DATA-DRIVEN PERFORMANCE ASSESSMENT OF TUNED MASS DAMPERS IN TOWER STRUCTURES

Kosmas DRAGOS¹, Maria STEINER², Volkmar ZABEL³, Kay SMARSLY⁴

ABSTRACT

The dynamic behavior of slender structures, such as tower structures, is frequently characterized by unfavorable responses resulting in resonant phenomena between the dynamic actions and the structures, such as the “lock-in” effect. For counteracting unfavorable vibrations of slender structures, tuned mass dampers are frequently employed. Tuned mass dampers are attached to structures as structural subsystems “tuned” to suppress a response frequency, typically corresponding to a vibration mode, whose oscillation amplitude exceeds the limit state criteria of structural design. The energy dissipation mechanism of tuned mass dampers is the out-of-phase response with respect to structures at the “tuning” frequency. Tuned mass dampers are either installed at the construction stage, based on design predictions of structural dynamic responses, or at a later stage of the structure’s life time following observations of large oscillation incidents, which have been unforeseen in design. However, given the potential changes in structural state, due to ageing or structural degradation, or the potential changes in the state of the tuned mass damper (TMD) itself (e.g. due to “de-tuning”), assessing the performance of tuned mass dampers is paramount for ensuring that the structural response remains within safety limits. This paper presents a data-driven approach towards assessing the performance of tuned mass dampers. Drawing from the theory of TMD operation, the proposed approach builds upon the expected phase difference between the TMD response and the structural response, which deviates from zero and, in the optimal case, lies in the vicinity of 90°. The proposed approach is validated using acceleration response data from a long-term structural health monitoring system installed in a telecommunication tower equipped with a TMD.

Keywords: Slender structures; tuned mass dampers; data-driven analysis; dynamic response; structural control

1. INTRODUCTION

In structural design, operational requirements often result in adopting unconventional structural systems, such as irregular systems with non-uniform distribution of mass and stiffness or structural systems with high slenderness. Slender structures, such as towers, masts and bridges, exhibit a complex structural dynamic behavior, which is characterized by the contribution of multiple vibration modes. Furthermore, in several cases specific oscillation phenomena due to load-structure interaction can be observed. In addition, advances in material science over the last decades with the introduction of high-strength materials, have enabled the considerable reduction of cross section dimensions of structural elements. This reduction of cross section dimensions allows for the construction of slender structures, which despite their cost-efficient construction are often prone to considerable oscillations. To ensure that structures meet serviceability and structural integrity criteria throughout their life time, counteracting excessive oscillation phenomena is paramount.

Standard practice in improving the dynamic behavior of structures focuses on artificially enhancing the damping forces in an attempt to counteract oscillations. The most widely used class of damping systems

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for civil engineering structures are the so-called “tuned mass dampers” (TMD). The favorable effect of a tuned mass damper (TMD) is expressed by the splitting of an initial frequency component, which dominates the structural response of the structure prior to installing the TMD, into two frequency components, one with higher frequency and one with lower frequency than the initial frequency component. The out-of-phase oscillation of the TMD with respect to the structure results in counteracting inertia forces, thus keeping the oscillation amplitudes of the structure within acceptable limits. The definition of TMD tuning properties depends either on the anticipated structural dynamic behavior, if defined during the design stage, or on the structural dynamic behavior observed following incidents of unanticipated excessive oscillation. However, both the structural properties and the TMD tuning properties may vary during the structural life cycle. To ensure proper TMD operation for keeping the structural response within target limits, assessing the TMD performance using information from observations of the structural dynamic behavior, e.g. obtained from structural health monitoring (SHM) data, is paramount.

