DESIGN CRITERIA OF TUNED MASS DAMPER SYSTEMS TO CONTROL VIBRATIONS OF BUILDING STRUCTURES

CRISTIAN PASTIA, SEPTIMIU-GEORGE LUCA, FIDELIU PAULET-CRAINICEANU
IONUT-OVIDIU TOMA, GEORGE TARANU

"Gheorghe Asachi" Technical University of Iasi (Romania), Faculty of Civil Engineering and Building Services, Dimitrie Mangeron Blvd, pastiacristian@yahoo.com

This paper investigates the effectiveness of a passive Tuned Mass Damper (TMD) attached to a three story building in reducing the response of the structure to harmonic and seismic excitations. Some examples of existing building structures that contain tuned mass dampers are briefly described. Generally, inertial mass is attached near the top, through springs and viscous damping mechanisms. The frequency of the TMD is normally tuned to a particular frequency of the structure so that the two peaks of the frequency response curve of the damped system have the same dynamic amplification, when expressed in terms of displacements. Design charts and equations to determine the optimum values of mass, damping, and stiffness for a passive TMD are illustrated. Numerical simulations have been performed to assess the optimum TMD efficiency in reduction of the seismic and harmonic response of the structure. In addition, this paper shows that a TMD is more effective to mitigate the vibrations induced by harmonic loads than earthquakes.

Keywords: tuned mass damper, vibration, seismic excitation

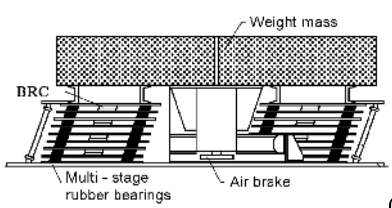
INTRODUCTION

A tuned mass damper (TMD) might be an efficient passive vibration suppression device consisting of a mass, springs and damping mechanisms (e.g. fluid damper) that is connected to a building in order to reduce the dynamic response of the building structure subjected to wind or earthquake loads. A fluid damper is normally incorporated into the TMD to allow the TMD's motion to quickly decay when the vibration input stops. Reduction of vibrations is accomplished by transferring some of the structural vibration energy to the TMD and dissipating the energy by the inertia force of the TMD acting on the structure. Usually, the frequency of TMD is tuned to one of the dominant frequencies of the structure.

The first structure in which a TMD was installed appears to be Certerpoint Tower in Sydney, Australia (Housner *et al.*, 1997). Also, the first major buildings using a TMD, in the USA were the John Hancock Tower in Boston, completed in 1975, the Citicorp Center in New York, completed in 1977 (Housner *et al.*, 1997). The Citicorp building is 279m high and has a fundamental period of around 6.5s with the viscous damping ratio of 1% along each axis. The TMD is installed on the 59th floor in the crown of the structure and has a mass about 2% of the effective modal mass that corresponds to the first mode. The TMD is designed to work in biaxially direction with a variable operating period of 6.25±20%, adjustable linear damping from 8% to 14%, and a peak relative displacement of ±1.4m. The damper is expected to reduce the building response about 50% to wind loads. An equivalent of around 4% damping ratio for the fundamental modes of the structure is the estimated TMD's performance. The concrete mass block (400 tons) is about 2.6m high with a plan cross section of 9.1m by 9.1m and is supported on a series of twelve hydraulic pressure-balanced bearings. During operation, the bearings are supplied oil from a separate hydraulic pump, which is capable of raising the mass block about 2cm to its operating position. The damper system is activated automatically whenever the horizontal acceleration exceeds 0.003g for two consecutive cycles and will automatically shut itself down when the building acceleration does not exceed 0.00075g in either axis over a 30 minute interval (Conner, 2003).

