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MEASURING EFFECTIVE DAMPING OF FULL-SCALE STRUCTURE-TMD AND TSD SYSTEMS

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Abstract: Modern tall buildings are often slender, lightweight, and possess low inherent damping, which can lead to excessive wind-induced motion. Dynamic vibration absorbers (DVAs) in the form of tuned mass dampers (TMDs) and tuned sloshing dampers (TSDs) are being commonly employed to increase the effective damping of structures, thereby reducing their resonant responses under wind excitation. While the performance of structure-DVA systems has been studied extensively theoretically as well as experimentally (at scale-model) in the laboratory, very few studies have reported on the as-built performance of real-world implementations. The performance of a DVA is typically quantified using the concept of "effective damping". Until recently, it has been difficult to measure the effective damping of a structure, since the coupled structure-DVA system is excited through unknown, ambient excitation. In this study, a method is presented that enables the inherent structural damping (that is, the damping of the structure without the DVA), as well as the added effective damping (the damping that the DVA appears to add to the structure) to be estimated from full-scale ambient measurements of a structure-DVA system. The method requires the DVA and the structural generalized masses to be known and the structural and DVA responses to be measured. With this information, the motion reduction achieved by the implementation of the DVA is estimated. After the method is briefly presented, the remainder of the paper focuses on real life applications where the efficacy of DVAs implemented in full-scale tall buildings is presented.

1 INTRODUCTION

Modern tall buildings are often susceptible to excessive wind-induced dynamic motion during common wind events due to their slenderness, lightweight construction, and low inherent damping. This excessive motion can cause occupant discomfort on the upper floors of the building due to high accelerations, or decrease the longevity of the partition walls and façade system due to large inter-floor drifts. Only two decades ago, if excessive motion was predicted, designers would typically make significant architectural or structural modifications to mitigate the motion. However, architectural and structural modifications are typically costly, undesirable, and inefficient. The most efficient means to decrease building motion is often to increase the damping of the structure.

The dynamic vibration absorber (DVA) is a technology that was initially proposed over a hundred years ago (Frahm 1909), but only within the last twenty years has it become commonplace in tall buildings. The two most commons types of DVAs are the tuned mass damper (TMD) and the tuned sloshing damper (TSD). In their simplest forms, both types of DVAs are represented as an auxiliary spring-mass-dashpot system that is attached to the structure at or near a location where the maximum motion occurs (i.e. near the top of a tall building). The natural frequency of the DVA is closely tuned to the natural frequency of the structure, and the DVA's damping is also carefully selected to optimize its performance (Warburton

1982). When properly tuned, the DVA mass will lag behind the motion of the structure, applying a force that is anti-phase to the external wind load, which significantly reduces the motion of the structure.

Although a TMD and TSD are often represented as auxiliary spring-mass-dashpot system, in practice the systems are composed of different elements. The simplest and most affordable TMD is the simple pendulum, in which the TMD mass is hung from cables, and relies on gravity as its restoring force. The damping is typically provided by viscous damping devices, which can be linear or nonlinear. A TSD consists of a tank that is partially filled with a liquid (typically water) that is free to slosh. The frequency of the sloshing liquid is tuned to the structural frequency by selecting the proper tank length and liquid depth. Flow obstructions, such as screens, baffles, or paddles are added to the tank to increase the liquid damping closer to its optimal value.

Although DVAs have been studied extensively analytically, numerically, and experimentally in the laboratory, there are very few studies that have investigated the real-world performance of TMDs or TSDs installed in full-scale structures. One of the earliest performance verifications was conducted by Tamura et al. (1995), in which four towers were monitored for several months with and without shallow-water TSDs. By comparing the tower response with and without TSDs under similar wind conditions, the performance of the system could be inferred. This methodology required long term monitoring in which the TSDs were inactive for many months, which leaves the tower in the undesirable "unprotected" state for a long period of time. More recently, a three-tank TSD system was studied in Toronto, Canada, in which structural monitoring commenced approximately two months prior to the tanks being filled, and monitoring continued after the TSDs were commissioned (Love and Morava 2018). The results showed that the TSD system reduced structural accelerations by approximately 50%.