Investigating the structural behavior of monitored structures holds a central part in structural health monitoring. Typical structural behavior aspects studied in SHM include, e.g., basic structural dynamic properties or structural responses indicative of specific physical phenomena. Depending on whether the underlying physical principles of the structural behavior aspect under consideration are accounted for, the SHM analysis is distinguished into physics-based analysis and into data-driven analysis. Given the simplifications associated with the theoretical analysis of specific physical phenomena, adopting physics-based analysis in SHM may not be always the optimal solution. Rather, the application of data-driven analysis, i.e. not considering the physical principles a priori, often yields patterns from the data that, combined with adequate background in the phenomena under investigation, may result in informative conclusions. Data-driven analysis has been extensively applied in SHM, for example, for leveraging artificial intelligence (Smarsly and Hartmann, 2007), for monitoring wind turbines (Smarsly and Hartmann, 2010; Smarsly et al., 2012, 2013), for enabling automated control in intelligent SHM systems (Smarsly and Hartmann, 2009), for identifying structural properties of civil infrastructure (Dragos and Smarsly, 2017), and for system identification (Peeters, 2000). By defining suitable patterns to be found within the SHM data, monitoring the behavior of structural subsystems, e.g. of tuned mass dampers, is also possible.

From the middle of the previous century, considerable research has been performed on the optimal design of tuned mass dampers. Following the introduction of TMD design by Den Hartog (1956), approaches on defining optimal TMD properties have been reported, for example, by Fujino and Abé (1993) and Rana and Soong (1998). With respect to TMD performance assessment, research efforts have been focusing on using numerical simulations (Piknaew and Fujino, 2001) or on combining numerical simulations with laboratory tests in wind tunnels (Tuan and Shang, 2014). Employing system identification methods for long-term TMD performance assessment has been proposed by Weber and Feltrin (2010), and the use of monitoring data for identifying the activation level of a TMD via the variation of damping forces on a pedestrian bridge has been presented by Caetano et al. (2010). This paper presents a TMD performance assessment approach through data-driven analysis on long-term SHM data. Contrary to the aforementioned approaches in assessing the performance of tuned mass dampers, the TMD performance assessment approach presented herein is based on extracting patterns indicative of TMD operation from SHM data. Specifically, drawing from the operation principle of out-of-phase TMD response with respect to the structure at the tuning frequency, estimates of the frequency band in which the TMD is tuned are obtained. The TMD performance assessment approach is validated through field tests with SHM data from a telecommunication tower.

In the second section of the paper, the patterns to be found within the SHM data are defined. Then, the steps of the TMD performance assessment approach are shown. Next, the validation tests are presented, followed by the summary and conclusions and by an outlook on future research.
2. PATTERN DEFINITION FOR DATA-DRIVEN PERFORMANCE ASSESSMENT OF TUNED MASS DAMPERS

In this section, the preliminary steps towards performing the data-driven assessment of tuned mass dampers are explained. Specifically, the pattern indicative of TMD operation is formulated. First, the basic principle for formulating the pattern is illuminated through a simple single-degree-of-freedom oscillator example. Then, for establishing the pattern, simulations on a cantilever structure equipped with a TMD are performed.

2.1 Basic TMD operation principle

The basic principle governing the TMD operation is illustrated through a simple single-degree-of-freedom example, as described by Connor (2002). As a TMD is normally tuned to a single vibration mode, the model of a single-degree-of-freedom oscillator can be used. The oscillator is composed of a mass \( m_o \), a spring with stiffness \( k_o \) and a dashpot with the viscous damping coefficient \( c_o \) simulating the energy dissipation in the structure. To this oscillator, a TMD of mass \( m_d \) is connected by a spring with stiffness \( k_d \) and a dashpot with damping coefficient \( c_d \), as shown in Figure 1.

