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RESPONSE OF FLEXIBLE STRUCTURES WITH TUNED MASS DAMPERS ACTED BY WIND

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Abstract: *Different types of structures like chimneys, high rise buildings or large openings floors can be considered as vibration sensitive structures. Beside classical measures applied to decrease the vibration sensitivity, a modern solution to control those vibrations is to endow the structure with special damping devices. This category of special devices can include tuned mass dampers, individually designed for every type of structure. Beside the theoretical background on TMD and wind loads on structures, the paper shows the response of a high rise chimney with TMD, acted by wind. The structure response is obtained from different analyses in time and frequency domain.*

Key words: *wind load generation, vibration damping, harmonic excitation*

1. Introduction

Usually the vibration sensitivity of a structure can be caused by a combination between a low natural frequency and a small damping capacity. For flexible structures, with great heights or openings, the outcome of vibration sensitivity can lead to different results, from simple discomfort of the occupants, the disruption of functionality or, in severe cases, the structural failure due to fatigue phenomenon.

Modern solutions can be applied to reduce the effect of the structure vibrations. In the world, for the last decades, different special devices are used as vibration dampeners. In this category can be found a number of acceleration dependent devices, individually tuned to the dynamic characteristics of each structures for that are used, called tuned mass dampers (TMD)

A theoretical response of a structure endowed with TMD is shown in figure 1.

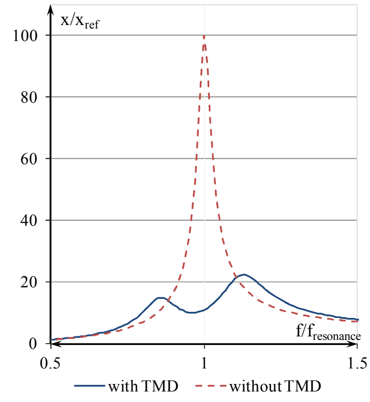

Fig. 1. *Theoretical response of a structure with and without TMD*

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2. TMD characterisation

A TMD device consists, under a simplified form, in a weight, a spring and a damper, all these endowed to a structure, usually in a area with the greatest displacements.

The tuned mass damper characterisation can start from a simple case of a two degrees of freedom system at which the first degree of freedom corresponds to an equivalent SDOF system and the second DOF represents the tuned mass damper [4].

2.1. Mathematical model and optimum characteristics

The motion equation of the two degrees of freedom system is generally depicted in equation (1):

$$M\ddot{X} + F_d + F_e = P. \tag{1}$$

where, M is the mass matrix, X is the absolute displacement vector, F_d is the dampening forces vector, F_e is the elastic resistant forces vector, P is the loads vector.

After the decomposition of the two equation system and naming the parameters according to figure 2, the final equations are shown in expression (2).

$$\begin{cases} m_d(\ddot{y} + \ddot{x}) + c_d\dot{y} + k_d y = 0 \\ m(\ddot{x} + \ddot{u}_0) + c\dot{x} + kx - c_d\dot{y} - k_d y = 0 \end{cases} \tag{2}$$

These equations are the same with the ones introduced by Den Hartog and Rana R. [1] [3] and can be considered as the basic mathematic model of the tuned mass damper system.

The principal dynamic characteristics of the TMD device can be described in terms of the phase ratio, $k = f_d / f$, representing the ratio between the natural frequency of the device and natural frequency of the structure, the mass ratio, $\mu = m_d / m$, and the damping ratio of the device, ξ_d.

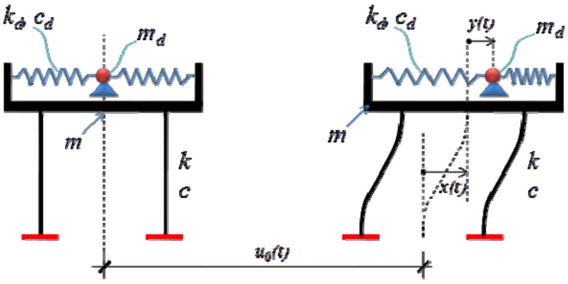


Fig. 2. *SDOF system with TMD*

Depending of the approached used to analyse the equations (2), in the technical literature there are a number of optimal values of some of these characteristics.