The efficacy of a DVA is often quantified as the amount of effective damping that the device appears to add to the structure; that is, the "added effective damping", ζ_{add}. The “total effective damping”, ζ_{eff} is the added effective damping plus the “inherent structural damping”, ζ_s; ($\zeta_{eff} = \zeta_{add} + \zeta_s$). The standard deviation of the structural responses with and without the DVA ($\sigma_{s\text{-damped}}$ and $\sigma_{s\text{-}0}$, respectively) are related to the inherent structural damping and total effective damping by:

[1] $$\frac{\zeta_{eff}}{\zeta_s} = \left(\frac{\sigma_{s-0}}{\sigma_{s-damped}}\right)^2$$

A method to determine ζ_{add} through structural monitoring has recently been proposed (Love and Tait 2017). The method requires knowledge of the masses of the structure and DVA, as well as the motion of the DVA and structure to be measured. The methodology has been evaluated using simulations, scale-model testing, and also full-scale measurements on two tall buildings equipped with DVAs (Love and Tait 2017, Love et al. 2018). The methodology was later expanded to enable the prediction of ζ_{add} and ζ_s by adapting the random decrement technique to be applicable to a coupled structure-DVA system (Love and Haskett 2019). Therefore, ζ_{eff} could be directly determined from structural monitoring of the coupled system, which enabled the acceleration reduction provided by the DVA to be directly determined.

In this paper, the methodology to predict ζ_{add} and ζ_s is briefly overviewed. Subsequently, the methodology is applied to three real-world buildings equipped with DVAs.

2 METHODOLOGY

This section briefly presents the methodology by which the added effective damping and inherent structural damping are determined through structural monitoring. The responses of the DVA and structure must be measured, and the equivalent DVA mass, m_{DVA} and generalized mass of the structure, M_s must be known. The equations of motion for the structure-DVA system are given by

[2] $$(M_s + m_{DVA})\ddot{X}(t) + m_{DVA}\ddot{x}(t) + 2\omega_s\zeta_s M_s\dot{X}(t) + \omega_s^2 M_s X(t) = F(t)$$

[3] $$m_{DVA}\ddot{x}(t) + 2\omega_{DVA}\zeta_{DVA}m_{DVA}\dot{x}(t) + \omega_{DVA}^2 m_{DVA}x(t) = -m_{DVA}\ddot{X}(t)$$

where ω_s, and ω_{DVA} are respectively the natural angular frequency of the structure and DVA. The variables, *X(t)* and *x(t)* are the displacement of the structure and relative displacement of the DVA, respectively. In the above equations, a dot above a variable denotes a time derivative, and *F(t)* represents the generalized excitation force applied to the structure. If the excitation is stationary white noise (as is typically assumed in wind engineering), the added effective damping can be estimated as (Love and Tait 2017):

[4] $$\zeta_{add} = \frac{m_{DVA}\omega_s E[\ddot{X}(t)\dot{x}(t)]}{2M_s\sigma_{\ddot{X}}^2}$$

where $E[\ddot{X}(t)\dot{x}(t)]$ is the covariance of the structural acceleration and DVA velocity, and $\sigma_{\ddot{X}}^2$ is the variance of the structural acceleration.

Although the traditional random decrement technique is suitable only for single degree of freedom oscillators, it has been adapted to determine the inherent structural damping of structure-DVA systems (Love and Haskett 2019). The methodology recognizes that the random decrement signatures of the structure and DVA can be calculated to create a forced vibration problem that must be solved. Details of the methodology and its derivation are found elsewhere (Love and Haskett 2019). A random decrement signature of the DVA acceleration, $D_{\ddot{x}}(\tau)$, as well as the structural acceleration, $D_{\ddot{X}}(\tau)$, velocity, $D_{\dot{X}}(\tau)$, and displacement, $D_X(\tau)$ can be estimated from the measured response according to:

[5] $$D_{\ddot{x}}(\tau) = \frac{E[\dot{X}(t)\ddot{x}(t+\tau)]}{\sigma_{\dot{X}}^2}$$

[6] $$D_{\ddot{X}}(\tau) = \frac{E[\dot{X}(t)\ddot{X}(t+\tau)]}{\sigma_{\dot{X}}^2}$$

[7] $$D_{\dot{X}}(\tau) = \frac{E[\dot{X}(t)\dot{X}(t+\tau)]}{\sigma_{\dot{X}}^2}$$

[8] $$D_X(\tau) = \frac{E[\dot{X}(t)X(t+\tau)]}{\sigma_{\dot{X}}^2}$$

These random decrement signatures are related by the structural equation of motion:

[9] $$D_{\ddot{X}}(\tau) + 2\zeta_s\omega_s D_{\dot{X}}(\tau) + \omega_s^2 D_X(\tau) = \frac{m_{DVA}}{M_s} D_{\ddot{x}}(\tau)$$

A best fit must then be conducted to determine the values of ζ_s and ω_s that best solve Equation (9). Love and Haskett (2019) recommended an energy approach to solve for ζ_s. When ζ_s and ω_s are estimated, the goodness of fit can be assessed by regenerating the random decrement signature of the structure using the parameters determined along with the measured DVA random decrement signature, $D_{\ddot{x}}(\tau)$. If the measured and regenerated random decrement signatures of the structural response are in good agreement, the system is well-described by the structural properties employed.