![Figure 1: Single degree-of-freedom system with a tuned mass damper attached.](image)

The oscillator is subjected to harmonic excitation \( P(t) = p_o \cdot \sin \Omega t \). Drawing from the field of structural dynamics, the dynamic equilibrium equations of the oscillator and the TMD are:

\[
\begin{align*}
 m_o \cdot \ddot{x} + c_o \cdot \dot{x} + k_o \cdot x &= P - m_d \cdot (\ddot{x}_d + \dot{x}) \\
 m_d \cdot (\ddot{x}_d + \dot{x}) + c_d \cdot \dot{x}_d + k_d \cdot x_d &= 0
\end{align*}
\]

Equations 1 and 2, \( \ddot{x}, \dot{x} \) and \( x \), represent the acceleration, the velocity, and the displacement of the oscillator, respectively. According to Connor (2002), a near-optimal tuning is achieved if the natural frequency \( \omega_d \) of the TMD coincides with the natural frequency \( \omega \) of the oscillator:

\[
\omega_d = \sqrt{\frac{k}{m_d}} = \omega = \sqrt{\frac{k}{m}}
\]

Considering the unfavorable case of resonance (\( \omega = \Omega \)), the structural responses of the oscillator and of the TMD are given in Equations 4 and 5, respectively.

\[
\begin{align*}
 x &= A \sin (\Omega \cdot t + \phi) \\
 x_d &= A_d \sin (\Omega \cdot t + \phi + \theta)
\end{align*}
\]

Equations 4 and 5, \( \phi \) represents the phase angle of the oscillator response, and \( \theta \) denotes the phase difference between the TMD and the oscillator. The maximum displacements of the TMD and of the oscillator are represented by \( A_d \) and \( A \), respectively. Since the excitation is harmonic, the phase angles of Equations 4 and 5 are calculated as follows:
\[ \varphi = \arctan \left( \frac{2\xi_m}{m_d} + \frac{1}{2\xi_d} \right) \quad \theta = \arctan \left( \frac{\pi}{2} \right) \] (6)

In Equation 6, \( \xi \) and \( \xi_d \) denote the damping ratios of the oscillator and of the TMD, respectively. Evidently, the effect of the TMD on the structural response is determined through a phase difference between the oscillator and the TMD, which in the simplified case of the aforementioned example is equal to \( \pi/2 \) (90°). For analyzing the TMD behavior in multi-degree-of-freedom systems, the basic principle described above is applied to vibration mode \( n \) related to the respective generalized mass \( m_n \), generalized stiffness \( k_n \), and generalized damping \( c_n = 2\xi_n m_n \omega_n \). It should be noted that the definition of optimal TMD properties is performed through further investigation for establishing the optimal tuning frequency ratio (\( k_{\text{opt}} = \omega_d/\omega \)) and the optimal critical damping ratio of the TMD (\( \xi_{d,\text{opt}} \)) corresponding to a predefined mass ratio (\( \mu = m_d/m_n \)). The aforementioned parameters are obtained as solutions of an optimization problem with the objective of minimizing the dynamic amplification factor, i.e. the factor which expresses the dynamic amplification of the structural response due to dynamic load \( p_o \sin \Omega t \) as compared to the structural response to an equivalent static application of load \( p_o \), of the oscillator response. For an optimal TMD design, the phase difference between the TMD and the oscillator generally deviates from 90°; as will be shown in the next subsection, however, the actual angle must deviate from 0° or 180° and usually lies in the vicinity of 90°.

2.2 Pattern definition through a simulation example

Using the basic TMD operation principle, the pattern used for the data-driven analysis in this study is formulated through simulations of a 5-degree-of-freedom cantilever structure, which is shown in Figure 2. The cantilever is equipped with a TMD attached to the 5th (uppermost) degree of freedom (DOF) and tuned to suppress the first vibration mode, which dominates the structural dynamic response. Therefore, the generalized mass and stiffness of the first vibration mode, \( m_1 \) and \( k_1 \), respectively, are used for estimating the optimal TMD properties. With respect to the excitation, the cantilever is subjected to wind-induced vibrations, typical of slender tower structures, which is assumed to be random with Gaussian distribution (Wagner et al., 2017). For the load profile, the provisions of Eurocode 1 (STN, 2010) for terrain category I, basic wind speed \( v_b = 35 \) m/s, air density \( \rho = 1.25 \) kg/m\(^3\) and orography factor \( c_o = 1 \) are adopted.