In Japan, the first TMD was installed in the Chiba Port Tower, completed in 1988, followed by other installations. Chiba Port Tower is a steel structure with 125m high and 1950 tons weight. The first and second mode periods for the X direction are 2.25s and 0.51s and for Y direction are 2.7s and 0.57s. The damping ratio for fundamental modes is estimated at 0.50%. For the TMD, the dynamic characteristics of are: the period in X direction is 2.24s; the period in Y direction is 2.72s; the damping ratio is 15%. The maximum relative displacement of the TMD is ±1m in each direction. Reductions in peak displacements and peak bending moments of the structure to wind loads are expected around 30% ÷40% (Conner, 2003).

Newer versions of TMDs employ multi-level elastomeric rubber bearings, which function as shear springs, but which provide viscoelastic damping capability. The device do not requires sophisticated controls, is multidirectional, and is easily assembled and modified (Conner, 2003), as in Figure 1(a). Figure 1(b) shows an actual installation in Huis Ten Boch Tower, Japan (Nagasaki Prefecture).



(a)

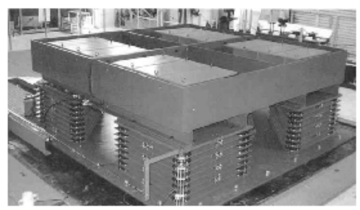

(b)

Figure 1. Tuned mass damper: (a) a simple scheme; (b) installed in Huis Ten Boch Tower, Japan (Conner, 2003)

Also, the TMDs and viscous dampers were used in London Millennium Bridge in order to reduce vertical and horizontal vibrations due to pedestrian induced forces. This reduction corresponds to increasing the basic damping ratio of the structure at 20% for assumed loading. 8 TMDs were used to provide secondarily additional damping in horizontal direction and a total of 26 pairs of TMDs were installed to supplement primarily the damping in vertical direction. Horizontal and vertical damping is provided by 37 viscous dampers, of 7 different types (Taylor, 2002). The vertical TMDs are located on top of the transverse arms beneath the deck. They are situated along the length so that they are approximately at the antinodes of the vertical modes that they are damping (Dallard *et al.*, 2001). Two of the dampers are contained in Figure 2.

Figure 2. Tuned mass dampers beneath the deck of the London Millennium Bridge

DESIGN ELEMENTS OF A TMD

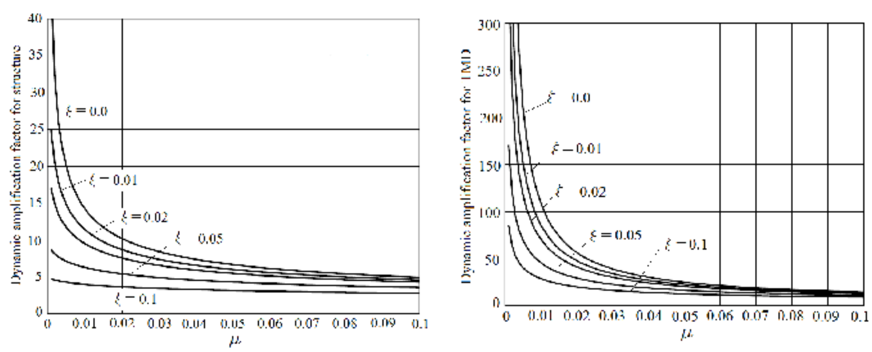
The first theoretical investigation of TMD design was carried out by Ormondroyd and Den Hartog in 1928 and detailed discussions of optimal tuning, damping parameters and design curves derived from the dynamic equations of motion are available (Conner, 2003; Hartog, 1947; Heinemeyer *et al.*, 2009; Pastia and Luca, 2013). A TMD can be very effective if it is precisely tuned on the resonance frequency, which we want to reduce it. The mass ratio, μ, between the TMD's mass and one of the dominant structural modal mass should be chosen typically between 1/100 and 1/10. Figure 3 represents graphs of the effectiveness of a TMD at various mass ratios typically found cost-effective for structures.

