

OPTIMUM TUNING PARAMETERS OF TUNED MASS DAMPERS FOR VIBRATION CONTROL OF IRREGULAR HIGHRISE BUILDING STRUCTURES

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Abstract. Tall buildings have become increasingly one-of-a-kind signature structures that are often irregular in plan and elevation with complicated dynamic behavior. Vibration control of irregular highrise building structures using a recently developed tuned mass dampers (TMD), the bidirectional TMD (BTMD), is investigated. A key issue for effective implementation of a TMD is the determination of their tuning parameters. Eight different sets of equations for tuning the parameters of TMDs are investigated using a 5-story building with plan and elevation irregularity, and a 15-story and a 20-story building with plan irregularity subjected to seismic loading. Appropriate equations are recommended for building structures with a fundamental period of vibrations of greater than one second.

Keywords: vibration control, tuned mass damper, highrise building, irregular structure.

Introduction

In recent years clients for tall buildings have increasingly been demanding one-of-a-kind signatures that are often irregular in plan and elevation with complicated dynamic behavior (Wang, Adeli 2014). An innovative approach to mitigate the vibration of structures during dynamic wind and earthquake events is the use of tuned mass dampers (TMDs). For example, Kang *et al.* (2012) evaluate performance of TMD installed on a 39-story, 184.6 m steel building located in Incheon, Korea under typhoon using system identification (Cen *et al.* 2013) and inverse wind load estimation. Amini *et al.* (2013) present optimal vibration control of a 10-storey shear-frame building structure using an active TMD and a wavelet-based (Ghodrati Amiri *et al.* 2012; Xiang, Liang 2012; Lin *et al.* 2012; Acharya *et al.* 2012; Kodogiannis *et al.* 2013) control algorithm and particle swarm optimization (Tao *et al.* 2012; Shafahi, Bagherian 2013; Hsu 2013). Ghaemmaghami *et al.* (2013) study dynamic behavior of annular tuned liquid dampers in tall wind towers. A review of TMDs is presented by Gutierrez Soto and Adeli (2013).

Vibration control of structures can be divided into passive, semi-active, and active control (Ko *et al.* 2012; Hemami *et al.* 2012; Rodriguez-Seda *et al.* 2012; Rigatos 2013; Boutalis *et al.* 2013; El-Khoury, Adeli 2013). Passive dampers have proven to be a good practical option for retrofitting of historic buildings (Branco, Guerreiro 2011; Nawrotzki 2006) and can be implemented on existing buildings with relatively modest rehabilitation. Nawrotzki (2008) points out adding TMDs create a dynamic upgrade in existing bridges and buildings that

increases structural safety and improves serviceability and comfort conditions.

Irregular structures behave in a complex manner during strong ground motions and their vibration control is of particular interest (Lei *et al.* 2012; Nigdeli, Boduroglu 2013). Kim and Adeli (2005a, b) investigate vibration control of irregular highrise building structures under various seismic excitations using a hybrid control system consisting of a passive supplementary damping system and a semi-active tuned liquid column damper (TLCD) system. They use a novel wavelet-based optimal control algorithm (Adeli, Kim 2004; Kim, Adeli 2004) to find the optimum control forces. Simulation results for control of two multistory moment-resisting space steel structures with vertical and plan irregularities show clearly that the hybrid damper-TLCD control system significantly reduces responses of irregular buildings subjected to various earthquake ground motions. Bitaraf *et al.* (2012) present active and semi-active adaptive control for undamaged and damaged building structures under seismic motions. Aldemir *et al.* (2012) propose a simple method to obtain the suboptimal passive damping and stiffness parameters from the optimal control gain matrix. They also propose a new performance index in order to consider the mechanical energy of the structure, control and the seismic energies simultaneously in the minimization procedure. The resulting closed-loop control algorithm does not require the solution of the nonlinear Riccati equation (Saleh, Adeli 1997; Adeli, Saleh 1999). They present examples of multistory frames subjected to seismic excitations.

Almazan *et al.* (2012) study the performance and placement of one or more TMDs in asymmetric buildings

using four different examples including a three-dimensional (3D) 15-story RC structure with plan irregularity subjected to seismic loading with the goal of minimizing the inter-story displacement or drift. They use the Kanai-Tajimi spectrum to model the seismic excitation and place 1 or 2 TMDs on the roof. For asymmetric structures, they conclude that the optimum TMD frequency and location on the roof depend on structural lateral and torsional stiffnesses, eccentricity between centers of mass and rigidity and the frequency content of the ground motion; and the optimum location is *near* the geometric center of the plan. They also conclude: “*if the uncertainty associated to the dynamic parameters of the main structure is ignored, a significant improvement would not be obtained by adding a second TMD*”.

Lu *et al.* (2013) present vibration control of The Shanghai World Financial Center Tower, currently the tallest building in China, with a height of 492 m under wind using two Active Tuned Mass Dampers (AMTD) placed on the 90th floor. AMTDs are employed to mitigate wind vibrations only. During earthquakes actuators are turned off and ATMDs behave like passive TMDs. The authors report a reduction in the wind acceleration response of up to 60% when the wind speed is below the design value.

It was noted by Villaverde (1985) that vibration absorbers yield a desired response subjected to earthquake

loading only if the appropriate parameters of vibration absorber device are selected. Equations for parameters of vibration dampers have evolved over the years. Early equations were based on harmonic excitations acting directly on the mass which is different when earthquake excitation is acted at the base.