![Cantilever structure used for simulations](image)

Both the inertial and elastic properties of the cantilever structure are discretized to the 5 degrees of freedom, which are all translational only in the \( x \)-direction. All rotational degrees of freedom are neglected. For defining the optimal TMD parameters, several approaches have been developed, as mentioned previously. An early approach for selecting optimal TMD parameters has been proposed by Den Hartog (1956). Despite its known weakness in neglecting the damping of the structure, Den Hartog’s approach is still widely used. As will be shown, the deviations from other approaches that may be found in literature, for example, the formulas proposed by Tsai and Lin (1993), are not significant.
for systems with weak damping. Therefore, in this study, adopting the optimal parameters from Den Hartog’s approach, shown in Equation 7, is considered adequate, while the corresponding parameters from the formulas by Tsai and Lin, shown in Equations 8 and 9, are provided for comparison purposes.

**Den Hartog (1956):**

\[
\xi_{d,\text{opt}} = \frac{3\mu}{8(1+\mu)} \quad \kappa_{\text{opt}} = \frac{\omega_d}{\omega_1} = \frac{1}{1+\mu} \quad \mu = \frac{m_d}{m_n}
\]  

(7)

**Tsai and Lin (1993):**

\[
\xi_{d,\text{opt}} = \sqrt{\frac{3\mu}{8(1+\mu)(1-0.5\mu)}} + 0.151\zeta - 0.17\zeta^2 + \left(0.163\zeta + 4.98\zeta^2\right)\mu \\
\kappa_{\text{opt}} = \sqrt{\frac{1-0.5\mu}{1+\mu}} + \sqrt{1-2\zeta^2 - 1 - \left(2.375 - 1.034\sqrt{\mu} - 0.426\mu\right)\zeta} \sqrt{\mu} \quad \mu = \frac{m_d}{m_n}
\]

(8)

(9)

By performing modal analysis on the cantilever structure, the first vibration mode is found at a frequency of \(f_1 = 2.13\) Hz having a generalized mass of \(m_1 = 12.76\) t. The mass ratio between the TMD and the structure is selected as \(\mu = 4\%\). Damping is assumed constant for all modes at \(\zeta = 2\%\). Using Equations 7, 8 and 9, the optimum TMD parameters are calculated and summarized in Table 1.

**Table 1. Optimum TMD parameters calculated from Den Hartog (1956) and Tsai and Lin (1993).**

<table>
<thead>
<tr>
<th></th>
<th>Den Hartog</th>
<th>Tsai and Lin</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\xi_{d,\text{opt}})</td>
<td>0.120</td>
<td>0.125</td>
</tr>
<tr>
<td>(\kappa_{\text{opt}})</td>
<td>0.9615</td>
<td>0.9616</td>
</tr>
</tbody>
</table>

From Table 1, it is evident that adopting the TMD optimal parameters estimated by Den Hartog’s approach is sufficient for this study. A series of time-history analyses under the loading profile shown in Figure 2 and considering Gaussian excitation is performed on the cantilever structure. Starting from the optimal parameters given in Table 1, different cases of “de-tuning”, i.e. deviations from the optimal parameters, are considered. The de-tuning is simulated by changing the mass and stiffness of the TMD, thus resulting in different values of \(\kappa\), while \(\zeta\) is kept constant. To illustrate the effect of the TMD in each analysis, the displacement response data sets from the 5th DOF and from the TMD are collected. Both displacement response data sets are transformed into the frequency domain and the cross spectrum between the response data sets is obtained. Subsequently, the phase angle of the cross spectrum, representing the phase difference between the response of the structure and the response of the TMD, is retrieved. Figure 3 shows the effect of the TMD parameters on the root mean square displacement of the 5th DOF for all sets of TMD parameters considered. The respective effect of modifying the TMD parameters on the phase angle of the cross spectra between the displacement response data set of the 5th DOF and the corresponding response data set of the TMD is illustrated in Figure 4.