A design procedure of a TMD follows the next steps:

- Establish the desired responses of the structure and the TMD for design loads. Choice TMD's mass, m_d, and determination mass ratio, μ, see Figures 3(a) and 3(b).
- Determine from Figure 3(c) the optimum tuning frequency ratio, r_{opt}, expressed as ratio between optimal TMD's frequency, $f_{opt,d}$, and dominant structural frequency.
- Calculation of the TMD's spring constant k_d.
- Determine from Figure 3(d) the optimal damping ratio of the TMD, $\xi_{opt,d}$.
- Calculation of the TMD's damping constant c_d.
- Determine from Figure 3(e) the performance of a TMD which is usually expressed as an equivalent viscous damping ratio, ξ_e.

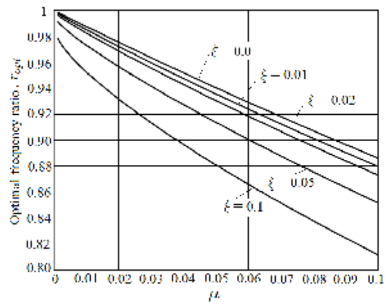
Simpler, if one does not want to use the design curves, the classical formula for optimal tuning parameters of a TMD as function of mass ratio, μ, is given in literature as:

$$f_{opt,d} = \frac{f}{1+\mu} \text{ and } \xi_{opt,d} = \sqrt{\frac{3\mu}{8(1+\mu)^3}} \quad (1)$$

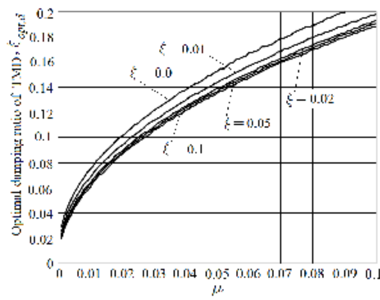


(a) (b)

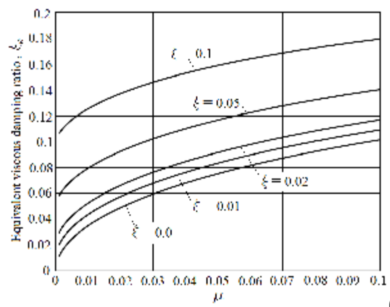
Figure 3. Effectiveness of a TMD (after Conner, 2003)



(c)



(d)



(e)

Figure 4. Effectiveness of a TMD (after Conner, 2003)

CASE STUDIES

The models for the numerical analyses are three story shear buildings with a tuned mass damper installed at the third floor. The governing equations of lumped mass structure as those of the models can be written as:

$$\begin{cases} M\ddot{X} + C\dot{X} + KX = \begin{bmatrix} 0 \\ 0 \\ c_d\dot{x}_d + k_d x_d \end{bmatrix} + \begin{bmatrix} -m_1\ddot{x}_g + P_1 \\ -m_2\ddot{x}_g + P_2 \\ -m_3\ddot{x}_g + P_3 \end{bmatrix} \\ m_d\ddot{x}_d + c_d\dot{x}_d + k_d x_d = -m_d(\ddot{x}_g + \ddot{x}_3) \end{cases} \tag{2}$$

where: $\ddot{x}_g$ represents the horizontal components of a recorded ground acceleration, $\{P\}$ is a vector containing the horizontal harmonic forces and mass, damping and stiffness matrices (M, C, K) are as follow (Olteanu *et al.*, 2011):