Vibration control of irregular highrise building structures using a recently developed TMD, the bidirectional TMD (BTMD), is investigated in this paper. Eight different sets of equations for tuning the parameters of tuned mass dampers (TMDs) are investigated using a 5-story building with plan and elevation irregularity, and a 15-story and a 20-story building with plan irregularity subjected to seismic loading. The BTMD is combination of two cables forming a Y-shape connected to the mass at the middle, and a friction damper connected to the mass from the bottom.

1. Selection of optimum parameters values for a TMD

Different researchers have presented equations for the optimum values of the parameters of the TMD using different criteria or approaches. A summary is provided in Table 1. Most researchers present optimum values of the damping and frequency ratios based on a given mass ratio.

Table 1. Equations for finding the optimum tuning parameters of a TMD

Author	Year of Publication	Frequency ratio γ_{opt}	Damping ratio ξ_{dopt}	Comments
Den Hartog	1956	$\frac{1}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu)^3}}$	First equations for optimum tuning parameters of a TMD and an undamped SDOF system based on a harmonic excitation
Warburton	1982	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\sqrt{\frac{\mu(1-\mu/4)}{4(1+\mu)(1-\mu/2)}}$	Based on random white-noise acceleration excitation and an undamped SDOF system
Fujino and Abe	1993	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\frac{1}{2}\sqrt{\frac{\mu(1+3\mu/4)}{(1+\mu)(1+\mu/2)}}$	Based on random excitations for undamped 2DOF systems
Feng and Mita	1995	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	(i) $\frac{1}{2}\sqrt{(1+\mu)\gamma^4 + \gamma^2 + \frac{1-3(1+\mu)^2\gamma^2}{(1+\mu)^3}}$ (ii) $\frac{\gamma}{2}\sqrt{(1+\mu)\gamma^2 + 1 - \frac{1}{1+\mu}}$	Based on white-noise excitation of 2DOF system for displacement (i) and acceleration (ii)
Sadek <i>et al.</i>	1997	(i) $\frac{1}{1+\mu}$ (ii) $\frac{1}{1+\mu}\left[1 - \xi_s \sqrt{\frac{\mu}{1+\mu}}\right]$	(i) $\sqrt{\frac{\mu}{1+\mu}}$ (ii) $\frac{\xi_s}{1+\mu} + \sqrt{\frac{\mu}{1+\mu}}$	Based on earthquake excitation for both undamped (i) and damped (ii) 3D SDOF systems

Continued Table 1

Author	Year of Publication	Frequency ratio γ_{opt}	Damping ratio ξ_{dopt}	Comments
Rudinger	2006	$\sqrt{\frac{2+\mu}{2(1+\mu)^2}}$	$\sqrt{\frac{\mu(4+3\mu)}{4(1+\mu)^3}}$	Based on white noise excitation on undamped and damped linear and nonlinear SDOF systems with linear damping =1
Krenk and Hogsberg	2008	$\frac{1}{1+\mu}$	$\frac{1}{2} \sqrt{\frac{\mu}{1+\mu}}$	Based on force and white noise acceleration excitation on undamped and damped 2DOF systems
Hoang <i>et al.</i>	2008	(i) $\frac{\sqrt{(1-6\mu)(1+\mu^2)}}{1+\mu} - 0.7\xi_s$ (ii) $\frac{\sqrt{\left(1-\frac{\delta\mu}{6}\right)}}{1+\mu} - \frac{0.7\xi_s}{1-\frac{\mu}{2}}$ (iii) $\frac{\sqrt{(1-\mu/2)}}{1+\mu} - \frac{0.7\xi_s}{1-\mu/2}$	(i) $\frac{\sqrt{\mu(1+2.5\mu+2\mu^2)}}{2(1+2.7\mu)}$ (ii) $\sqrt{\frac{\mu\left(1-\frac{\mu}{4}\right)}{4(1+\mu)\left(1-\frac{\mu}{2}\right)}} + 0.25\mu\xi_s$ (iii) $\sqrt{\frac{\mu(1-\mu/4)}{4(1+\mu)(1-\mu/2)}} + 0.25\mu\xi_s$	Based on seismic excitation of a damped SDOF for ranges of ground frequency ratio of $\delta = \omega_g / \omega_s$ (i) $\delta = 1$ (ii) $1 < \delta < 3$ (iii) $\delta \geq 3$

Den Hartog (1956) derived equations by minimizing the response of an undamped single-degree-of-freedom (SDOF) system subjected to sinusoidal loading. The author showed how damper and spring coefficients can be determined to minimize vibration for a specific structural mode.

Warburton (1982) uses Den Hartog’s approach and *broadband optimization* to derive equations considering harmonic and white noise random excitations directly on a SDOF system based on the notion that the average damping ratios of the resonant modes between the uncontrolled structure and the TMD is approximately equal to the effective damping ratio of a building with TMD.

Villaverde (1985) derived equations for a TMD with high damping and small mass ratios by minimizing the dynamic response of a 2D three-story frame and a 3D 10-story building subjected to seismic loading.

Fujino and Abe (1993) present four sets of equations for optimum tuning parameters of a 2DOF system for four different cases: free vibration, harmonic, self-excited, and random excitations based on a modification of the 2DOF dynamic characterization proposed by Igusa and Der Kiureghian (1985) using the mean-square values of *perturbation solutions* of displacement responses assuming both mass and structural damping ratios of less than two percent. The equations for the case of random excitations are included in Table 1.