Therefore, the inherent structural damping, and the added effective damping provided by the DVA are determined using the measured response of the structure and DVA, as well as the masses of the structure and DVA. Equation (1) can then be employed to determine the acceleration reduction produced by the DVA.

3 APPLICATION

The methodology presented in Section 2 is applied to structural monitoring that has been conducted on three tall buildings equipped with DVAs. The first tall building is 10 Barclay, located in New York City, USA, which has been equipped with a unidirectional TSD. The second building is an anonymous supertall tower equipped with two identical bidirectional TMDs. The third building is also an anonymous supertall tower equipped with one bidirectional TMD.

3.1 Building #1

The first tall building studied is 10 Barclay Street, located in New York City, USA. The tower has a height of 204m, with typical floor plate dimensions of 19.5m x 45.4m. Wind tunnel testing indicated that with the assumed inherent structural damping ratio of 2%, the predicted peak total accelerations would be 23 milli-g at the 10-year return period wind event. The total accelerations are dominated by motion along the narrow building dimension, therefore a unidirectional TSD was implemented to reduce accelerations for this structural mode of vibration. The TSD was designed to provide 1.5% added effective damping to reduce the 10-year peak acceleration to less than 18 milli-g, which is the typical 10-year acceleration criteria used in North American practice.

The final as-built natural angular frequency of the structure was 0.97 rad/s. The internal TSD tank dimensions were 13.72m x 5.49m, with a quiescent water depth of 1.98m. Damping screens with an empirically-determined loss coefficient of 1.96 were positioned at ¼, ½, and ¾ of the tank length to increase the liquid damping. The DVA-structure mass ratio was m_{DVA}/M_s = 0.9%. Additional details on the structure and TSD design can be found elsewhere (Love et al. 2018, Morava et al. 2010).

The structural accelerations and TSD wave heights were monitored during a significant wind event that occurred February 19-20, 2011 (Love et al. 2018). Figure 1 shows the measured structural accelerations (in the dominant sway direction) and the measured TSD wave heights near the tank end wall over a six-hour period. The peak acceleration during this period is ~8 milli-g, while the peak wave height is ~0.4m. Figure 2 shows the spectra associated with these monitoring results. The frequency axis of the spectra is normalized by the structural frequency, while the spectral amplitude is normalized by the response variance. The double-peak associated with a coupled structure-TSD system is clearly visible in the spectra.

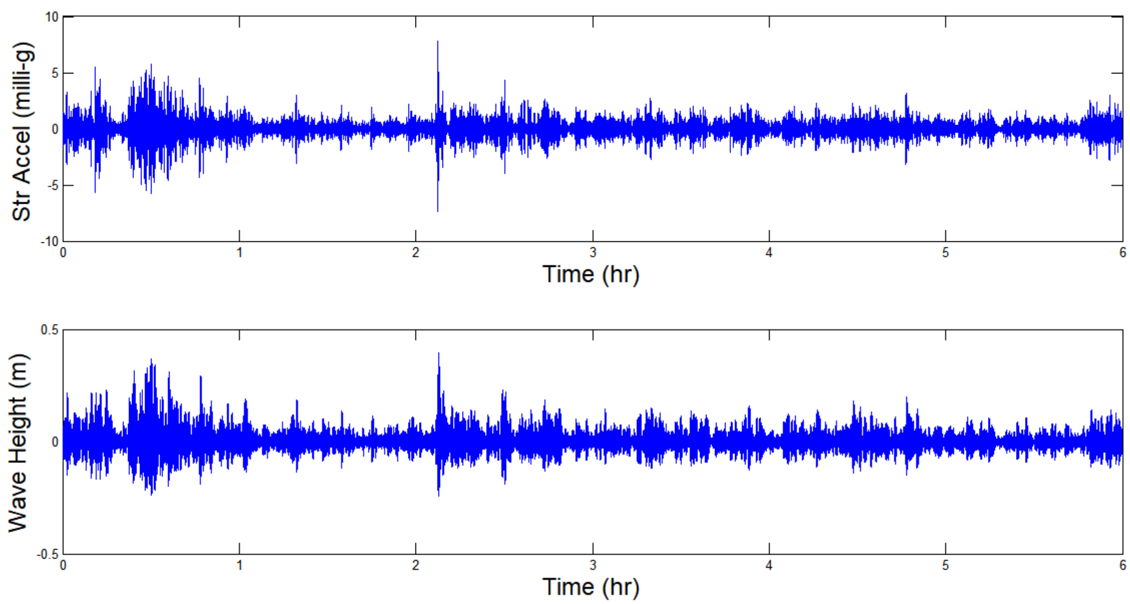

Figure 1: Measured structural acceleration and TSD wave heights (Building #1)