![Figure 3: Effect of TMD on the RMS of the displacement of the 5th DOF.](image-url)
In Figure 3, the favorable effect of the TMD on the structural response of the cantilever for the case of optimal TMD design is evident. Combining this effect with observations from Figure 4, it is clear that for optimal TMD operation the phase angle of the cross spectrum between the displacement response data set of the 5th DOF and the displacement response data set of the TMD deviates from zero. It can also be observed that the phase angle is close to 90° for frequency ratios close to $\kappa_{opt}$, which is usually slightly smaller than 1. Even though small deviations from $\kappa_{opt}$ can lead to noticeable changes in the phase angle, it should still remain in the vicinity of 90°, rather than becoming zero, when the TMD generates a damping effect. Therefore, the pattern that will be used for data-driven analysis, is the deviation of the phase angle of the cross spectrum between the displacement response data of the structure and the displacement response data of the TMD from zero. The steps of the TMD performance assessment approach are described in the next section.

Figure 4: Effect of TMD parameters on the phase angle of the cross spectra between the displacement of the 5th DOF and the displacement of the TMD.

3. PERFORMANCE ASSESSMENT OF TUNED MASS DAMPERS USING SHM DATA

This section presents the steps of the TMD performance assessment approach using data from structural health monitoring systems. Contrary to the simulation example described in the previous section, in the TMD performance assessment approach, no assumptions on the optimal TMD parameters are made. Rather, the pattern indicating TMD operation, formulated in the previous section, is utilized in a purely data-driven manner. The steps of the TMD performance assessment approach are described below.

1. Acceleration response data sets from the structure and from the TMD are collected.
2. The acceleration response data sets are integrated to obtain the corresponding displacement response data sets using numerical integration.
3. The displacement response data sets are transformed into the frequency domain via the fast Fourier transform (FFT).
4. The Fourier amplitude spectra of the displacement response data sets from the structure are plotted for detecting peaks corresponding to vibration modes (modal peaks).
5. The cross spectrum between the displacement response data sets from the structure and the displacement response data sets from the TMD is obtained.
6. The phase angles of the cross spectrum at the modal peaks are retrieved.
7. Based on the deviation of the cross spectrum phase angles from zero and on the proximity of the phase angles to 90°, the TMD operation at the corresponding frequency band is assessed.

For integrating the acceleration response data sets in this study, the Newmark-$\beta$ integration algorithm is used (Newmark, 1959), which is shown in Equations 10 and 11.

$$\ddot{x}_{n+1} = \ddot{x}_n + (1 - \gamma) \cdot \Delta t \cdot \dddot{x}_n + \gamma \cdot \Delta t \cdot \dddot{x}_{n+1}$$

$$x_{n+1} = x_n + \ddot{x}_n \cdot \Delta t + \frac{1}{2} \cdot (\Delta t)^2 \cdot \left[ (1 - 2\beta) \cdot \dddot{x}_n + 2\beta \cdot \dddot{x}_{n+1} \right]$$

![Equations 10 and 11](attachment:equations.png)
In Equations 10 and 11, \( \gamma \) and \( \beta \) are the Newmark coefficients, which are set equal to \( \gamma = 0.5 \) and \( \beta = 0.25 \). For conducting FFT, the Cooley-Tukey algorithm is used (Cooley and Tukey, 1965), as shown in Equation 12.

\[
X_j = \sum_{n=0}^{N-1} x_n \cdot e^{-2\pi j n/N} \quad j \in [0, N) \quad N \in \mathbb{Z} \quad \omega = \frac{j}{N \cdot \Delta t}
\]  

(12)

In Equation 12, \( X \) is the \( N \)-sized complex discrete Fourier transform of the \( N \)-sized time series \( x \) holding the displacement response data. The time increment is denoted as \( \Delta t \). The amplitude \( A \) and the phase angle \( \theta \) of the FFT are calculated, as follows:

\[
A(\omega_j) = |X_j| = \sqrt{\text{Re}(X_j)^2 + \text{Im}(X_j)^2}
\]