Feng and Mita (1995) proposed two sets of equations, one for displacement and the other for acceleration, for the optimum values of frequency and damping ratios of a TMD attached to a 2 DOF system by minimizing the dynamic response in terms of mean square values of

displacement and acceleration. They study the performance of their equations on a 200-m tall building 2D frame with a structural damping ratio of 0.02 subjected to earthquake excitations.

Sadek *et al.* (1997) modified the equations proposed by Villaverde (1985) and use curve fitting to find optimum parameters of the TMD system such that the first two modes of the controlled structure have the same damping ratio with a value equal to greater than the average damping ratio of TMD and structure alone. Their formulation varies from previous authors in that it includes the structural damping ratio of the primary structural system (ξ_s). The authors also investigated the optimum parameters for MDOF using a factor that takes into consideration the amplitude of the first mode shape of the structure.

Rudinger (2006) used statistical linearization to derive TMD optimum tuning parameter equations for a damped SDOF system subjected to white noise excitation. The author concludes TMD optimum tuning parameters are the same for undamped and damped systems that behave linearly. On the contrary, optimal parameters for a nonlinear system with TMD are dependent on structural damping and excitation intensity (vibration amplitude).

Krenk and Hogsberg (2008) use an approximation technique with minimum standard deviation to obtain optimum tuning equations for undamped and damped 2DOF systems subjected to force and acceleration excitations independent of the structural damping ratio. Their approximation suggests that optimal performance can be obtained without considering the structural damping of the primary system in formulation of tuning and frequency ratios.

Hoang *et al.* (2008) present equations for optimum parameters of a TMD for a SDOF system subjected to seismic excitation for different ranges of the ratio of the dominant ground frequency (ω_g) to structural frequency (ω_s), $\delta = \omega_g / \omega_s$, using a nonlinear programming technique (Adeli, Soegiarso 1999; Adeli, Sarma 2006) that follows the Davidon-Fletcher-Powell algorithm (Hoang, Warnitachai 2005). The idea is to have site-specific parameters thus taking into account the type of soil on which a structure is built. In their formulation, the authors simulate artificial earthquakes using the Kanai-Tajimi model and choose a specific ground frequency. Their formulas are separated on a specific range of ground frequency ratio, but in reality the range of ground frequencies in an earthquake cannot be known a priori. The authors presented a numerical example where they studied the effect of large mass ratios for TMD in a bridge subjected to earthquake loading. They noted that the: “optimal TMD has lower tuning frequency and higher damping ratio with increasing mass ratio”. They also concluded that using a large mass ratio can help against parameter uncertainties that occur between theoretical model and the actual practical implementation. In their example, they use a mass ratio of 77% and provided response reduction for their example, yet using such high mass ratio on a building such as a TMD on roof can influence the practicality and constructability of the design.

2. Bidirectional tuned mass damper

Almazan *et al.* (2007) proposed a bidirectional TMD (BTMD) device and applied it to control vibrations of a 25-story reinforced concrete 3D regular frame and an 80-m by 3-m diameter thin-walled cylindrical steel structure subjected to seismic loading. This device is a combination of two cables forming a Y-shape connecting to the mass at the middle, and a friction damper connected to the mass from the bottom (Fig. 1).

The tuning parameters of a BTMD are the mass m_d and the lengths L_x and L_y of the cables from the mass to the fix support and to Y-intersection (Fig. 1) which are determined as follows:

$$L_x = \frac{g}{\gamma_{opt}^2 \omega_{sx}^2}; \quad (1)$$

$$L_y = \frac{g}{\gamma_{opt}^2 \omega_{sy}^2}; \quad (2)$$

$$\tilde{f}_o = \frac{f_o}{m_d g} = \frac{(\xi_{opt} - \xi_o)\pi}{1 - L_x / L_f}, \quad (3)$$

where ω_{sx} and ω_{sy} stand for the fundamental natural frequencies of the structure in the two principal directions considered. These frequencies are determined based on the dominance of the modal mass participation factor. The optimal slip force \tilde{f}_o is obtained based on the equiv-

alent damping ratio, ξ_{eq} (which is set equal to ξ_{opt}). ξ_o is the viscous damping ratio of the friction damper located at the connection and L_f is the remaining distance from the intersection to the floor. The optimum tuning parameters can be the same values used for a standard TMD presented in Table 1.

A mass ratio (μ) within the range of 3–5% has been recommended in the literature (Connor 2003). A mass ratio of $\mu = 0.03$ and structural damping (ξ_s) of 0.05 is recommended for steel structures (ASCE 2010) are used in this paper.

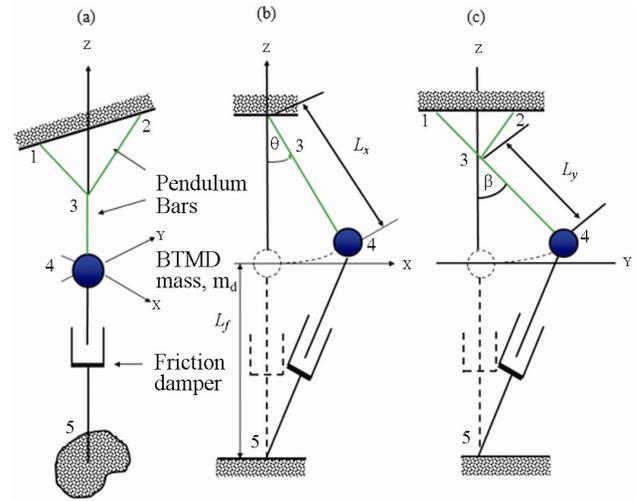


Fig. 1. Bidirectional Tuned Mass Damper (BTMD): (a) Three-dimensional view; (b) view in the xz plane; (c) view in the yz plane

3. Comparison of equations for optimum BTMD tuning parameters

In this section, eight different sets of equations for finding the optimum BTMD tuning parameters summarized in Table 1 are compared using three different irregular structures.