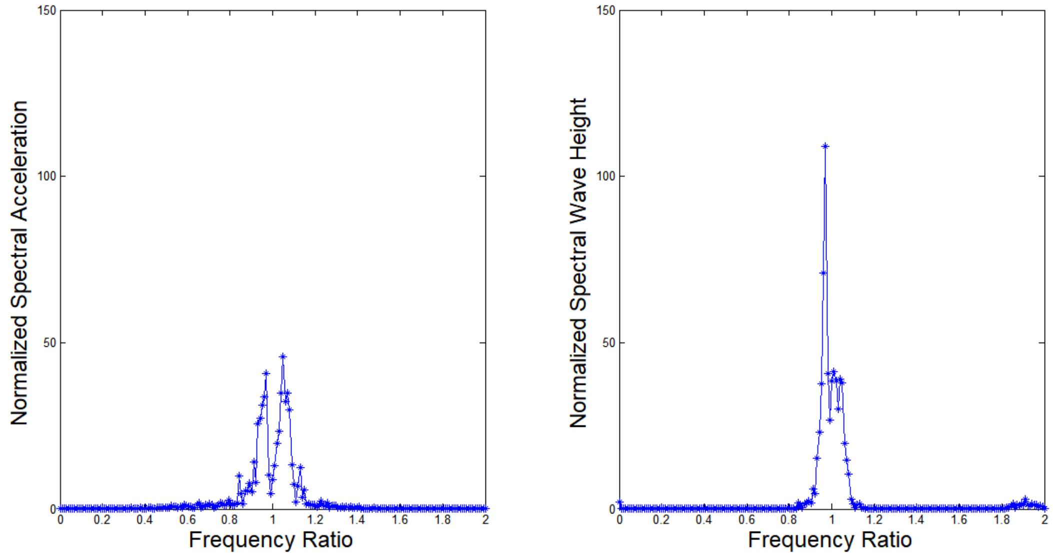

Figure 2: Normalized structural acceleration and TSD wave height spectra (Building #1)

The methodology outlined in Section 2 is applied to estimate the as-built added effective damping and inherent structural damping. The added effective damping is predicted to be 2.1% and the inherent structural damping is estimated to be 1.6% based on these six hours of monitoring results. Therefore, the total effective damping is 3.7%, which results in an acceleration reduction of 34%. Figure 3 shows the measured and regenerated random decrement signatures of the structure. The two signatures are in reasonable agreement, which suggests that the structural properties determined are representative of the actual properties.

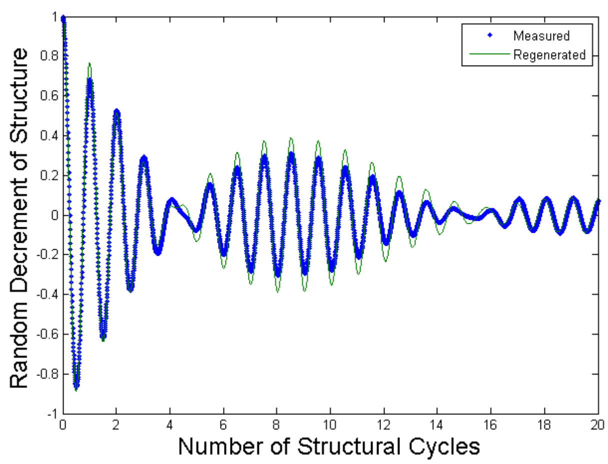

Figure 3: Measured and regenerated random decrement signatures of structural response (Building #1)

3.2 Building #2

The second building considered in this study is an anonymous super-tall building. Wind tunnel tests indicated that the first two sway modes were susceptible to excessive wind-induced motion during common wind events. Assuming 1% inherent structural damping, wind tunnel testing predicted that the peak total accelerations would be high even at the monthly return period wind event. Therefore, two bidirectional TMDs having a total TMD-structure mass ratio of m_{DVA}/M_s = 4.4% were implemented to reduce monthly accelerations by a factor of two.

