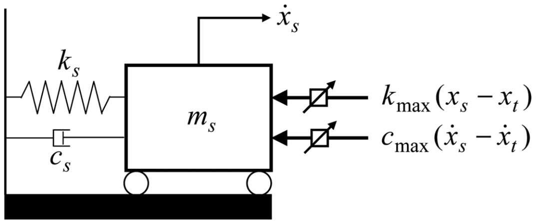


Fig. 1 Force-displacement relationship of inelastic structures (Whittaker *et al.* 1998)



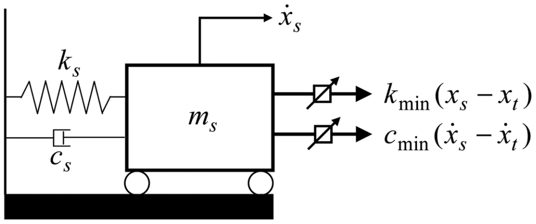


Fig. 2 Variation of stiffness and viscosity of STMD: (a) maximum stiffness and viscosity and (b) minimum stiffness and viscosity

$$m_t = \mu m_s,\ k_t = f_r^2 \omega_1^2 m_t,\ c_t = 2\xi_t \sqrt{m_t k_t} \tag{3}$$

where μ is the ratio of TMD mass to the effective modal mass of the structure, f_r is the tuning frequency ratio, ω_1 is the first modal radial frequency of the structure, and ξ_t is the damping ratio of TMD.

Passive TMD is designed to keep f_r and ξ_t constant, and accordingly, the stiffness and damping do not change during its lifetime. Extensive research has been conducted to find the optimal values of f_r and ξ_t, which are known to be dependent on the mass ratio, structural damping, type of excitation, and the optimization criteria.

For an undamped SDOF system subject to harmonic excitation, TMD having the identical natural frequency to the structural one and zero damping perfectly eliminates the resonant response of the main structure (Chopra 2001). So zero damping can be said to be optimal in controlling resonant responses of an undamped SDOF system under harmonic load. TMD with zero damping, however, amplifies other frequency responses than resonant one and requires too excessive stroke of the TMD itself. So, the TMD is generally designed to have some damping (so called optimal damping) in order to provide response reduction in overall frequency bandwidth although perfect cancellation of the resonant response cannot be realized due to the damping (Soong and Dargush 1998). Also, even when the frequency feature of the excitation load is similar, the optimal values of f_r and ξ_t, differs according to the loading type, that is earthquake or wind excitation, because ground acceleration in earthquake is directly transferred to the TMD mass while wind load is applied only to the main structure. Moreover, the structural period and damping which influences on the optimal values of f_r and ξ_t, is expected to increase with large deformation induced by seismic load. Accordingly, the optimal design of TMD in the seismic engineering should be conducted by considering these conditions.

Sadek *et al.* (1997) presented regression equations for determining the optimal f_r and ξ_t based on eigenvalue analysis of a damped structure with TMD and showed that the mean response spectrum for 52 ground accelerations can be reduced over all structural periods by using a passive TMD. Since the consideration of both the effects of structural damping and seismic loads in the study by Sadek *et al.* is in accordance with the purpose of this study, the following tuning frequency ratio and damping ratio presented by Sadek *et al*. are used in the design of the passive TMD in this study.

$$f_r = \frac{1}{1+\mu}\left(1 - \xi_s\sqrt{\frac{\mu}{1+\mu}}\right) \qquad \xi_t = \frac{\xi_s}{1+\mu} + \sqrt{\frac{\mu}{1+\mu}} \tag{4}$$

Where, ξ_s is the structural damping. With increasing ξ_s, f_r decreases but ξ_t increases.

3. Control algorithm for SATMD

A passive TMD mitigates mainly the vibration energy corresponding to the tuning frequency. Rana and Soong (1998) investigated the performance of multiple TMDs whose masses were tuned to different modes of a multi-degree-of-freedom structure and verified that the multiple TMDs designed as so were not effective in structural control because the masses tuned to higher modes excited the first mode responses. This fact indicates that a passive TMD has a limitation in that it should be designed to tune the fundamental frequency and the structural response should be governed by the tuned mode.

An active TMD (ATMD) adopting an additional excitation system and an STMD were proposed to improve the performance of the passive TMD. The ATMD and STMD controlled the force of TMD transferred to the structure. The performance of ATMD or STMD is known to be better than the passive TMD. ATMD, however, requires additional power supply, computer for computing control force, sensors for measuring structural responses, and a signal processing system, which makes the application of the ATMD impractical. Although STMD also requires supplemental devices for controlling the stiffness or damping and measuring structural responses, STMD is stable and economical because STMD does not utilize an exciter requiring a large power supply. Accordingly, this study focused on the application of STMD to seismic engineering and compared the performance of the STMD to that of the passive TMD.