3.1. Example 1: five-story 3D ordinary moment resisting steel frame with plan and elevation irregularity

The 5-story 3D ordinary moment resisting steel frame with setbacks and an L-shape plan is shown in Figures 2 and 3. This example was originally created by Young and Adeli (2014). Roof and floor framing systems consist of steel beams and 6-in thick lightweight concrete over steel metal deck working as a rigid diaphragm. The loads on the structure include dead load from self-weight for steel members and concrete floors, additional dead load for roofing, partitions, furniture and other structure items (15 psf), cladding (25 psf), live load (50 psf) and wind load corresponding to 85 mph, gust factor 0.85 and exposure category C. It is designed for a design acceleration spectrum of 0.28 g, site class D and seismic category B (ASCE 2010). Centers of mass (CM) and centers of rigid-ity (CR) are identified in Figure 3.

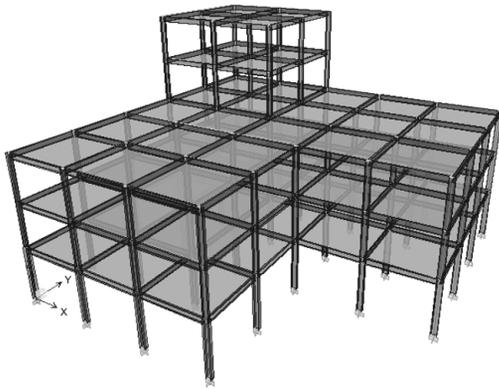


Fig. 2. Five-story irregular 3D building structure with setbacks and an L-shape plan

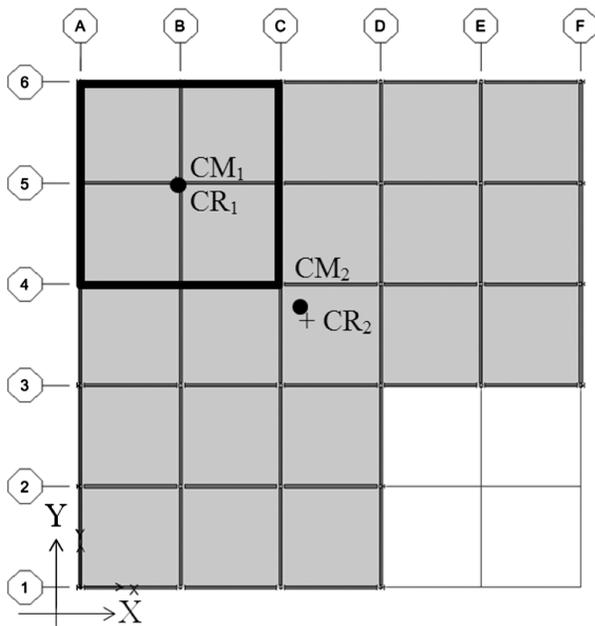


Fig. 3. Plan view of 5-story building (+CR₁ = center of rigidity of the top two floors, +CR₂ = center of rigidity of the bottom three floors, ●CM₁ = center of mass of the top two floors, ●CM₂ = center of mass of the bottom three floors)

Table 2 presents the modal response data for the first 7 modes of the uncontrolled structure. The more irregular the structure, the larger number of modes is needed for accurate determination of the dynamic response of the structure (Liang *et al.* 2012). In this example, the dominant modes, that is, the modes with the largest modal participation factors, are found to be the first two modes (their participation factors are identified in shaded boxes in Table 2).

The most dominant mode turns out to be the second mode of vibrations. As such, the parameters of BTMD are determined for the first two modes of vibrations and used for the comparative study in this paper. A BTMD is placed at the CR of the roof location (B-5 in Fig. 3). The uncontrolled structure is subjected to Loma Prieta, Northern California, earthquake of October 17, 1989, with Magnitude 7.1 (Station 58378) displayed in Figure 4 using twenty four different incident angles with increments of 15 degrees in the horizontal plane. The maximum absolute displacement

Table 2. Free vibration modal response of the 5-story irregular building

Mode	Period (sec)	Modal Participation Factors		
		X Translation	Y Translation	Z Torsion
1	0.807	0.001	0.470	0.036
2	0.553	0.480	0.071	0.770
3	0.502	0.150	0.360	0.021
4	0.444	0.180	0.027	0.003
5	0.279	0.052	0.000	0.016
6	0.240	0.018	0.004	0.027
7	0.183	0.047	0.027	0.023
Total Sum =		0.928	0.959	0.896

was obtained for the incident angle of 75 degrees with respect to the X-axis shown in Figure 3. Next, the incident angle resolution was reduced to 5 degrees around the incident angle of 75 degrees between 60 and 90 degrees. The maximum absolute displacement was obtained for the incident angle of 80 degrees at joint A6-5. Each node is identified with a letter (indicating the grid in the N-S direction) followed by a number (indicating the grid in the E-W direction), a dash, and a second number (indicating the floor number). For example, A6-5 indicates the node at the intersection of grid lines A and 6 on the 5th floor. The maximum absolute joint displacements in cm for different earthquake incident angles are depicted in Figure 5.