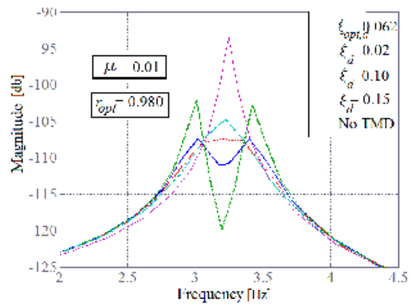
- Case (**A**) where f_1=3.247Hz, T_1=0.308s, damping ratio ξ = 0.01,

$$M = \begin{bmatrix} 7 & 0 & 0 \\ 0 & 7 & 0 \\ 0 & 0 & 7 \end{bmatrix}[t],\ K = \begin{bmatrix} 3.1 & -1.55 & 0 \\ -1.55 & 2.6 & -1.05 \\ 0 & -1.05 & 1.05 \end{bmatrix} \cdot 10^4 \left[\frac{kN}{m}\right],\ C = \begin{bmatrix} 30.8 & -15.4 & 0 \\ -15.4 & 25.3 & -9.9 \\ 0 & -9.9 & 9.9 \end{bmatrix}\left[\frac{kNs}{m}\right].$$

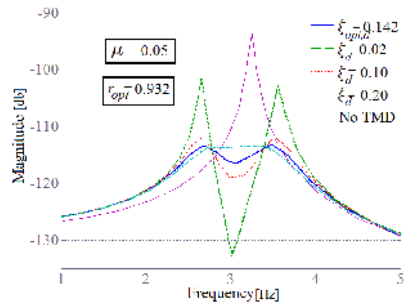
- Case (**B**) where f_1=0.230Hz, T_1=4.355s, damping ratio ξ = 0.01,

$$M = \begin{bmatrix} 70 & 0 & 0 \\ 0 & 70 & 0 \\ 0 & 0 & 70 \end{bmatrix}[t],\ K = \begin{bmatrix} 15.5 & -7.75 & 0 \\ -1.55 & 13.0 & -5.25 \\ 0 & -5.25 & 5.25 \end{bmatrix} \cdot 10^2 \left[\frac{kN}{m}\right],\ C = \begin{bmatrix} 22.4 & -11.2 & 0 \\ -11.2 & 18.4 & -7.2 \\ 0 & -7.2 & 7.2 \end{bmatrix}\left[\frac{kNs}{m}\right].$$

The magnitude curves of the transfer function between the excitation input and the system output (displacement of third floor, case **A**) for some values of TMD damping ratio and mass ratio equal to 0.01 and 0.05 are showed in Figure 4. It is observed that outside of the frequency range, about $\pm 0.15 f_1$, centered on the first natural period of the structural model, the response is not significantly influenced by the TMD.



(a)



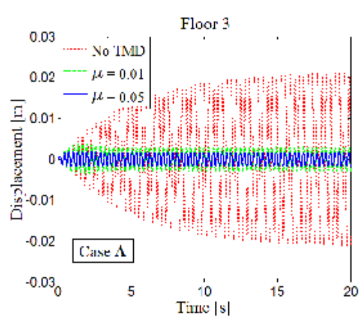
(b)

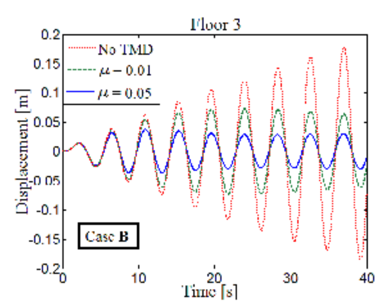
Figure 5. Response curves for magnitude of structural model (case A)

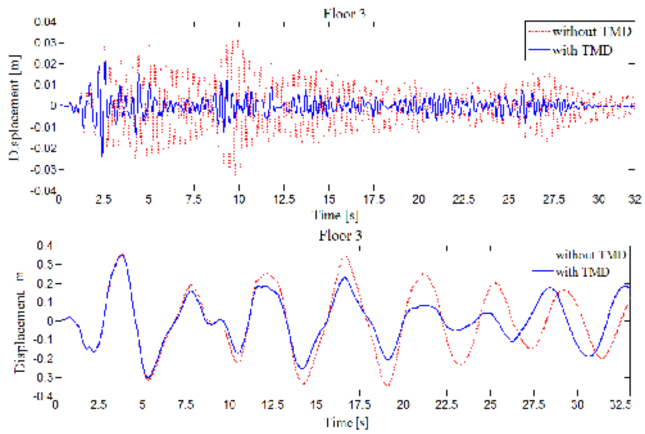
The optimal parameters for the TMD were considered and several simulations have been performed using as input El Centro acceleration and harmonic excitations. The harmonic forces are used at the frequency f =3.247Hz for case (**A**) and f =0.230Hz for case (**B**), with the force amplitude equal to 1000N. Table 1 and Figures 4 and 5 show the comparisons among the responses of the structural models in the uncontrolled case and controlled one with TMD.