When $\alpha = 1$, the first row of Eq. (1) can be expressed as follows by transposing the TMD generated force to the right side.

$$m_s\ddot{x}_s + c_s\dot{x}_s + k_s x_s = -m_s\ddot{x}_g + c_t(\dot{x}_t - \dot{x}_s) + k_t(x_t - x_s) \tag{5}$$

Eq. (5) indicates that the TMD generates damping and restoring forces which result, respectively, from the velocity and displacement of the TMD relative to the TMD-installed floor. The derivative of the structural conservatory energy with respect to time is

$$\begin{aligned} dE/dt &= d(0.5m_s\dot{x}_s^2 + 0.5k_s x_s^2)/dt = m_s\dot{x}_s\ddot{x}_s + k_s x_s\dot{x}_s \\ &= -c_s\dot{x}_s^2 - m_s\ddot{x}_g\dot{x}_s + c_t\dot{x}_s(\dot{x}_t - \dot{x}_s) + k_t\dot{x}_s(x_t - x_s) \end{aligned} \tag{6}$$

Eq. (6) implies that the spring or damper of the TMD reduces the conservatory energy only when the sign of the displacement or velocity of the TMD relative to the TMD-installed floor is opposite to that of the velocity of TMD-installed floor while the structural damping c_s always plays a role of decreasing the conservatory energy.

Most previous studies on STMD considered only the damping term as a variable or controllable while stiffness remained unchanged because viscosity can be easily modulated by adopting the MR damper whose viscosity varies in milliseconds according to the applied magnetic field. Yamada and Kobori (1995) proposed an active variable stiffness (AVS) system using a Chevron brace-beam connection detail, in which the connection state can be controlled between ‘connected’ and ‘disconnected’. In this study, it is assumed that both the stiffness of the spring and damping of the viscous damper can have maximum and minimum values simply by changing the connection states of the elements which have stiffness and viscosity.

The spring and viscous damper of the STMD have maximum stiffness and viscosity, respectively, when they play a same role of reducing the conservatory energy as the structural damping while they have minimum ones when they increases the conservatory energy of the main structure. The control algorithm is as follows.

$$c_t = \begin{bmatrix} c_{\max} & \dot{x}_s(\dot{x}_t - \dot{x}_s) \le 0 \\ c_{\min} & \dot{x}_s(\dot{x}_t - \dot{x}_s) > 0 \end{bmatrix} \qquad k_t = \begin{bmatrix} k_{\max} & \dot{x}_s(x_t - x_s) \le 0 \\ k_{\min} & \dot{x}_s(x_t - x_s) > 0 \end{bmatrix} \tag{7}$$

Where $c_{\max}$, $c_{\min}$, $k_{\max}$ and $k_{\min}$ are, respectively, maximum viscosity, minimum viscosity, maximum stiffness and minimum stiffness.

4. Numerical example

4.1 Elastic structures

In this section, numerical analyses of mass normalized elastic SDOF systems with the TMD and STMD are performed by using 20 accelerograms measured in rock sites (Lee *et al.* 2005). All the results for elastic structures are obtained by averaging the peak responses induced by 20 accelerograms. Fig. 3 shows the averaged peak displacement response spectra of the controlled and uncontrolled structures and Fig. 4 shows the corresponding coefficient of variation of the response spectra.

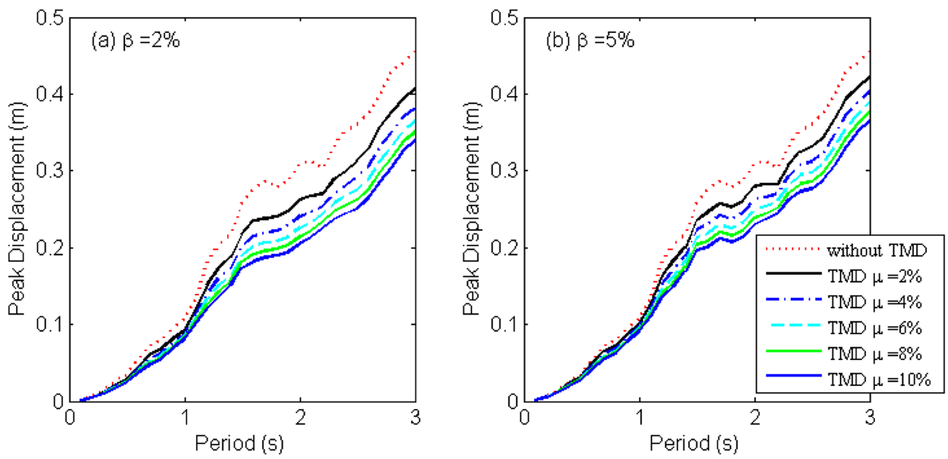


Fig. 3 Peak displacement response spectra of uncontrolled and controlled: (a) $\xi_s = 2\%$ and (b) $\xi_s = 5\%$