(13)

\[
\theta(\omega_j) = \text{arg}(X_j) = \arctan\left(\frac{\text{Im}(X_j)}{\text{Re}(X_j)}\right)
\]

(14)

Finally, the corresponding \( N \)-sized cross spectrum \( G_{x,x+\hat{x}} \) between the displacement response data set of the structure \( (x) \) and the displacement response data set of the TMD \((x+\hat{x})\) is calculated in Equation 15.

\[
G_{j(x,x+\hat{x})} = X_{j,x} \cdot X_{j,x+\hat{x}}^* \quad j \in [0, N) \quad N \in \mathbb{Z}
\]

(15)

In Equation 15, “*” denotes complex conjugate. The steps of the TMD performance assessment approach are flowcharted in Figure 5.

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**Figure 5: Flowchart of the TMD performance assessment approach.**
4. VALIDATION TESTS ON A TELECOMMUNICATION TOWER

The applicability of the TMD performance assessment approach is validated through field tests on a telecommunication tower, which are shown in this section. First, the telecommunication tower and the SHM instrumentation are described, followed by the presentation of the results from applying the TMD performance assessment approach.

4.1 Description of the telecommunication tower and of the SHM instrumentation

The telecommunication tower is approximately 190 m tall and comprises three structural segments, as shown in Figure 6: i) the bottom 108 m-high segment is made of reinforced concrete, ii) the middle 60 m-high segment is made of steel, and iii) the top 22 m-high segment is made of glass fiber reinforced plastic (GRP). All three structural segments are fixed to each other through bolts at the interfaces. The reinforced concrete segment is tapered and has a circular hollow section with a variable thickness ranging from 45 cm at the ground level to 27 cm at the interface with the middle (steel) segment. The maximum outer diameter of the concrete segment is 8.6 m (at the ground level) and the minimum diameter is 4.8 m (at the interface with the middle segment). The steel segment is divided into three subsegments with constant circular hollow sections: one with a diameter of 3.0 m at the interface with the concrete segment, followed by one subsegment with a diameter of 2.0 m, and ending with a subsegment of diameter 1.8 m. The wall thickness of all subsegments is 2 cm. The transition from one subsegment to the next subsegment is achieved through short tapered steel sections. The GRP segment has a constant 2.5 cm thick circular hollow section with an outer diameter of 1.6 m. A TMD is installed at the top of the GRP segment, consisting of a steel-profile ring filled with concrete and suspended from the top of the tower. The TMD is connected to the tower shell through spring-damper elements.

As can be seen in Figure 6, the SHM system installed in the telecommunication tower consists of accelerometers, temperature sensors, and anemometers. For applying the TMD performance assessment approach, the accelerometers of instrumentation level 2 (top) are considered. Specifically, at the top of the GRP segment, two pairs of accelerometers are installed, one pair on the tower shell and one pair on the TMD. The accelerometers of each pair are perpendicular to each other, one measuring along the east-west direction and one measuring along the north-south direction.
In this study, the analysis has been limited to the acceleration response data sets measured in the east-west direction. The application of the TMD performance assessment approach is performed in the following steps:

1. The acceleration response data sets from the two accelerometers, one on the tower shell and one on the TMD, collected in hourly intervals over a period of six months, are divided into 10-minute segments.
2. Each segment is integrated for obtaining the corresponding displacement response data segment using the Newmark-β algorithm.
3. The displacement response data segments are transformed into the frequency domain and the modal peaks are identified.
4. The cross spectra between the displacement response data segments of the tower shell and the displacement response data segments of the TMD are calculated.
5. The phase angles of the cross spectra at the modal peaks are retrieved.

From step 3, a total of four modal peaks are identified, and, subsequently, the respective phase angles of the cross spectra are retrieved in step 5. The results from applying the TMD performance assessment approach over a period of one month are illustrated in Figure 7.