The two structural sway frequencies (X- and Y-directions) were predicted to be very similar, therefore the TMD was designed to have identical frequencies in the X- and Y-directions. The as-built natural angular frequencies of the tower were approximately 0.52 and 0.54 rad/s (Love et al. 2018). Structural monitoring conducted while the TMDs were "locked-out" under small motions (<0.5 milli-g) revealed an inherent structural damping of 0.6% (Love et al. 2018). However, inherent structural damping often increases somewhat with increasing structural response amplitudes, therefore it is anticipated that higher inherent structural damping levels will be measured on windy days when the structural response is much greater than 0.5 milli-g.

Figure 4 shows the structural acceleration and TMD relative displacement measured on a windy day in March 2017. Although the TMDs are bidirectional, for brevity, only the Y-direction of motion is considered herein. The peak structural acceleration is ~4 milli-g, while the peak TMD displacements (relative to the structure) are ~0.4m. The displacements of the two TMDs are nearly identical, as expected. Figure 5 shows the corresponding spectra, where the frequency has been normalized by the natural frequency of the structure and each spectral amplitude has been normalized by its corresponding response variance. Two peaks are not clearly visible in either the structure or the TMD spectra. The lack of two peaks in these spectra is hypothesized to be a result of the friction that exists in the TMD (Love et al. 2018). Although it has been shown that the TMD overcomes friction at building accelerations of 0.5-1 milli-g, friction is a highly nonlinear force that can alter the system behaviour substantially as the TMD is "locked-out" by friction forces. When the TMD is locked-out by friction, the TMD damping and stiffness characteristics are altered by a significant amount. Moreover, the locked-out TMD will also adjust the natural frequency of the structure itself since the inactive TMD mass will lump its mass onto the structure.

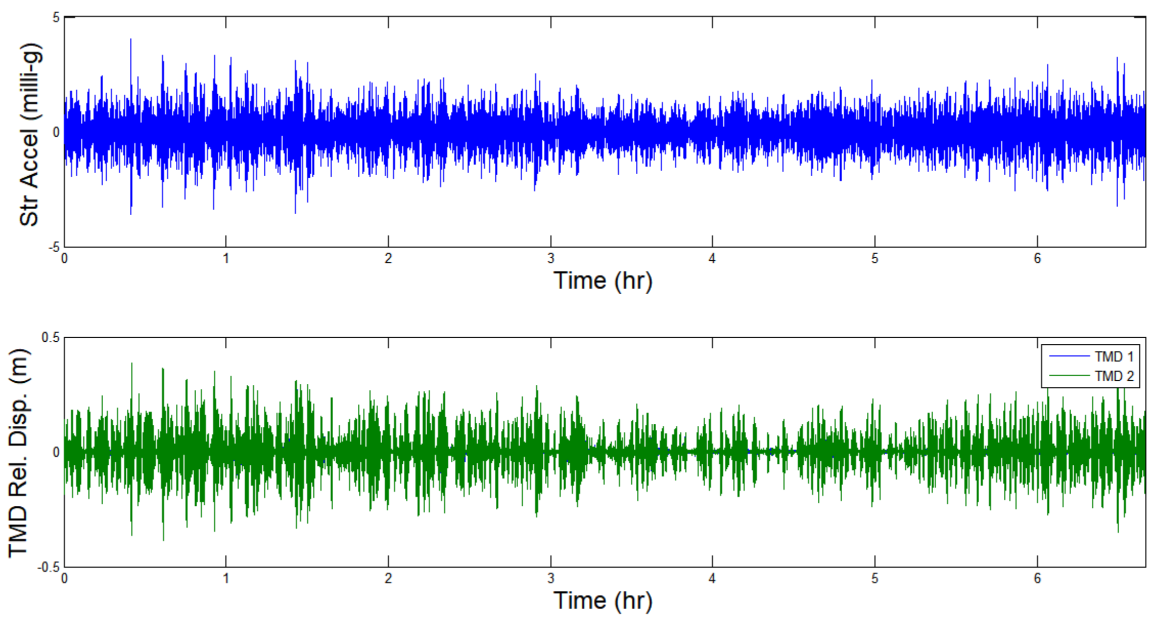

Figure 4: Measured structural acceleration and TMD displacements (Building #2)

The methodology outlined in Section 2 is applied to estimate the as-built added effective damping and inherent structural damping. The added effective damping is predicted to be 3.7% and the inherent structural damping is estimated to be 1.2% based on these monitoring results. Therefore, the total effective damping is estimated to be 4.9%, which would result in an acceleration reduction of 50%. The inherent structural damping measured during this significant wind was a factor of two greater than that measured when the TMD was locked-out on a quite calm day. This data may suggest that the inherent structural damping increases with structural response amplitude. Figure 6 shows the measured and regenerated random decrement signatures of the structure. The two signatures are in reasonable agreement during the first five cycles when the response amplitude decreases rapidly. However, after these initial cycles, the regenerated response shows some discrepancies with the measured random decrement signature, including a shift in the natural frequency of the structure.