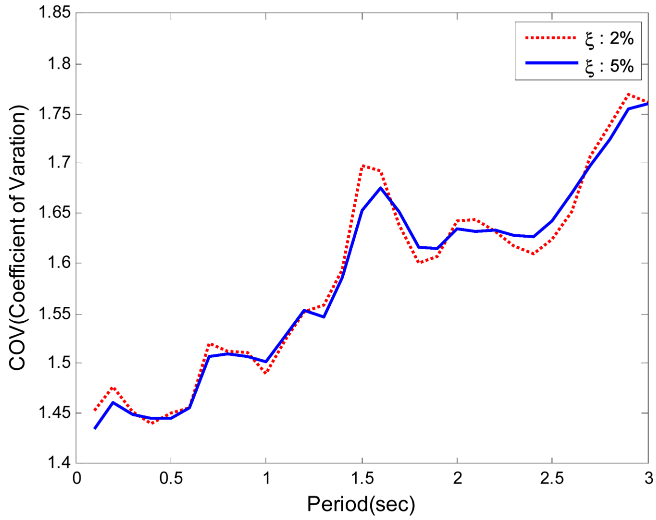


Fig. 4 Coefficient of variation by period

4.2 Displacement response spectra of SDOF systems with the passive TMD

Fig. 5 shows displacement response spectra of the TMD-installed structures. The spectra are normalized to the value of uncontrolled response for easy identification of control performance of the TMD. The tuning frequency ratio and damping ratio of the passive TMD are determined by using Eq. (4). The structural damping ratios of 2% and 5% are considered, and the response spectra are normalized to the uncontrolled one for easy identification of control effectiveness. The seismically induced peak displacement can be reduced by using an optimally designed passive TMD over all structural periods. The reduction effects become significant with increasing mass ratio. The comparison between Figs. 5(a) and (b) shows that the increase of structural damping from 2% to 5% causes the deterioration of the performance of the TMD when identical mass ratio is used. Considering that structural damping generally increases when a structure experiences large deformation under seismic

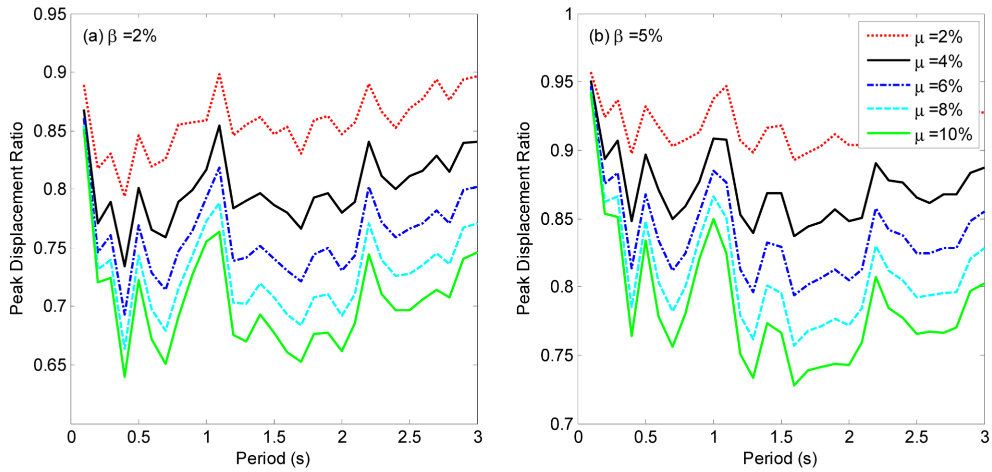


Fig. 5 Displacement response spectra: (a) $\xi_s = 2\%$ and (b) $\xi_s = 5\%$

load, the performance of a passive TMD, which is good for lightly damped structures, should be evaluated carefully based on the accurate assessment of the structural damping expected under seismic load.