For a harmonic excitation, the optimal phase ratio is considered as follows [3]:

$$k_{opt} = 1/(1+\mu) < 1 \tag{3}$$

Also, the optimal damping ratio of the device can be retrieved as a function of the mass ratio [6]:

$$\xi_{d\,opt} = \sqrt{\frac{3\mu}{8(1+\mu)^3}} \qquad (4)$$

2.2. Design principles for tuned mass dampers

Starting from the equation (2) and considering that the structure is acted by an harmonic excitation, different correlations between the behaviour of the structure and the dynamic characteristics of the TMD can be made, related to the optimal values.

Should be noted that a major influence in the response structure with TMD it is represented by the structure own damping capacity. For example it was considered that the SDOF system is represented by a steel structure, for which was considered a damping ratio of 2%.

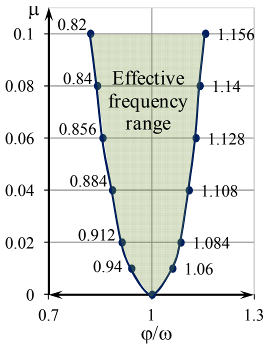
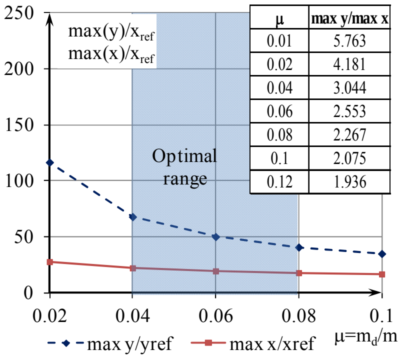


Fig. 3. *Frequency efficiency interval related with the increase of the mass ratio*

The frequency efficiency interval can be enlarged by an increase of the mass ratio, as is shown in figure 3.

Beside the narrow effective frequency range, small mass ratio conducts to larger displacements of the TMD, as depicted in figure 4. On the other side, a greater additional mass means a greater vertical load on the structure, for which the reduction of the displacement amplitude is no longer significant. As one can observe from figure 4, and also the technical reviews shows that, the usual interval of mass ratio is between 4% and 8% from the mass of the structure [6].



μ	max y/max x
0.01	5.763
0.02	4.181
0.04	3.044
0.06	2.553
0.08	2.267
0.1	2.075
0.12	1.936

Fig. 4. *The influence of the mass ratio on structure and TMD displacement*

The deviation from the optimal phase ratio, shown in figure 5, can conduct to greater displacements of the structure, meaning a smaller reduction of the displacement amplitude related to the structure without TMD.

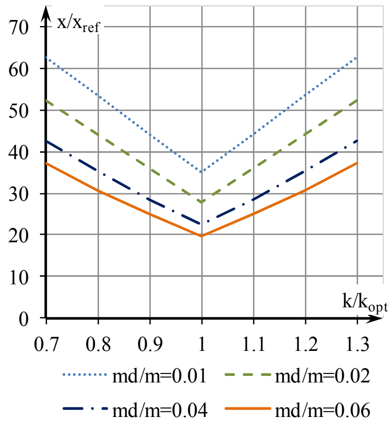


Fig. 5. *The impact of the deviation from the optimal phase ratio*

Small damping ratios of the TMD, related to the optimal value from equation (4), can conduct to a large increase of the TMD displacement, compared with the increase of the structure displacement, as shown in figure 6. Using damping ratios greater than the optimal value conducts to almost null differences in structure displacement, but a small reduction of the TMD oscillations.

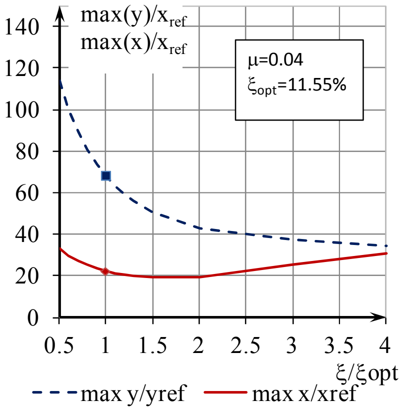


Fig. 6. *Structure and TMD displacement related with de TMD damping ratio*

3. Wind loading

As previously stated, tuned mass dampers are acceleration dependent devices.