It is expected that the discrepancies observed after several response cycles are a result of the nonlinearity of the system. Although not shown herein, additional analysis was conducted on this structure-TMD system during other wind events. These analyses indicated that the predicted added effective damping is consistent. However, the predicted inherent structural damping varied significantly from approximately 0.3% to 2%, and the agreement between measured and regenerated random decrement signature was inconsistent. It is expected that the methodology is not well suited for this type of system, where the structure and TMD are highly nonlinear due to friction, which can alter both the TMD damping, as well as the structural frequency. Moreover, it has previously been shown that the method accuracy decreases when the added effective damping is high, and the inherent structural damping is very low (Love and Haskett 2019), as is the case for this structure.

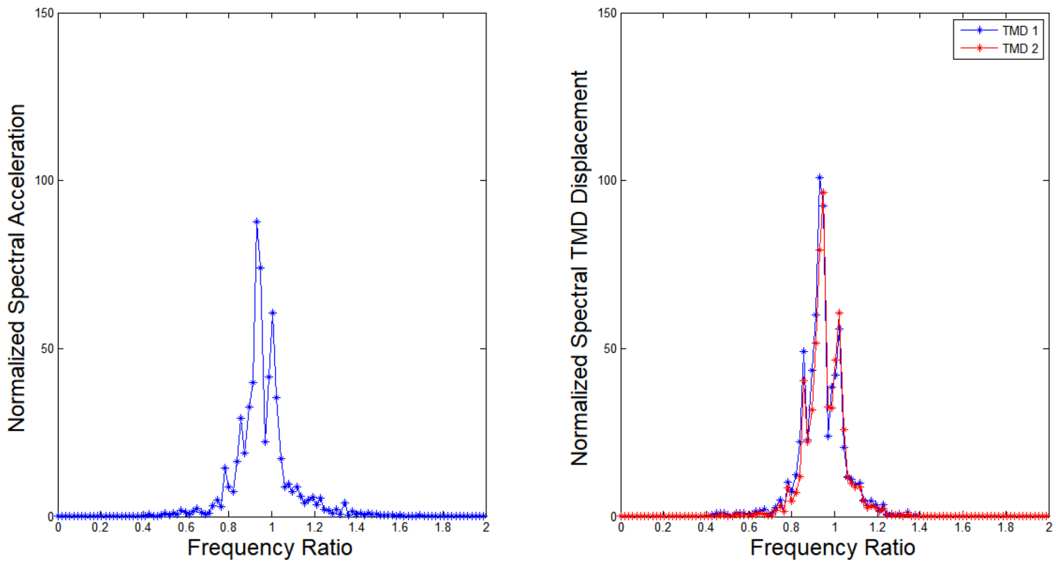

Figure 5: Normalized structural acceleration and TMD relative displacement spectra (Building #2)

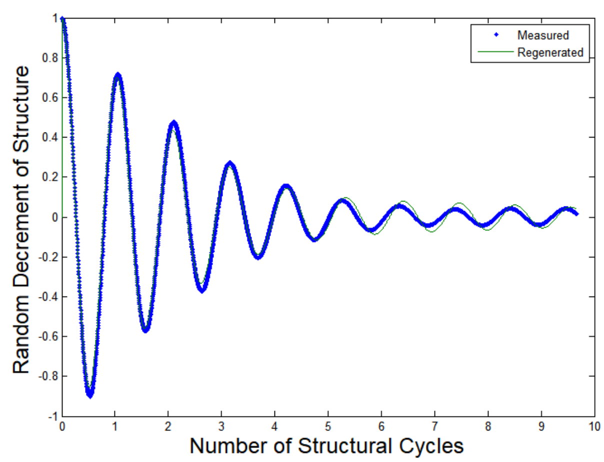

Figure 6: Measured and regenerated random decrement signatures of structural response (Building #2)

3.3 Building #3

The third building considered is also an anonymous super-tall building that has been equipped with a bidirectional TMD. Wind tunnel testing indicated that the structure, with an assumed inherent structural damping of 1%, could experience excessive wind-induced dynamic motion during bi-annual (6 month) wind events. To reduce building motions to acceptable levels for occupant comfort, implementation of a TMD that could increase the total effective damping of the structure to 2.6% was proposed. To achieve this target level of total effective damping, a TMD with a TMD-structure mass ratio of m_{DVA}/M_s = 1.6% was considered for this building.