4.3 Robustness of the passive TMD

The performance of the passive TMD observed in Fig. 5 is based on the accurate identification of the frequency of the structure and subsequent accurate tuning of the TMD. An inaccurately tuned TMD may amplify the structural response. In seismic application, it is very difficult to accurately identify the structural frequency since the target frequency should be determined to consider to some degree the crack of the concrete element or the damage of the non-structural elements which affect the frequency. This situation is different from the condition in wind engineering, in which the initial stiffness of the structure including the effect of non-structural elements is considered and the corresponding frequency can be measured on site. Accordingly, in this section, the robustness of the passive TMD is investigated by varying the tuning frequency ratio under the assumption that the frequency of the structure used in the design of the TMD may have 10% error.

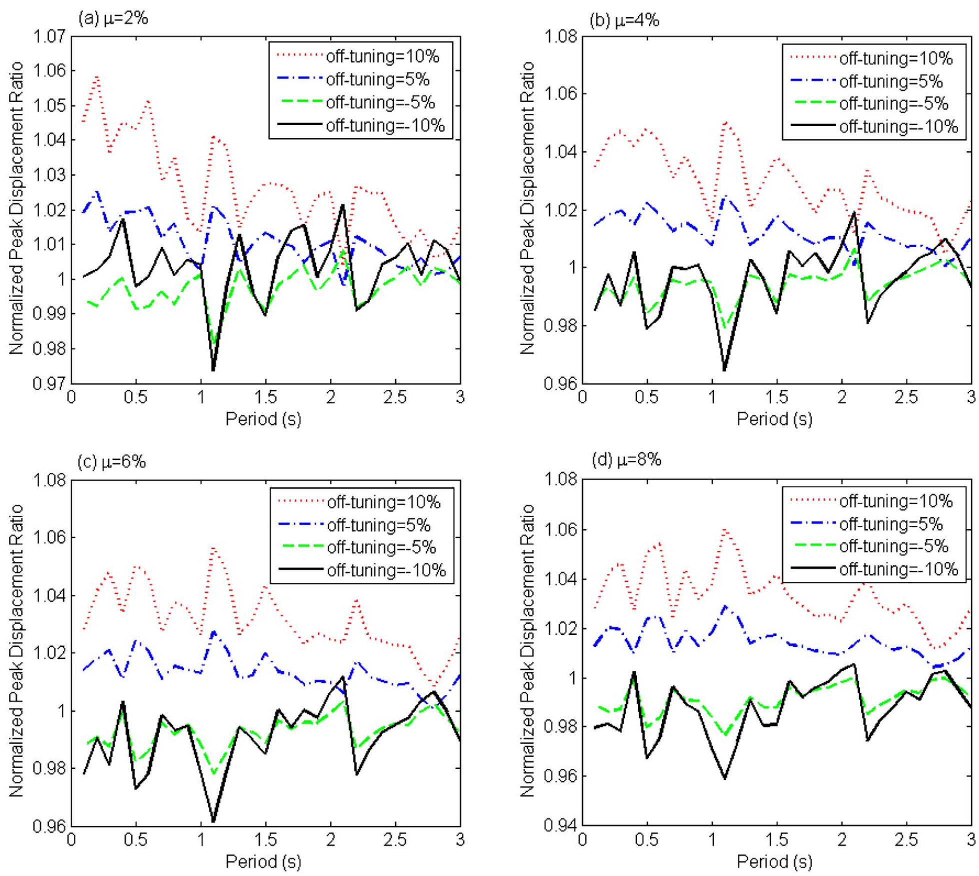


Fig. 6 Normalized peak displacement considering off-tuning effect: (a) $\mu = 2\%$, (b) $\mu = 4\%$, (c) $\mu = 6\%$ and (d) $\mu = 8\%$

Fig. 6 shows the variation of the peak displacement for a tuning frequency ratio different from the one in Eq. (4). All the values in Fig. 4 are normalized to those obtained by using Eq. (4), and thus, a value over 1 means the deterioration of performance of the TMD and the value less 1 indicates that the optimal tuning frequency ratio is different from Eq. (4) for the earthquake loads used in this study. Fig. 6(a) with the 2% mass ratio shows that all the (+) off-tuning cases gave values over 1 while the (-5%) off-tuning case gave a value slightly less than 1. Figs. 6(a)-(b) show that the performance variation due to the off-tuning effect becomes insignificant with increasing mass ratio. Since (+) off-tuning always brings about undesirable results, the frequency of the structure should not be overestimated.

4.4 Displacement response spectra of SDOF systems with the STMD

In this section, the performance of the STMD is evaluated by comparison with that of the passive TMD. The 2% structural damping ratio and 2% mass ratio for both the STMD and TMD are used. Following 4 cases are considered for designing the STMD.