In this case, a time-history analysis is required to evaluate the behaviour of the structure with TMD

Usually, the wind action design codes, provides the wind loading as a static force acting on the structure, equivalent to the extremes effects of a turbulent wind.

For numerical analysis, two methods can be used to provide a time-history record of the wind force.

First method presumes the use of real records of the wind pressure or velocity individually determined for a certain site or provided by specific wind data bases.

The second method that can be used, it is to artificially generate a wind velocity record.

It is used to describe wind velocity pressure and forces, as stationary random processes, accepting that due the complexity of nature, the forces generated by wind action can not be described or predicted deterministically. However, by using probabilistic average quantities, such as standard deviation, correlation and spectral analysis, one can describe the main characteristics of both excitation force and the structural response [5].

Thus, wind velocity, pressure on the structure and the structural response being treated as stationary random processes, it can be considered a separation of the averaged values, over a period of time, to fluctuating components of the same quantities.

Such separation can be described mathematically by the following general relation

$$A(t) = A_{average} + A_{fluctuating}(t) \tag{5}$$

where $A_{average}$ is the average value of the action and $A_{fluctuating}(t)$ is the time-history of the fluctuating component.

Fluctuating component of wind is modelled as a stationary random process of zero mean, considering Gaussian distribution of generated values.

In figure 7 are shown the fluctuating components of the wind velocity, at different highs of a structure, used later in the case study. The time-history records were generated with an online wind generation software "NatHaz On-line Wind Simulator (NOWS)" [7]

Using the calculation relations for global assessment of the wind force, described in norm NP 082-04 [8], and neglecting the factors that take into account the shape of the structure and the variation of average velocity on the height of the structure (which is generated automatically by the software), can be considered that the time variation of the total force wind to level *z*, $F_z(t)$, is described by the equation (6):

$$F_z(t) = F_{avg,z} + F_{f,z}\ (t) = .612 A_{ref,z}\ U^2_{avg,z} + \\ + 0.612 \cdot A_{ref,z} U_{avg,z} U_z(t) \qquad (6)$$

where, $F_{avg,z}$ is the average wind force al z level, $F_{f,z}\ (t)$ is the fluctuating component of the wind force at z level, $A_{ref,z}$ is the reference area of the structure at z level, $U_{avg,z}$ is average wind velocity al z level and $U_z(t)$ is fluctuating wind velocity at z level.

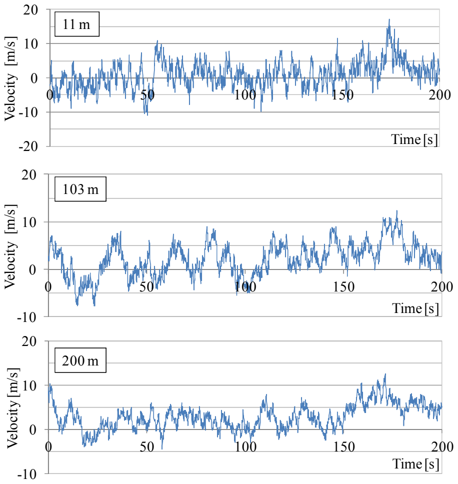


Fig. 7. *Wind velocities at different levels of the structure*

Another approach for the wind response of the structure is an analysis in the frequency domain.

In some cases, the wind induces in the structure vibrations with narrowband frequency. In this case, an example is the response of structures with low vibration frequencies, on the direction parallel to wind. For such cases, the stresses variation over time can be considered as a quasi-sinusoidal variation with randomly varying amplitudes [5].

Based on previous observation, one can consider that the most adverse response shall be recorded when fluctuating component of wind force at level z of the structure is described by a harmonic function with a frequency near the natural frequency of the structure, according to the following relationship:

$$F_{f,z}(t) = F_{fz}\ sin(\omega_w t + \varphi) \qquad (6)$$

where F_{fz} is the fluctuating component amplitude of the oscillation, $\omega_w = 2\pi f_w$ is the pulsation of the wind oscillation for a frequency f_w and φ is the phase shift of the oscillation.

3. Study case

To study the behaviour of TMD devices in structures sensitive to wind loads the study case was made on a concrete chimney with 200 m height [2]. The study aimed the comparison of the structure response, with and without TMD, to wind action.