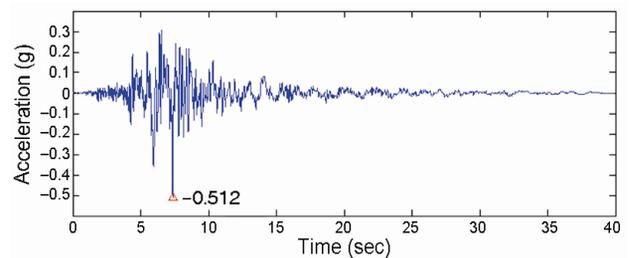


Fig. 4. Loma Prieta earthquake accelerogram (October 17, 1989, Magnitude 7.1)

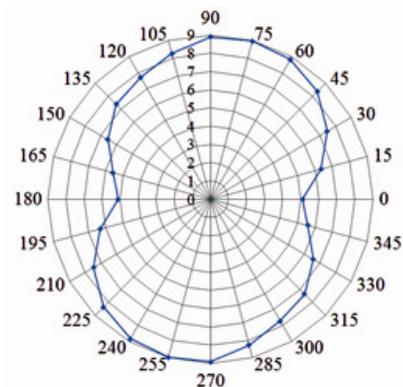


Fig. 5. Maximum absolute joint displacement (cm) for different earthquake incident angles for the 5-story building

The critical angle of incidence of 80 degrees is used to compare the 8 sets of equations for PTMD/BTMD tuning parameters presented in Table 1. The results of the comparisons for maximum absolute displacements, accelerations and base shear are presented in Table 3.

Based on Table 3, the formulas suggested by Hoang *et al.* (2008) using $\delta = 1$ present the most effective control in decreasing the maximum displacement by 41.3%, maximum acceleration by 35.3%, while formulas suggested by Sadek *et al.* (1997) result in the highest decrease in absolute base shear by 25.7%.

Results for the maximum inter-story drifts for the eight sets of equations are presented in Figure 6 and for the maximum floor accelerations are presented in Figure 7. Figure 8 shows the maximum displacement and acceleration of joint A6-5 and base shear of the 5-story building with a BTMD system. Equations proposed by Hoang *et al.* (2008) provide the most effective inter-story displacement control. Equations proposed by Fujino and Abe (1993), Feng and Mita (1995), Sadek *et al.* (1997), Krenk and Hogsberg (2008) and Hoang *et al.* (2008) yield similar maximum floor acceleration results and are more effective than the rest of the equations.

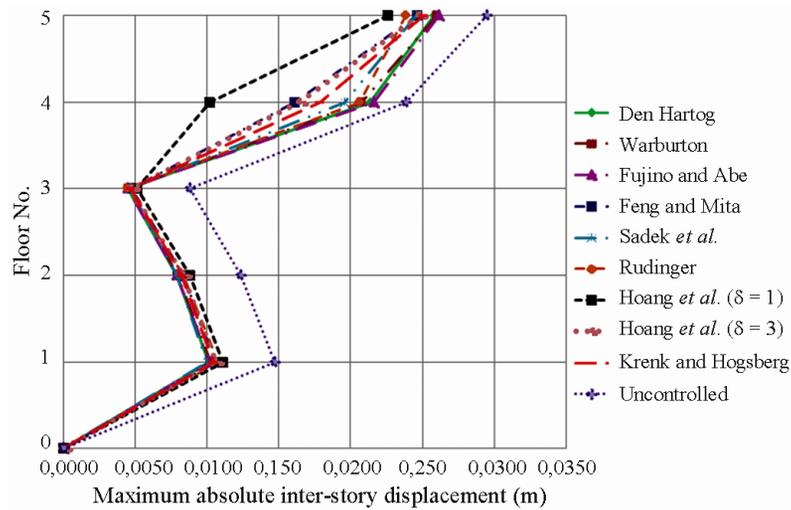


Fig. 6. Maximum interstory displacement (drift) for 5-story building using different equations for optimum TMD tuning parameters

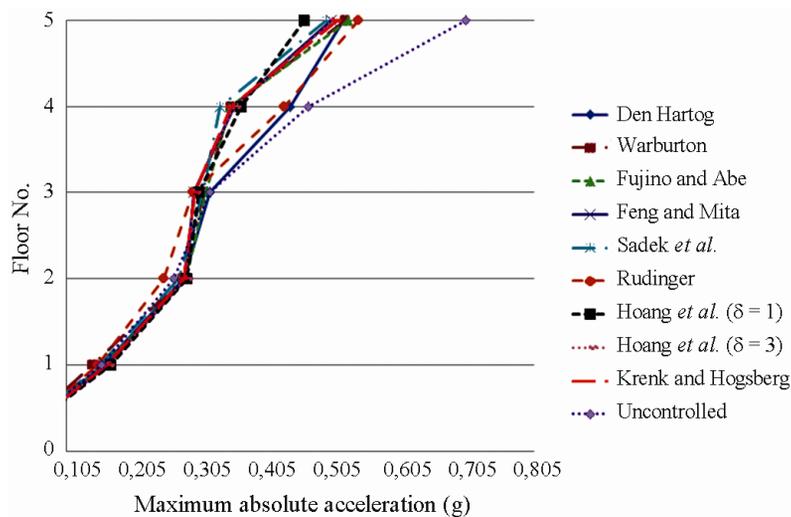


Fig. 7. Maximum absolute floor acceleration for 5-story building using different sets of equations for optimum TMD tuning parameters

Table 3. Optimum tuning parameters and maximum absolute displacement (cm) and acceleration (g) and base shear (kN) for the 5-story building