Case-1: $k_{\max} = 10k_d$, $k_{\min} = 0.1k_d$, $c_{\max} = 10c_d$, $c_{\min} = 0.1c_d$
Case-2: $k_{\max} = 5k_d$, $k_{\min} = 0.1k_d$, $c_{\max} = 5c_d$, $c_{\min} = 0.1c_d$
Case-3: $k_{\max} = k_d$, $k_{\min} = k_d$, $c_{\max} = c_d$, $c_{\min} = 0.1c_d$
Case-4: $k_{\max} = k_d$, $k_{\min} = k_d$, $c_{\max} = c_d$, $c_{\min} = 0.1c_d$ (different control algorithm from Case-3)

where k_d and c_d are, respectively, the stiffness and viscosity obtained by using the optimal tuning frequency ratio and damping ratio in Eq. (4). Both the stiffness and viscosity, respectively, switch from $k_{\min}$ and $c_{\min}$ to $k_{\max}$ and $c_{\max}$ for Case-1 and Case-2 while only viscosity is variable for Case-3 and Case-4. The property of the STMD for Case-4 is identical to the one for Case-3, but the control algorithm presented by Kim *et al*. (2006) are used for the Case-4. The control logic used for the Case-4 is as follows

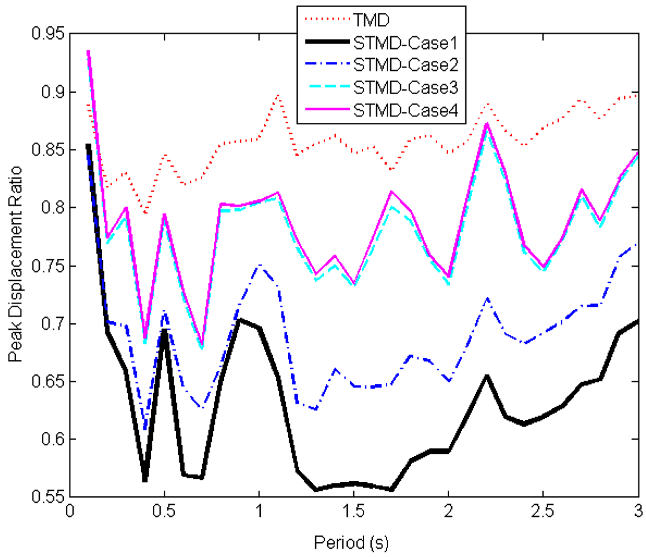


Fig. 7 Displacement response spectra of structures with STMD and TMD

$$c_t = \begin{cases} c_{\max} & x_s(\dot{x}_t - \dot{x}_s) \le 0 \\ c_{\min} & otherwise \end{cases} \tag{8}$$

The displacement response spectra of SDOF systems with the TMD and STMD are shown in Fig. 7. The STMD significantly improves the peak displacement mitigation performance of the TMD. The performances of the STMD for Case-1 and Case-2 are superior to those for Case-3 and Case-4, and Case-1 having larger variable stiffness magnitude provides more response reduction effect than Case-2. This fact implies that, not only the viscosity but also the stiffness of the STMD should be changed based on control logic, and the variation magnitude should be as large as possible to obtain better control performance. Case-3 proposed in this study shows better performance than Case-4, although the difference is not so significant.

4.5 Time histories of displacement and stroke

Fig. 8(a) compares the displacement time histories of a SDOF system which has 2% damping ratio and 1.0 second natural period and is excited by El Centro (1942, NS component) whose peak acceleration is scaled to 0.3 g. The property of the TMD having 2% mass ratio is determined by using Eq. (4), and Case-1 is considered for the STMD. It is obviously shown in Fig. 8(a) that the STMD is much more effective in reducing both initial non-stationary response and peak displacement than the passive TMD. Because the large stroke in a mass type damper causes a stability problem in which a moving mass may strike the rail end or reference wall, the TMD generally has an impact-proof bumper, and the stroke expected under a given load should be estimated. Fig. 8(b) compares the strokes of the TMD and STMD. The peak strokes of the TMD and STMD are, respectively,