3.1. Structure description and modelling

The analysed chimney is made from reinforced concrete, with a 200 m height and the shape of the structure like a truncated cone with variable wall thickness (figure 8). The class of the concrete is C25/30.

The foundation and the upper chimney structure consists of a series of cylindrical and conical shapes. The foundation depth is considered from -11.20 m to -2.70 m. The base section, from -11.20 m to -8.00 m, is shaped like a cylindrical tube with an outside diameter of 32.64 m. The next section, between the depth -8.00 m and +27.00 m level, is a truncated cone with the upper outside diameter of 15.86 m. Between +27.00 m şi +33.00 m the structure is again a cylindrical tube. The wall, for all these three sections, has a 50 cm thickness.

The last variation of the shape is from +33.00 m to +200.00 m. The wall has a variable thickness from 50 cm to 18 cm, at the top of the chimney. The thickness varies with 2 cm at every 10 m.

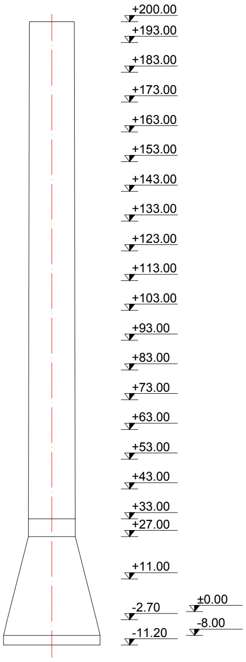


Fig. 8. *Concrete chimney used for the study case*

The numerical model of the chimney, shown in figure 9, was made with shell finite elements, in SAP2000. The model complies with the chimney wall thickness variation.

As result of the height of the chimney structure, the restraints section was considered at -9.00 m, instead of the ground level section, at ± 0.00 m .

To simplify the modelling process, the wind load was applied in the same cross-sections as the change in thickness of the wall. In this way, 20 cross-sections where considered for the wind load.

3.2. Wind force distribution

For the time history analysis a dataset of velocities records for wind fluctuating component was generated, for every level of calculating sections of the chimney.

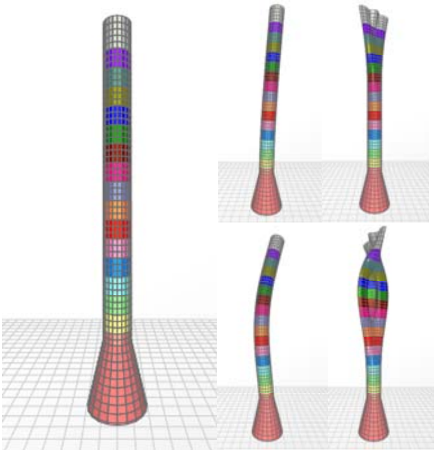


Fig. 9. *Finite elements model of the chimney and typical vibration modes*

In the same time were generated also the values for the average wind velocities. For three sections, in figure 7 is shown the variation of the wind velocity.

Using the procedure described in the previous chapter, the global wind forces where obtained. To mobilize the whole mass of the chimney the distribution of wind force along the perimeter of the cross-section was chosen according with NP 082-04 [8], and is shown in figure 10.

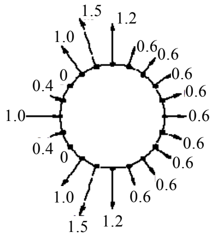


Fig. 10. *Distribution of wind force along the perimeter of the cross-section*

For the frequency domain analysis, starting from the values generated by the NOWS software [7], the fluctuating wind component was considered as a single frequency harmonic oscillation with a random phase shift of $\pi/2$ at different z levels, but with the same amplitudes and values as the fluctuating forces and, respectively, the average forces

3.3. TMD modelling

Following the idea of minimizing additional weight that increases the vertical load on the structure, a TMD with a mass ratio of 2% was chosen, the mass was about 187 tons. According to the natural period of the structure equal to 2.99 s, resulted form modal analysis, for the TMD device was considered a vibration period of 3.05 s and a damping ratio of 8.4%, according to the following relationship:

$$\begin{aligned} &\mu = m_{TMD}/m_{str} = 9371/187 \cong 0.02 \\ &f_{TMD} = \frac{1}{1+\mu} f_{str} = 0.328 \ [Hz] \\ &\Rightarrow T_{TMD} = \frac{1}{f_{TMD}} = 3.05 \ [s] \qquad (6) \\ &\xi_{TMD,opt} = \sqrt{\frac{3 \cdot 0.02}{8(1+0.02)^3}} = 8.407 \ \% \end{aligned}$$

For modelling, a infinitely rigid beam is used. The beam is circular shaped, in plane and is located at the top of the chimney. The beam is supported by 20 link type elements. The additional mass is distributed along the length of the beam.