Method	Optimal Parameters		Maximum Response of the Controlled Structure		
	Frequency ratio γ_{opt}	Damping ratio ξ_{dopt}	Absolute Displacement (cm)	Absolute Acceleration (m/s ²)	Absolute Base Shear (kN)
Hoang <i>et al.</i> (2008) ($\delta = 1$)	0.858	0.083	5.44	0.460	802.7
Sadek <i>et al.</i> (1997)	0.963	0.219	6.37	0.494	720.2
Feng and Mita (1995)	0.964	0.083	6.44	0.502	764.0
Hoang <i>et al.</i> (2008) ($\delta = 3$)	0.928	0.086	6.49	0.504	763.9
Krenk and Hogsberg (2008)	0.971	0.086	6.67	0.511	754.5
Warburton (1982)	0.964	0.0860	6.95	0.523	745.4
Den Hartog (1956)	0.971	0.101	6.96	0.520	764.2
Fujino and Abe (1993)	0.964	0.0857	7.03	0.527	743.2
Rudinger (2006)	0.971	0.005	7.82	0.583	788.9
Uncontrolled	–	–	9.27	0.710	969.6
% Max. Reduction			41.3	35.3	25.7

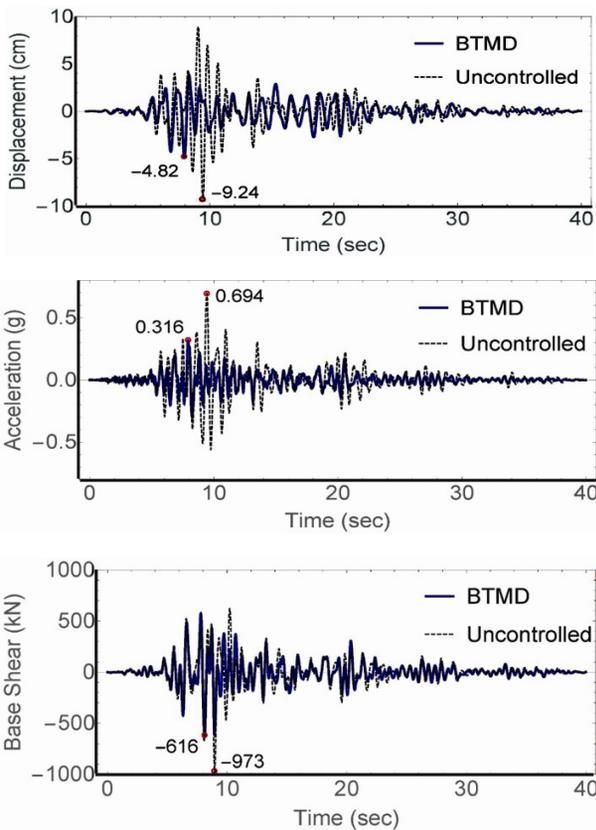


Fig. 8. Maximum displacement and acceleration of joint A6-5 and base shear of 5-story building with a BTMD system

3.2. Example 2: fifteen-story reinforced concrete moment resisting frame with plan irregularity

This example is a 15-story irregular reinforced concrete moment-resisting frame with an L-shape plan shown in Figure 9. This building was created originally by Almazan *et al.* (2012). The beams have a 0.3 m by 0.7 m cross-

sectional area. All columns have a square 0.5 m by 0.5 m cross-sectional area, except columns along grid line 4 which have a 1 m by 0.4 m cross-sectional area and grid location C-3 which have a 0.4 m by 1 m cross-sectional area. The slab is 0.15 m thick and is modeled as a rigid diaphragm.

Table 4 presents the free vibrations response data for the uncontrolled structure. This structure is subjected to the same Loma Prieta earthquake accelerogram used in the previous 5-story example (Fig. 4) and studied using 24 different incident angles with increments of 15 degrees in the horizontal plane. The maximum absolute displacement was obtained for the incident angle of 45 degrees with respect to the X-axis (Fig. 9). Next, the incident angle resolution was reduced to 5 degrees around the incident angle of 45 degrees between 45 and 60 degrees.

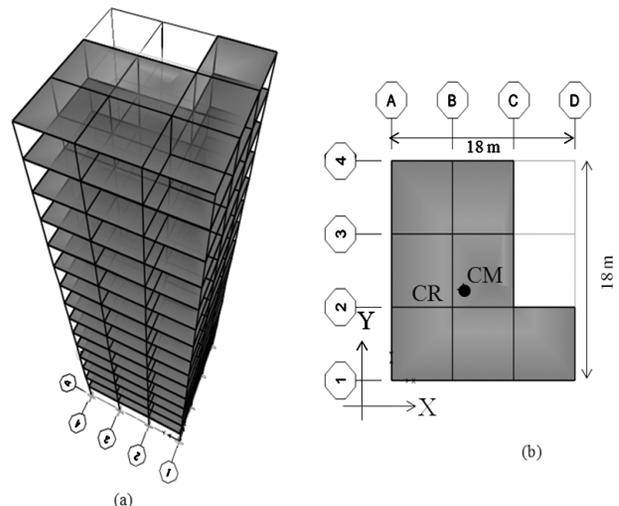


Fig. 9. 15-story building reinforced concrete 3D building with plan irregularity: (a) Perspective view; (b) Plan view; +CR = center of rigidity; ●CM = center of mass

Table 4. Free vibration modal response of the 15-story irregular building

Mode	Period (sec)	Modal Mass Participation Factors		
		X Translation	Y Translation	Z Torsion
1	2.933	0.006	0.781	0.187
2	2.875	0.774	0.006	0.296
3	2.317	0.001	0.000	0.289
4	0.946	0.000	0.099	0.028
5	0.918	0.099	0.000	0.029
6	0.739	0.002	0.000	0.044
7	0.533	0.000	0.037	0.013
8	0.513	0.036	0.000	0.007
9	0.407	0.002	0.000	0.021
Total Sum =		0.921	0.925	0.915

The maximum absolute displacement was obtained for the incident angle of 50 degrees at joint D1-15. The maximum absolute joint displacements in cm for different earthquake incident angles are also plotted in Figure 10. This angle of incidence is used to compare the eight sets of equations for TMD tuning parameters presented in Table 1. The results of the comparisons for maximum absolute displacements, accelerations and base shear as well as detailed values of the optimum tuning parameters for the eight sets of equations are presented in Table 5.