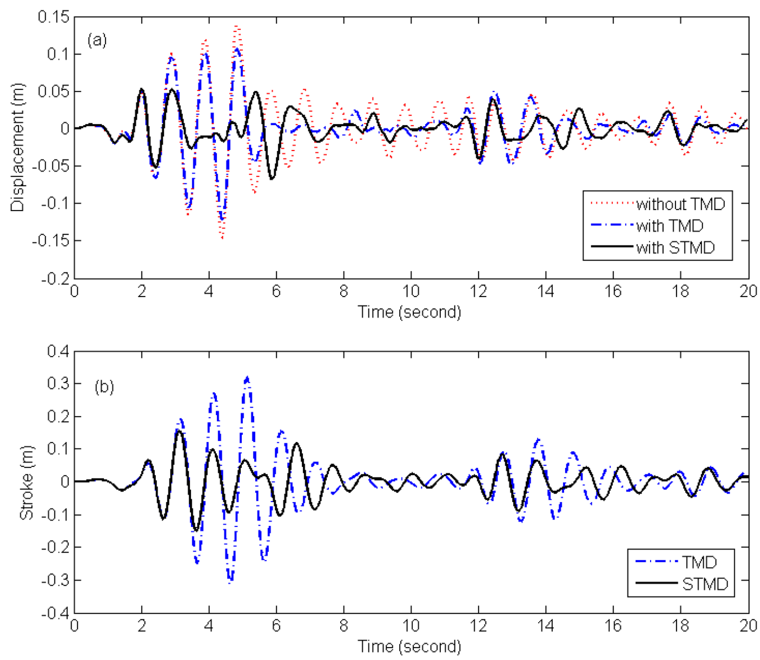


Fig. 8 Time histories of displacement and stroke: (a) Displacement and (b) stroke

31.8 cm and 15.6 cm, which imply that the STMD can provide better control performance although it has about half the stroke of the TMD.

4.6 Inelastic structure

The structural period and damping of most structures excited by design earthquake load change when the structures experience inelastic ductile motion. Both period and damping increase with increasing ductility (ATC-40, 1996), but the TMD is generally designed based on the constant period and damping in elastic behavior range. Therefore, for the seismic application of the TMD, its performance should be verified for inelastic structures with varying structural period and damping.

Fig. 9(a) and (b), respectively, show frequency response of the elastic and inelastic structures. The TMD and STMD having 2% mass ratio were used for controlling structures. The structural parameters for the elastic structure are identical in the previous section, and the inelastic structure has $e = 0.3$ and $\alpha = 0.05$. Fig. 9(a) shows that the resonant response components significantly reduce when the TMD or STMD is applied and the STMD provides better displacement reduction effect than the passive TMD in frequency domain as well as in time domain. Fig. 9(b) shows that installation of the passive TMD reduces resonant response of the inelastic structure but the reduction effect is not as good as the case for the corresponding elastic structures. The inelastic structure has much smaller resonant frequency response than the elastic structure. This is due to the increase of damping resulting by inelastic deformation, and the large structural damping induces performance deterioration of the passive TMD. In comparison, the STMD provides the consistent control performance both for elastic and inelastic structures. Especially, it is noted that the structure with the STMD gives smaller plastic deformation, which is represented by the low frequency response in Fig. 9(b), than the structure without damper and one with the passive TMD. The superior performance of the STMD over the passive TMD is once more ascertained in Fig. 10 showing the force-displacement curve of the inelastic structure since the enclosed area for the structure with the STMD is the smallest.

Numerical analyses were conducted for a mass normalized SDOF system with 1.2 second period and 5% damping ratio by using the El Centro earthquake (1940, NS component) as ground acceleration. $\alpha = 1.0$, 0.5, 0.25, 0.15, 0.05 and $e = 0.7$, 0.5, 0.3 were considered as the parameters for modeling the elastic and inelastic behaviors of the system. $\alpha = 1.0$ denotes the elastic structures,

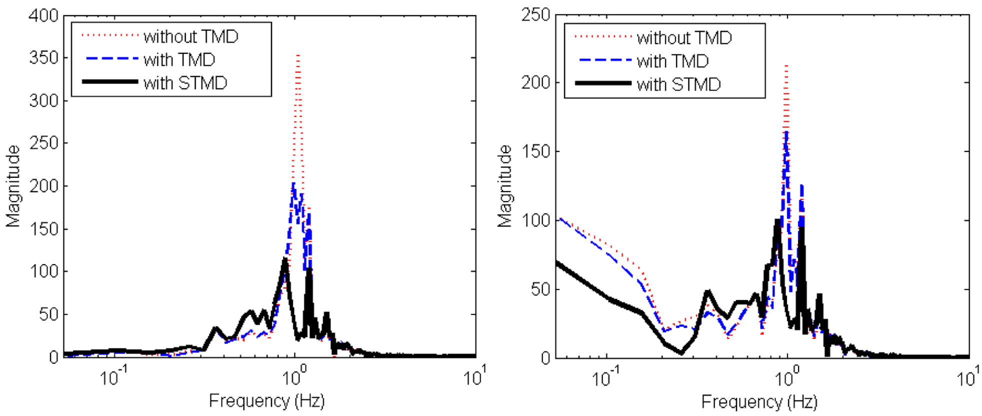


Fig. 9 Frequency response of elastic and inelastic structures: (a) elastic structure and (b) inelastic structure