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3.3. Numerical analysis results

Comparison of the results of time-history analysis was done in terms of displacement (figure 11) and acceleration (figure 12) at the top of the structure. All quantities were considered in the direction of the wind, making a comparison between response of the structure with and without TMD.

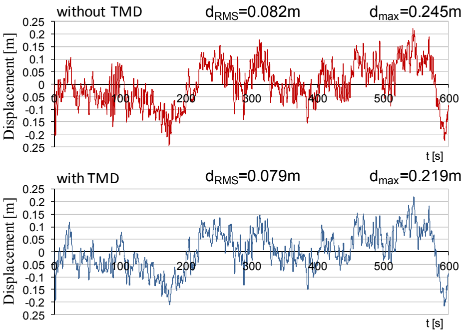

Fig. 11. *Displacement at the top of the chimney subjected to the wind generated action*

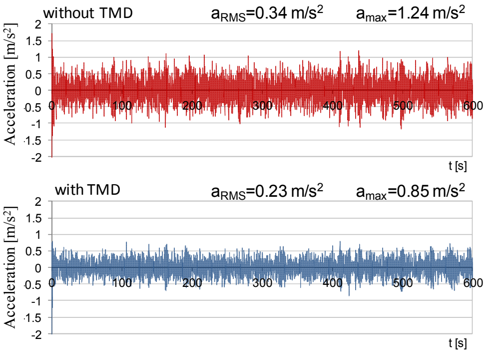

Fig. 12. *Horizontal acceleration at the top of the chimney subjected to the wind generated action*

For the frequency domain analysis the displacement response curves were obtained, also for the structure with and without TMD.

In figure 13 are depicted the normalised displacement response curves for a point at the top of the chimney.

4. Conclusions

From the numerical analysis can be highlighted a few observations.

In the case of the action of the generated fluctuating force, the TMD device with an mass ratio of 2%, has a small reduction of the peak of the fluctuating displacement values of the structure up to 4%, for the average amplitude, respectively, of 10.6%, for maximum values. A significant reduction is recorded for peak accelerations. This is approximately 30%.

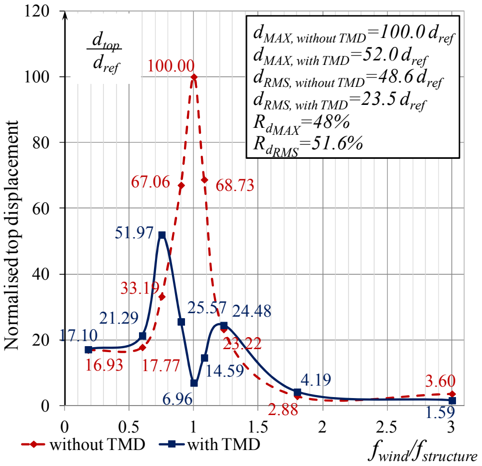

Fig. 13. *Normalized displacement response spectrum at harmonic excitation of the chimney with and without the TMD (d_{ref} = 0.119 m)*

While the TMD has a narrow band of frequencies for which the device is tuned to the structure, it is still effective when the prevailing wind vibration frequency coincides with the frequency of the structure. In this case was recorded a reduction of the amplitude over 90 %.

In general, the wind, having a large frequency range, both in the efficiency bandwidth and beyond, a comparison of the average displacements for the structure with and without TMD, can give an order of magnitude about the mitigation capacity of the device. In this case the mean attenuation of the oscillations is closer over 50%.

Given the observations above, notwithstanding the fact that the case study presented in this paper does not cover a sufficient number of parametric analysis, it can be concluded that the TMD device can be an effective way to mitigate vibrations induced by wind action.

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