During the tuning of the TMD, the TMD's natural angular frequency was set to 1.01 rad/s. Since the TMD was tuned, the natural angular frequency of the structure has decreased. At the time of the structural monitoring reported herein (in July 2017), the structural frequency was found to be approximately 0.91 rad/s in each sway direction. Figure 7 shows the structural acceleration and the TMD displacement (relative to the structure) measured during a major wind event. Although the TMD is bidirectional, only the Y-direction is reported herein for brevity. The peak structural acceleration was approximately 25 milli-g, while the peak TMD relative displacement is 0.6m.

Figure 8 shows the corresponding normalized spectra of the structural acceleration and TMD relative displacement. The frequency axis is normalized by the natural frequency of the structure, while the spectral amplitudes are normalized by the variance of the response. Since the structural frequency is lower than the TMD frequency, only one peak is clearly visible in the spectra.

The methodology outlined in Section 2 is applied to estimate the as-built added effective damping and inherent structural damping. The added effective damping is predicted to be 1.5% and the inherent structural damping is estimated to be 1.2% based on these 2 hours of monitoring results. Therefore, the total effective damping is estimated to be 2.7%, which would result in an acceleration reduction of 33%. Figure 9 shows the measured and regenerated random decrement signatures of the structure. The two signatures are in very good agreement, indicating that the estimated structural damping is reliable. However, unlike the previous two buildings studied herein, the random decrement signature does not show a "beating" phenomenon that is commonly associated with a coupled system. This lack of beating is due to the mistuning. Despite the current mistuning, the achieved total effective damping is still greater than the target level determined at the design stage of the TMD.

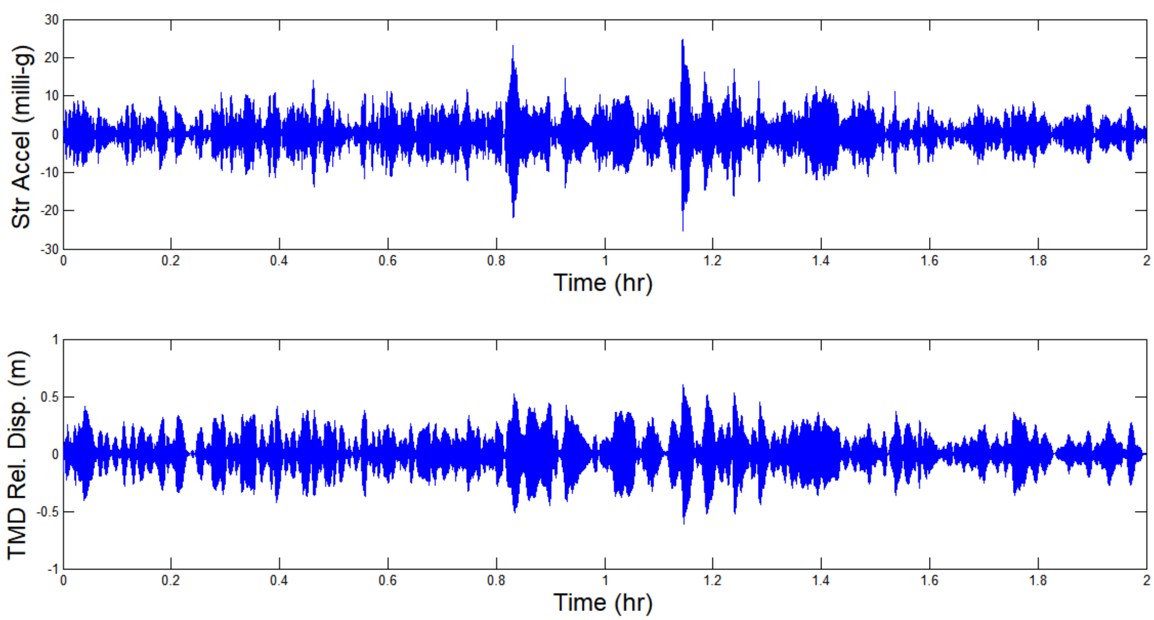

Figure 7: Measured structural acceleration and TMD displacements (Building #3)