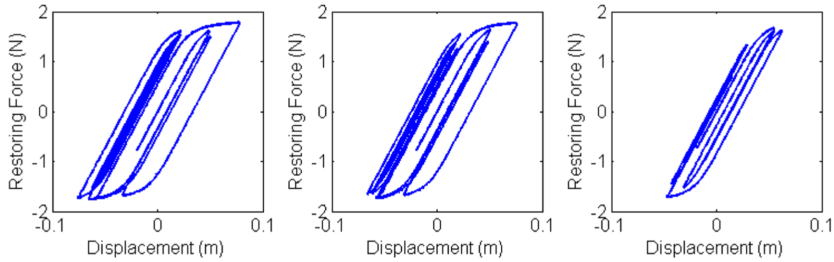


Fig. 10 Force-displacement relationship: (a) uncontrolled, (b) TMD controlled and (c) STMD controlled

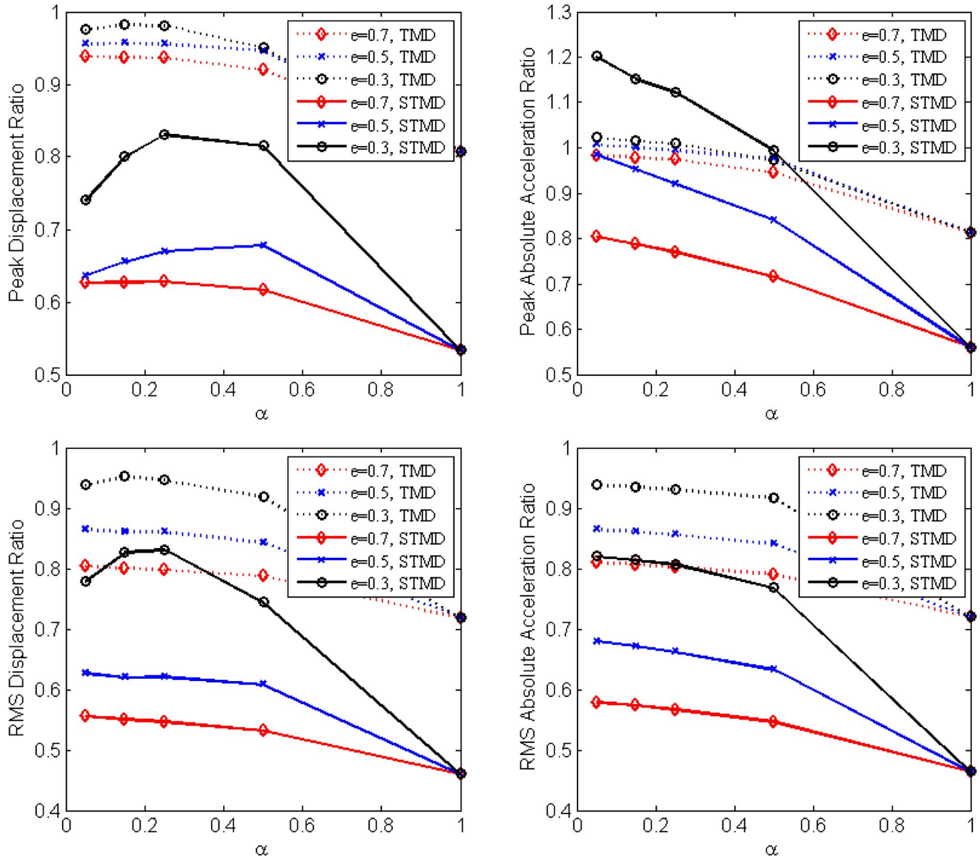


Fig. 11 Performance comparison of TMD and STMD for various inelastic structures: (a) peak displacement, (b) peak acceleration, (c) RMS displacement and (d) RMS acceleration

and the less values of α and e make the inelastic behavior more dominant. Fig. 11 shows the peak and RMS response normalized to those of uncontrolled structures. The Case-1 was considered for designing the STMD. The displacement responses of the structure with the passive TMD increased almost to 1 for the structure experiencing large inelastic motion due to small α and e. This indicates that the TMD was ineffective in response control of structures showing large inelastic behaviors. The STMD reduced all the responses much more than the TMD, except for the absolute acceleration when $e = 0.3$. The performance of the STMD became the worse for the smaller than e, as is the case for the TMD, but the degree of deterioration was not so significant as TMD providing little response reduction effect for the structure with small e. Also, a smaller α did not always induce performance deterioration of the STMD. These facts indicate that unlike the passive TMD, the STMD is comparatively insensitive to the variation of the structural properties, and can be designed to show consistent performance to control inelastic structures under severe earthquake loads.