Table 1. Peak displacements and peak accelerations of the third floor

Excitation	Case (**A**)	d3 (cm)	a3 (m/s^2)	Case (**B**)	d3 (cm)	a3 (m/s^2)
seism	Without TMD	3.21	14.38	Without TMD	35. 24	3.518
seism	TMD (μ = 0.01)	2.70	12.84	TMD (μ = 0.01)	34. 77	3.516
seism	TMD (μ = 0.05)	2.07	11.72	TMD (μ = 0.05)	33. 21	3.504







Case (**A**), $\mu = 0.05$, $T_1 = 0.308$s

Case (**B**), $\mu = 0.05$, $T_1 = 4.355$s

Figure 6. Displacement responses of structural models under El Centro earthquake

CONCLUSION

In this paper the effectiveness of the TMD using the proposed tuned parameters has been investigated through numerical analyses. Significant reduction in the responses of the structures under harmonic loads is observed. The results of the responses of the structural model with high fundamental period show that the performance of TMD is ineffective for seismic excitation versus harmonic excitation. The disadvantages of the TMD are the very narrow band of suppression frequency and the sensitivity problem due to detuning. The advantage of TMD systems is that they are relatively simple, inexpensive and reliable in suppressing the undesired vibrations of structural systems under assumed loads.

REFERENCES

Dallard, P., Fitzpatrick, A.J., Flint, A., Le Bourva, S., Low, A., Ridsdill Smith, R.M., and Willford, M. (2001), "The London Millennium Footbridge", *The Structural Engineer*, 79(22), 17-33.

Hartog, J.P.D. (1947), *Mechanical Vibrations*, McGraw-Hill, New York and London.

Heinemeyer, C., Butz, C., Keil, A., Schlaich, M., Goldack, A., Trometer S., Lukić, M., Chabrolin, B., Lemaire, A., Martin, P.O., Cunha, Á. and Caetano, E. (2009), "Design of lightweight footbridges for human induced vibrations", *JRC scientific and technical reports*, EUR 23984 EN, European Commission, Luxembourg, ISSN 1018-5593, ISBN 978-92-79-13387-9, 88.

Housner, G.W, Bergam, L.A, Cauchy, T.K., Chassiakos, A.G., Claus, R.O., Skelton, S.M. and R.E., Soong, T.T., Spencer, B.F., and Yao, J.P.T. (1997), "Structural control: Past, present and future control", *Journal of Engineering Mechanics (ASCE)*, 123(9), 897-971.

Olteanu, I., Alistar, A., Budescu, M. (2011), "Nonlinear Analysis of Reinforced Concrete Frames in Atena 3D", *Bul. of the Polytech. Inst. of Jassy*, Romania, Tomme: LVII, Fasc. 2, ISSN: 1224-3884, 93-103.

Pastia, C. and Luca, S.G. (2013), "Vibration Control of A Frame Structure using Semi-Active Tuned Mass Damper", *Bulletin of the Polytechnic Institute of Jassy*, Romania, Tomme: LIX (LXIII), fasc. 4, ISSN 1224-3884, 31-40.

Taylor, D.P. (2002), "Damper retrofit of the London Millennium Footbridge – A Case Study in Biodynamic Design", *Proceedings of the 73d Shock and Vibration Symposium*, Newport, USA, 10.