# The Constructions Vibration Control by Tuned Mass Dumper

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## Abstract

In this paper the Vibration Control by the Tuned Mass Dumper (TMD), in important and symbolic constructions, are illustrated. Some TMD optimization procedures are discussed for different types of constructions (new tall buildings, an existing masonry chimney and a new footbridge) for the seismic and wind actions. By the TMDs, the structural behaviors improvements are shown, either for the Ultimate Limit State (ULS), either for the Serviceability Limit State (SLS). To appreciate the structural improvements by TMD installation, for each types of constructions, structural analysis by finite elements model (FEM) are carried out with and without TMD. Finally, for each TMDs, a design hypothesis are showed pointing out the installation procedures and the related costs.

**Keywords:** vibration control, TMD, tall building, chimney, footbridge, wind and seismic action mitigation

## 1 Introduction

Tuned mass damper (TMD) solutions to mitigate the effects of the wind and seismic loads on the structures are analyzed for three important constructions: a new tall building, an historical chimney and a footbridge. In the tall building case the mitigation regards the serviceability behaviour; in the footbridge case the TMD works either to control the vibrancy and to reduce the seismic effects; instead, in the historical chimney case, the TMD represents a structural improvement (in the following for the seismic action).

## 2 Wind effects mitigation for a tall building

To improve the structural behaviour under the seismic and wind loads in tall buildings a TMD installation could be a valid solution, specially if the tall building has a symbolic shape characterized by an high geometrical slenderness (λ) value (λ is the ratio between the building height and the minor side of the building plant). In some new tall building the architectural solution provides a λ value more than a limit range λ=5÷7.
The passive mass damper reduces the dynamic effects specially due to the wind actions so its contribution has to be considered in the serviceability limit state (SLS); in the ultimate limit state (ULS), using the Eurocode (EC) or national

annex, the structural verifies have to be carried out without the damping due to the TMD. In some cases the TMD is used coupled by particular structural solutions, like braces trusses positioned slightly more over the half of the building height. In the following case, a 220m height tall building with 74m x 28m rectangular section (λ=7.85), is afflicted by dynamic wind effects for some reasons (like the vortex shedding). The building structure is characterized by: two lateral cores in high strength reinforced concrete (HSC, C70/85) located about 50m mutual distance, beams and columns in HSC, and 0.25m post tense concrete slab deck. In the wind analysis, for a return period ($T_R$) $T_R$=50 years, the directional wind velocity (v) is taken in account. Considering the building located in a circumference subdivided in 16 parts (representing the 16 wind direction): the X longitudinal building axis (along the 74m length side) is directed like the 0°-180° wind direction and the Y transversal building axis (along the 28m length side) is directed like the 90°-270° wind direction. The maximum wind velocity ($v_{max}$) corresponds to the 292.5° wind direction. By the wind tunnel tests and the application of the High Frequency Force Balance (HFFB) [1], [2], [3] procedure -that includes a finite element model (FEM) implementation and an eigenvalue analysis - the base shear (V), the base moment (M) and the top acceleration ($\alpha_{max}$) is evaluated for all the wind directions. The first three building frequencies (f) are $f_1$=0.14Hz, $f_2$=0.15Hz and $f_3$=0.26 Hz.

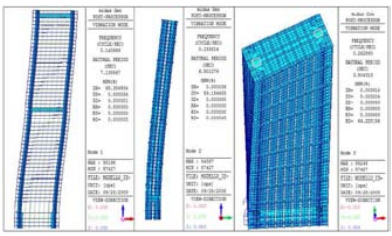

Figure 1. *main vibration modes of the building*

A results in terms of base shear V, obtained with the Italian CNR code and HFFB procedure, are shown in the next Table 1. The HFFB considers the dynamic effects for all the directions not only for the along and across wind direction - about that, the Eurocode (EC) and Italian codes have some limits-.

Moreover, knowing the applicable limits of the TMD in the last two top floors of the building (e.g. limits due the mass invasiveness and its structural support), by the HFFB, the V,M and $\alpha_{max}$ are evaluated considering a total damping coefficient (ξ) value in the range ξ=1÷4%.