5. Conclusions

In this study, the control performances of the passive TMD and semi-active TMD (STMD), which can modulate the stiffness and damping by controlling the connectivity of the spring and dampers, were evaluated in terms of the seismic response control of elastic and inelastic structures under seismic loads. Elastic displacement spectra were obtained for the damped structures with a passive TMD, which was optimally designed by using the frequency and damping ratio presented by a previous study, and for the damped structures with a STMD proposed in this study. Also, the robustness of the passive TMD was evaluated by off-tuning the frequency of the TMD to that of the structure. The control performance of the passive TMD significantly deteriorated with increasing structural damping and (+) off-tuning while the STMD provided 40% less response spectra and smaller stroke than the passive TMD. Finally, numerical analyses were conducted for an inelastic structure with hysteresis described by the Bouc-Wen model. The results indicated that the performance of the passive TMD whose design parameters were optimized for an elastic structure considerably deteriorated when the hysteretic portion of the structural responses increased. On the other hand, the STMD showed about 15-40% more response reduction than the passive TMD. Especially, it was identified in the frequency domain that the STMD has better performance than passive TMD in reducing the permanent deformation induced by inelastic behavior.

Acknowledgments

This research was supported by a grant (Code# ’09 R&D A01) from Cutting-edge Urban Development Program funded by Ministry of Land, Transport and Maritime Affairs of Korean government.

References

Amini, F. and Doroudi, R. (2010), “Control of a building complex with Magneto-Rheological dampers and tuned mass damper”, *Struct. Eng. Mech.*, **36**(2).

Applied Technology Council (1996), *Seismic evaluation and retrofit of concrete buildings*, Report ATC-40.

Chopra, A.K. (2001), *DYNAMICS OF STRUCTURES*, 2nd edition, Prentice Hall, New Jersey.

Jiang, G. and Hanagan, L.M. (2006), “Semi-active TMD with piezoelectric friction dampers in floor vibration control”, *Proceedings of the SPIE 6169*, San Diego, March.

Kaynia, A.M., Veneziano, D. and Biggs, J.M. (1981), “Seismic effectiveness of tuned mass dampers”, *J. Struct. division ASCE*, **107**(8), 1465-1484.

Kim, H.S., Kim, S.J. and Lee, D.G. (2006), “Use of semi-active tuned mass dampers for vibration control under various excitations”, *J. Earthq. Eng.*, **10**, 51-62.

Lee, S.H., Min, K.W. and Lee, Y.C. (2005), “Modified sliding mode control using target derivative of Lyapunov function”, *Eng. Struct.*, **27**, 49-59.

Leung, A.Y.T., Zhang, H., Cheng, C.C. and Lee, Y.Y. (2008), "Particle swarm optimization of TMD by nonstationary base excitation during earthquake", *Earthq. Eng. Struct. D.*, **37**(9), 1223-1246.
Petti, L., Giannattasio, G., De Iuliis, M. and Palazzo, B. (2010), "Small scale experimental testing to verify the effectiveness of the base isolation and tuned mass dampers combined control strategy", *Smart Struct. Syst.*, **6**(1), 57-72.
Rana, R. and Soong, T.T. (1998), "Parametric study and simplified design of tuned mass dampers", *Eng. Struct.*, **20**(3), 193-204.
Sadek, F., Mohraz, B., Taylor, A.W. and Chung, R.M. (1997), "A method of estimating the parameters of tuned mass dampers for seismic applications", *Earthq. Eng. Struct. D.*, **26**, 617-635.
Setareh, M. (2001), "Application of semi-active tuned mass dampers to base-excited systems", *Earthq. Eng. Struct. D.*, **30**(3), 449-462.
Soong, T.T. and Dargush, G.F. (1997), *Passive Energy Dissipation Systems in Structural Engineering*, John Wiley& Sons, New York.
Wang, J.F., Lin, C.C. and Lian, C.H. (2009), "Two-stage optimum design of tuned mass dampers with consideration of stroke", *Struct. Health Monit.*, **16**(1), 55-72.
Whittaker, A., Constantinou, M. and Tsopelas, P. (1998), "Displacement estimates for performance-based seismic design", *J, Struct. Eng.- ASCE*, **124**, 905-912.
Yamada, K. and Kobori, T. (1995), "Control algorithm for estimating future responses of active variable stiffness structure", *Earthq. Eng. Struct. D.*, **24**(8), 1085-1099.
Yang, R., Zhou, X. and Liu, X. (2002), "Seismic structural control using semi-active tuned mass dampers", *Earthq. Eng. Eng. Vib.*, **1**(1), 111-118.

CC