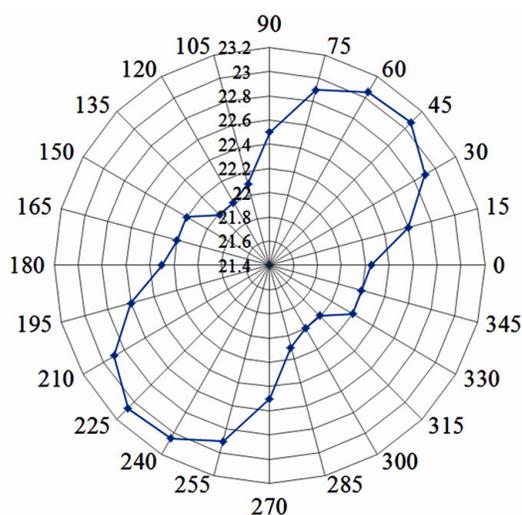


Fig. 10. Maximum absolute displacement (cm) per earthquake incident angles of 15-story building

Equations proposed by Sadek *et al.* (1997) are the most effective in reducing the structural response, shaded in Table 5. Equations proposed by Hoang *et al.* (2008) and Krenk and Hogsberg (2008) yield the next best results. Equations presented by Sadek *et al.* (1997) decrease the maximum absolute displacement by 49.76%, maximum acceleration by 32.14%, and absolute base shear (kN) by 42.22%.

3.3. Example 3: twenty-story steel moment resisting frame with plan irregularity

This example is a 20-story steel moment resisting frame structure with plan irregularity created originally by Liew *et al.* (2001) and used by Jiang *et al.* (2002) for plastic analysis of steel 3D buildings. This example is also used by Jiang and Adeli (2005) and Adeli and Jiang (2009) in their dynamic wavelet neural network model for structural system identification, Van Long and Hung (2008) for local buckling determination during plastic analysis, and Chiorean (2009) for development of a computer program for nonlinear inelastic analysis.

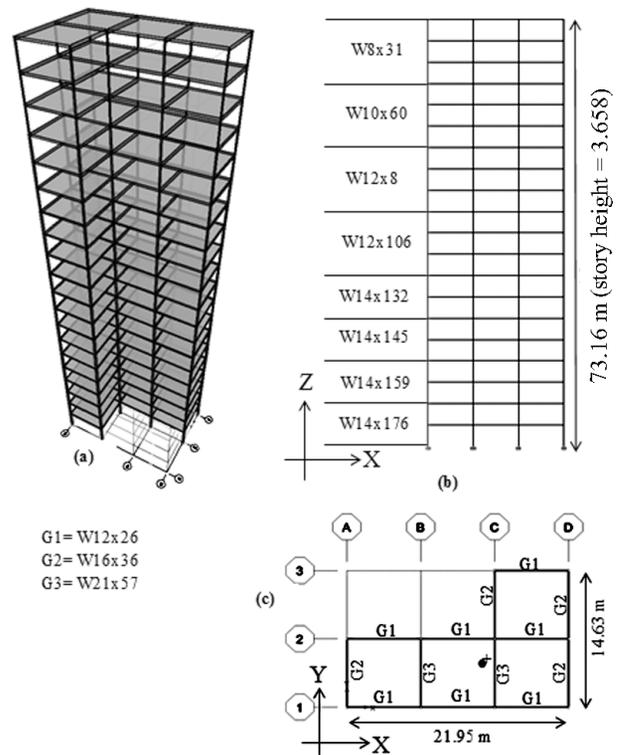


Fig. 11. 20-story building with plan irregularity: (a) Perspective view; (b) Elevation; and (c) Plan view; +CR = center of rigidity; ●CM = center of mass

The perspective, elevation and plan view of this building are shown in Figure 11. Roof and floor framing systems consist of steel beams and lightweight concrete with a slab thickness of 0.15 m over steel metal deck working as a rigid diaphragm (Fig. 11c). The building is designed for a combination of 4.8 kN/m² static gravity load and 0.96 kN/m² wind load acting on the y-direction. Table 6 presents the free vibrations response data for the uncontrolled structure including the modal mass participation factors. This structure is also subjected to the same Loma Prieta earthquake accelerogram displayed in Figure 4. The maximum displacement was found at joint D3-20 on the roof. The maximum absolute joint displacements in cm for different earthquake incident angles are also plotted in Figure 12. The critical angle of earthquake incidence for the uncontrolled structure was determined to be 5 degrees with respect to the X axis (Fig. 11c). This angle of incidence is used to compare the eight sets of

equations for TMD tuning parameters presented in Table 1. The results of the comparisons for maximum absolute displacements, accelerations and base shear as well as detailed values of the optimum tuning parameters for the eight sets of equations are included in Table 7.