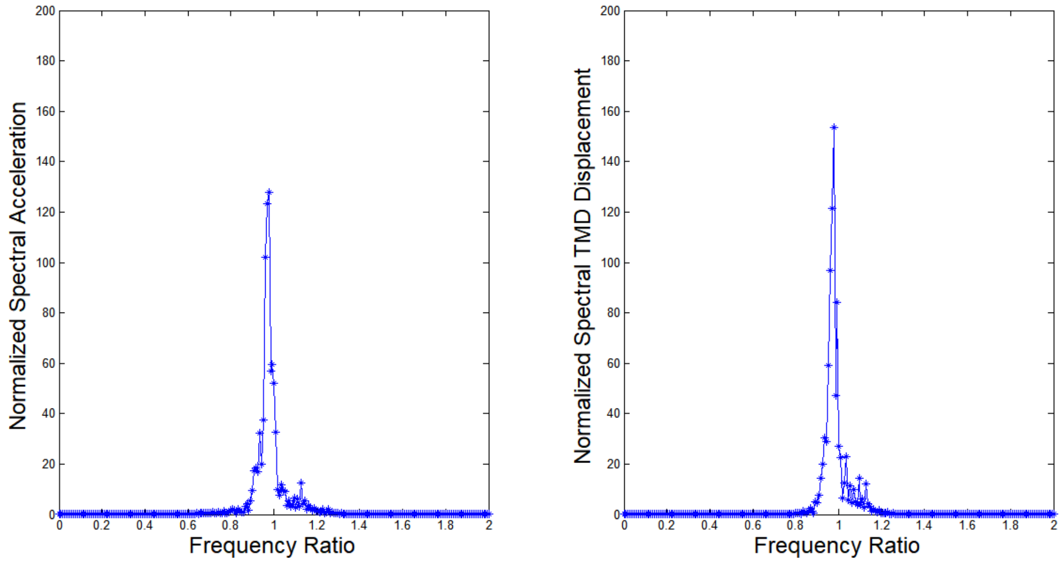

Figure 8: Normalized structural acceleration and TMD relative displacement spectra (Building #3)

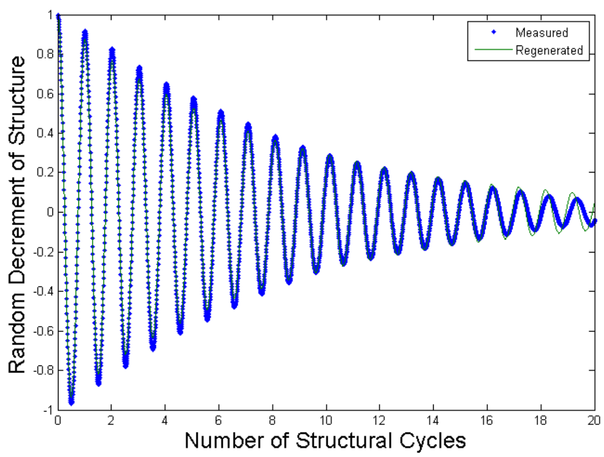

Figure 9: Measured and regenerated random decrement signatures of structural response (Building #3)

4 CONCLUSIONS

In this study, the performance of DVAs installed in three real-world structures has been presented. Their performance has been evaluated using structural monitoring of the structure-DVA systems to determine both the inherent structural damping, as well as the added effective damping provided by the DVA. With this information, the motion reduction provided by the DVA can be determined. The findings for each of the three buildings are as follows:

- Building #1 was equipped with a unidirectional TSD with a TSD-structure mass ratio of 0.9%. During a wind event in which the peak structural acceleration was 7 milli-g, the measured inherent structural damping and added effective damping were 1.6%, and 2.1%, respectively. The TSD reduced wind-induced structural motion by 34%.
- Building #2 was equipped with two identical bidirectional TMDs with a total TMD-structure mass ratio of 4.4%. During a wind event in which the peak structural acceleration was 4 milli-g, the measured inherent structural damping and added effective damping were 1.2%, and 3.7%, respectively. The TMD system therefore reduced structural motion by 50%. System

nonlinearities associated with TMD friction was hypothesized to affect the prediction of the inherent structural damping.

- Building #3 was equipped with a bidirectional TMD with a TMD-structure mass ratio of 1.6%. The TMD is currently not optimally tuned due to the structural frequencies having decreased since the TMD was tuned. During a wind event in which the peak structural acceleration was 25 milli-g, the measured inherent structural damping and added effective damping were 1.2%, and 1.5%, respectively. The TMD reduced structural motion by 33%.

The results of this study have demonstrated that the TSD and TMDs considered have reduced building motions considerably. DVAs are therefore an effective means to improve the serviceability performance of tall buildings susceptible to excessive wind-induced motion.

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