*Table 1. base shear V comparison*

| | wind direction | by CNR [kN] | by HFFB [kN] |
|---|---|---|---|
| 0°-180° | along-w | 12211 | 10816 |
| | across -w | 10079 | 33863 |
| 90°-270° | along-w | 31881 | 33597 |
| | across -w | 13322 | 15499 |
| 292.5° | along-w | --- | 27955 |
| | across -w | --- | 46052 |

The ξ value is the summation of the concrete structure damping ($\xi_s$) at the SLS ($\xi_s$=1%) and the TMD damping ($\xi_{tmd}$) evaluated $\xi_{tmd}$=3%. The TMD considered could be represented by a tuned mass damper or by liquid tuned mass damper. For the SLS comfort building conditions, by the TMD, the $\alpha_{max}$ has to be minor than the limit acceleration ($\alpha_{lim}$) suggested in literature [3] for tall buildings using for office (in $T_R$=10 years case, $\alpha_{lim}$ = 0.25 m/s$^2$ ). In order to consider acceptable the HFFB results for different ξ values, a comparison in terms of acceleration between wind time history analysis (TH) and HFFB application is carried out. In the TH, the time histories (in terms of wind pressure) are applied along the floors of the FEM in two condition: without TMD (ξ=$\xi_s$=1%) and with TMD (ξ=4%). The results for TH and HFFB are shown in the next Table 2, in $T_R$=10 years case.

*Table 2. $\alpha_{max}$ in TH and HFFB analysis*

| | $\alpha_{max}$ in TH [m/s$^2$] | $\alpha_{max}$ in HFFB [m/s$^2$] |
|---|---|---|
| ξ=1% | 0.45 | 0.47 |
| ξ=4% | 0.25 | 0.24 |

Moreover, in the TH's FEM, the TMD located at the top floor is implemented by a nodal mass linked to the center of gravity of the top last floor by general links (represented a spring and dashpot in parallel). The links' characteristics come out from [4], [5]. Another damping solution to control the vibration is the viscous damper (VD) solutions positioned in the first 20 cores (from the base), limiting the invasiveness in terms of commercial surfaces occupancy; nevertheless the oblique positioning limits the inner layout. By VD, the acceleration decreases from 0.40m/s$^2$ to 0.32m/s$^2$ in the time histories analysis.

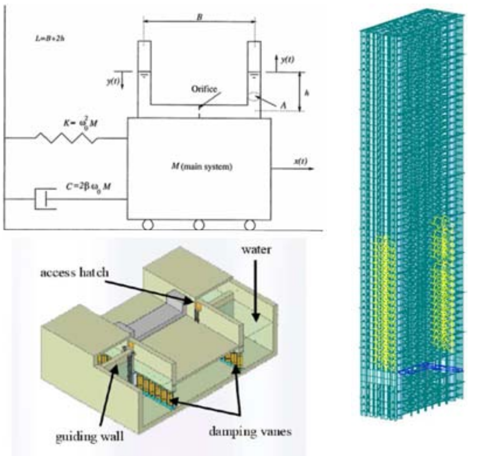


Figure 2. *LTMD case(left), FEM with VD(right)*

The active control systems, also called Active Mass Damper (AMD) differ from the passive ones because of the additional energy supplied from the outside to the structure, to reduce the dynamic response [6]. This control is achieved through three components:

1) sensors, installed in a convenient location, which measure alternately or simultaneously the external excitation and structural response;
2) computers, which analyze the measurements of the sensors in accordance with appropriate algorithms derived from the theory of optimal control and define control actions;
3) actuators that apply forces to the structure of control established by computers.

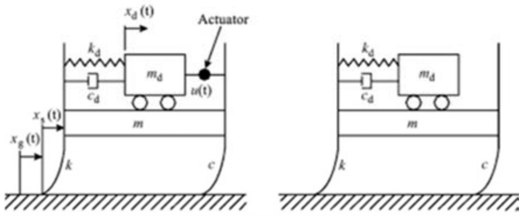


Figure 3. *Differences between Active Control Systems (AMD – left side) and Tuned Mass Damper (TMD – right side).*

To evaluate AMD characteristics is possible to refer [6].