![Figure 7](image-url)

Figure 7: Results from applying the TMD performance assessment approach over a period of one month.

From Figure 7, it is evident that for the first three modal peaks the phase difference between the TMD and the tower is negligible, as \(\cos\theta \approx 1\), which means \(\theta \approx 0\) or \(\theta \approx 2\pi\). Some deviations from \(\cos\theta \approx 1\) for low wind speeds observed in the first three modal peaks are attributed to the high contribution of noise as a result of low structural response. In the fourth modal peak, and for high wind speeds, a considerable deviation from \(\cos\theta \approx 1\) is observed. Based on the pattern formulated previously for the data-driven analysis, it can be conjectured that the tuning frequency of the TMD lies in a frequency band in the vicinity of \(f_4 = 2.04\) Hz. The lack of convergence of the phase angles of the cross spectra to a dominant value at the fourth modal peak is attributed to the highly complex nature of the structural dynamic response under wind-induced vibrations, where the combined effect of phenomena, such as vortex-induced vibrations, galloping and wind turbulence, may interfere with the TMD operation.
5. SUMMARY AND CONCLUSIONS

The adoption of unconventional structural systems in structural design, such as irregular structural systems or systems with high slenderness, may result in unfavorable structural dynamic behavior, characterized by excessive oscillations. For meeting the safety and serviceability requirements, excessive structural oscillations need to be accommodated. To this end, structural subsystems are frequently installed in an attempt to artificially enhance the damping forces for dissipating the oscillations. A specific class of dampers, termed “tuned mass dampers”, have been increasingly employed, owing to the merit of tuning the dampers to target and suppress specific frequency components that are dominant in the structural dynamic response. The design of tuned mass dampers is conducted either at the design stage of the entire structure, using prediction models of the anticipated structural dynamic behavior, or following incidents of violation of serviceability criteria. In both cases, the tuned mass damper (TMD) properties are estimated for a specific structural state. However, due to ageing and structural degradation, assessing the performance of tuned mass dampers at regular intervals is paramount for verifying the proper TMD operation and, thus, for ensuring user safety. This paper has presented a TMD performance assessment approach by applying data-driven analysis on structural health monitoring (SHM) data collected from structures equipped with tuned mass dampers. According to the basic TMD operation principle, the phase difference between the TMD and the structure is expected to deviate from zero and lie in the vicinity of 90°. The pattern of this expected phase difference is sought in a data-driven manner in the cross spectrum between the displacement response data sets derived from integrating acceleration response data sets from the structure and from the TMD.

The TMD performance assessment approach has been validated through field tests on a telecommunication tower equipped with a TMD. Acceleration response data sets from the TMD and from the tower, recorded at the same level as the TMD, have been collected and integrated to obtain the corresponding displacement response data sets. Then, the displacement response data sets have been transformed into the frequency domain, and four modal peaks have been detected through the Fourier amplitude spectra. Next, the cross spectra between the displacement data sets of the tower and the displacement data sets of the TMD have been calculated. Then, the phase angles from the cross spectra at the modal peaks, representing the respective phase differences between the tower and the TMD, have been retrieved. From the results, no phase difference has been observed in the first three modal peaks, which implies that the tuning frequency does not lie in the vicinity of any of the first three vibration modes. In the fourth modal peak, there are noticeable deviations of the phase difference from zero at high wind speeds, indicating that the tuning frequency may lie in the vicinity of the fourth mode. However, since no convergence to a single phase difference value is observed, no clear conclusion on the tuning frequency can be drawn. To enhance the accuracy of the proposed TMD performance assessment approach, complementing the current approach with analysis on complex wind-induced vibration phenomena, which may interfere with the structural response, is necessary and will be part of future research. Furthermore, combining the data-driven TMD performance assessment methods with elements from the theory of tuned mass dampers to extract more informative conclusions on the condition of tuned mass dampers is also foreseen.

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7. REFERENCES