Similar to Example 2, equations proposed by Sadek *et al.* (1997) yield the most effective vibration reductions for the 20-story irregular structure; they decrease the maximum absolute displacement by 43.4%, maximum acceleration by 41.8%, and absolute base shear (kN) by 25.3%.

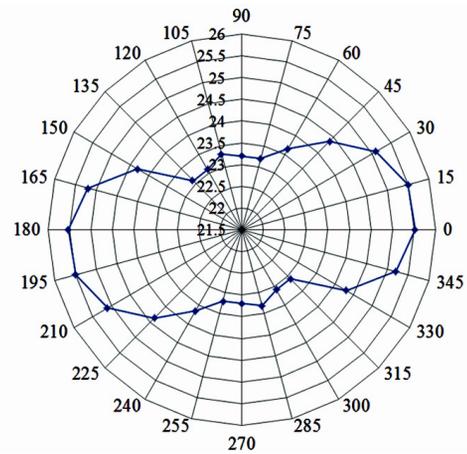


Fig. 12. Maximum absolute joint displacement (cm) per earthquake incident angles for the 20-story building

Table 5. Optimum tuning parameters and maximum absolute displacement (cm), acceleration (g) and base shear (kN) response for the 15-story building

Method	Optimal Parameters		Maximum Response of the Controlled Structure		
	Frequency ratio γ_{opt}	Damping ratio ξ_{dopt}	Absolute Displacement (cm)	Absolute Acceleration (g)	Absolute Base Shear (kN)
Den Hartog (1956)	0.971	0.101	14.7	0.128	1011
Warburton (1982)	0.964	0.086	13.5	0.128	922.6
Fujino and Abe (1993)	0.964	0.086	13.5	0.128	922.6
Feng and Mita (1995)	0.964	0.083	13.56	0.129	925.5
Sadek <i>et al.</i> (1997)	0.963	0.219	11.31	0.124	792.9
Rudinger (2006)	0.971	0.005	14.89	0.128	1003
Hoang <i>et al.</i> (2008) ($\delta=1$)	0.858	0.083	14.09	0.124	1022
Hoang <i>et al.</i> (2008) ($\delta=3$)	0.928	0.086	13.68	0.130	930.3
Krenk and Hogsberg (2008)	0.971	0.085	13.58	0.129	920.0
Uncontrolled	–	–	22.52	0.228	1372.2

Table 6. Free vibration modal response of the 20-story irregular building

Mode	Period (sec)	Modal Participation Factors		
		X Translation	Y Translation	Z Torsion
1	4.666	0.756	0.001	0.121
2	4.256	0.001	0.718	0.444
3	3.659	0.002	0.000	0.166
4	1.707	0.000	0.127	0.088
5	1.647	0.101	0.000	0.010
6	1.396	0.000	0.000	0.021
7	1.043	0.000	0.054	0.037
8	0.981	0.040	0.000	0.005
9	0.853	0.000	0.000	0.009
10	0.728	0.000	0.025	0.017
11	0.679	0.023	0.000	0.003
Total Sum =		0.924	0.926	0.926

Table 7. Optimum tuning parameters and maximum absolute displacement (cm), acceleration (g) and Base Shear (kN) response for the 20-story building

Method	Optimal Parameters		Maximum Response of the Controlled Structure		
	Frequency ratio γ_{opt}	Damping ratio ξ_{dopt}	Absolute Displacement (cm)	Absolute Acceleration (g)	Absolute Base Shear (kN)
Den Hartog (1956)	0.971	0.101	18.77	0.188	378.2
Warburton (1982)	0.964	0.086	16.11	0.188	378.1
Fujino and Abe (1993)	0.964	0.086	16.11	0.188	378.1
Feng and Mita (1995)	0.964	0.083	16.15	0.177	378.5
Sadek <i>et al.</i> (1997)	0.963	0.219	14.45	0.167	349.5
Rudinger (2006)	0.971	0.005	18.31	0.179	395.9
Hoang <i>et al.</i> (2008) ($\delta = 1$)	0.858	0.083	17.55	0.179	393.8
Hoang <i>et al.</i> (2008) ($\delta = 3$)	0.928	0.086	17.25	0.179	389.5
Krenk and Hogsberg (2008)	0.971	0.085	16.10	0.187	377.8
<i>Uncontrolled</i>	–	–	25.53	0.287	466.2

Concluding remarks

An innovation of this research is vibration control of *irregular three-dimensional highrise building structures* using the bidirectional TMD (BTMD). A key issue for effective implementation of the BTMD is the determination of their tuning parameters. The second novelty of this research is an investigation of eight different sets of equations for tuning the parameters of TMDs for highrise building structures with both plan and elevation irregularities. To the best of the authors' knowledge such a comparative study has not been reported in the literature previously.

Equations proposed by Hoang *et al.* (2008) yield the best result for the 5-story irregular structure and those proposed by Sadek *et al.* (1997) yield the best results for the 15- and the 20- story irregular buildings. In Tables 3, 5, and 7 it is observed that Sadek *et al.* (1997) yields a relatively large value for the optimum damping ratio, 0.219, which shows the impact of this ratio in the TMD tuning equations for flexible structures with a fundamental period of vibrations of, say, greater than 1 second ($T > 1$ sec).

By examining the displacement, acceleration and base shear results, it is observed the performance of the BTMD in reducing the vibration responses is affected by the rigidity of the structure; it is more effective for taller and more flexible structures. Sadek *et al.* (1997) equations appear to be more suitable for structures with a fundamental period of vibrations greater than one second.

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