## 2.1 Types of actuators

Depending on the type of actuator used the active control systems are usually divided into two main groups:

a. tendon systems;
b. active mass systems.

The tendon systems (Active Tendon Control System), proposed by Freyssinet in 1960, consists in steel cables, such as those used for pre-stressed concrete, linked to the structure and by pulleys, an electro-hydraulic mechanism which controls the state of tension. Alternatively, the active mass systems (or Active Mass Damper) provides that on the structure a mass similar to the TMD is installed. Unlike the latter, in AMD the active mass must be phased to the frequency of the vibration mode that has to be deadened. In this way, the mass undergoes the control forces produced by an electro-hydraulic mechanism. It's clear the mass is no longer a passive element, but a tool able to be activated, according to the situation occurring from time to time. The next illustrates the scheme of a 2D AMD.

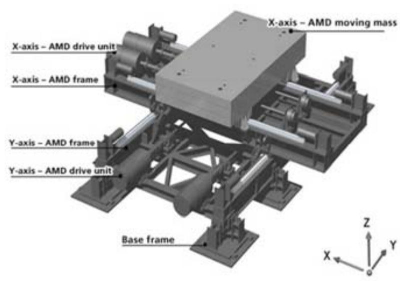


Figure 4. *Active Mass Damper by IHI Co. LTD.*

### 2.2 Costs

Typically, the AMD systems are not interesting solutions for most of the structural problems, especially for heavy structures. The main reason for the statement is due to the need of installing more sophisticated technology, which results in increasing costs and energy consumption. For this the actual applications of this type of control in civil engineering structures are few. However, AMDs can be an interesting solution and alternative to passive control systems (TMDs) for small structures, such as walkways or small and slim bridges. In this case, active systems have some significant advantages. In fact, AMD systems can adapt better to the variations of the dynamic parameters of the structure and may be more effective in controlling small vibrations.
It is also important to consider that AMDs must not be tuned to all of the natural frequencies of the structure because they work with the measured response of the system. Furthermore, in case of small structures, requiring low control forces, there is the possibility of using some electric actuators that can minimize costs, maintenance and noise.

## 3 A symbolic construction seismic improvement by TMcS

A tuned mass control system (TMcS) can be considered to improve the structural behaviour under the seismic (or wind) load in existing constructions. The structural improvements due to a TMcS installation in a historical chimney are here discussed. The masonry chimney, having 50m height, is characterized by two cylindrical skins connected themselves by n.°8 meridians and n.°11 parallels.

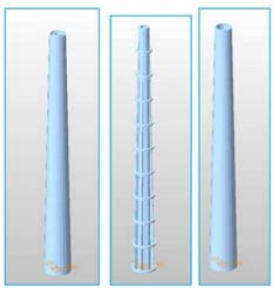

Figure 5. *structure of the chimney (inner and outer skins, meridians and parallels system)*

All the structural parts having 0.27m thickness. For the finite element model (FEM) implementations, the masonry modulus of elasticity (E) and the ultimate compression strength value ($f_d$) - deduced by flat jacks tests- are E = 12185 MPa and $f_d$ = 4.96 MPa. Applying the Italian Structures Code (NTC), $f_d$ value decreases to the reduced value $f_{dc,rid}=(f_d\ /c_1)\cdot c_2$ = 3.55 MPa, considering $c_1$=1.2 ($c_1$ depends by the tests number carried out on the structure) and $c_2$ = 0.86 ($c_1$ depends by the geometrical slenderness and the chimney's vertical loads eccentricity). The $f_{dc,rid}$ will be used in the structural verifies by NTC in comparison to the maximum compression stress ($\sigma_{max}$) at the base section. A first chimney FEM model (FEM_1) is implemented by using beam elements and a perfect join base boundary condition. The loads considered are the dead loads and the seismic action (by the seven spectrum-compatible accelerograms) combined like mentioned in the NTC. In the FEM_1 an eigenvalue analysis is also carried out to identify the main vibrational mode shapes with the relative percentagse of the mass associated to each mode shapes. By the seismic analysis it's possible to evaluate the values of stresses actions, in terms of base shear (V), base bending moment (M) and top displacement (η). By the eigenvalues analysis, it's possible to obtain the optimum damper mass (like a percentage of the mass associated to the vibrational mode to "shoot down") and, consequently, the stiffness and damping values. Carrying out the seismic analysis, the stresses effects on the chimney without the TMcS are: V = 451kN, M = 8120kNm and η = 0.04m (the chimney self-weight are $F_z$ = 4225kN).
Moreover the eigenvalue analysis shows the first and second vibration modes (respectively $T_1$ and $T_2$) are $T_1$=$T_2$=1.08s and the related masses ($m_1$ and $m_2$) are $m_1$ = $m_2$ = 0.56%. The following relations ([7] and [8]) are used to evaluate the better TMcS characteristics, starting from a ratio ($\mu = m_{TMcS}/m_{str}$ ) between the TMcS mass ($m_{TMcS}$ ) and the main structure mass ($m_{str}$) to define: the optimal coefficient of frequencies ($\alpha_{opt}$), the optimal equivalent viscous damping ratio ($\xi_{opt}$) for the TMcS, the TMcS horizontal stiffness ($k_{TMcS}$) and the TMcS horizontal damping coefficient of each viscous damper ($C_{TMcS}$). Instead, to control the wind effects it's possible to refer [9].

$$\alpha_{opt} = \left(\frac{\sqrt{1-0.5\mu}}{1+\mu} + \sqrt{1-2\xi^2} - 1\right) - [2.375 - 1.034\sqrt{\mu} - 0.426\mu]\cdot\xi\cdot\mu - (3.730 - 16.903\sqrt{\mu} + 20.496\mu)\cdot\xi^2\cdot\sqrt{\mu} \ ; \qquad (1)$$

$$\xi_{opt} = \left(\frac{\sqrt{3\mu}}{8(1+\mu)(1-0.5\mu)}\right)^{0.5} + (0.151\xi - 0.175\xi^2) + (0.163\xi + 4.98\xi^2)\cdot\mu \ ; \qquad (2)$$

$$k_{TMcS} = m \cdot \alpha_{opt}^{\,2} \cdot \omega_s^{\,2} \ ; \qquad (3)$$

$$C_{TMcS} = 2\xi \cdot (k_{TMcS} \cdot m)^{0.5} . \qquad (4)$$

Before to calculate $\alpha_{opt}$, $\xi_{opt}$ and the $k_{TMcS}$ it's necessary to optimize $\mu$, considering the $\mu$ value is usually included in the 1.5%÷4% percentage range. So, applying the predicted seven spectrum-compatible accelerograms, varying the $\mu$ value, the time histories of the stresses effects on the structure are studied, identifying the $\mu$ value to minimize V, M and $\eta$. In this case the better improvements are obtained by $\mu$=0.032 (with $m_{TMcS}$=75.71kN, $k_{TMcS}$=143.68kN/m and $C_{TMcS}$ =7.53kNs/m) and so, applying the site seismic spectrum, the stresses effects (V,M and $\eta$) under the seismic action decreases like shown in the following Table 3 (in terms of $\Delta V$, $\Delta M$ and $\Delta\eta$ percentage).The seismic improvements are evaluated applying different site spectrums obtained using the ductility structure factor (q), defined in the NTC, q=1 or q=2. When q=1 the seismic analysis is conduced hypothesizing the damping structure coefficient is the TMcS damping (in this case, 9.6%), like the NTC indicated for structures isolated at the base.

*Table 3. Seismic improvements by TMcS*

| | ΔV [%] | ΔM [%] | Δη [%] |
|---|---|---|---|
| q=1 | 17 | 30 | 11 |
| q=2 | 32 | 43 | 28 |

By the eigenvalue analysis, implementing the TMcS in the FEM_1 (like a nodal load converted in mass, linked to the node of the chimney by a spring and linear dashpot), the vibration mode shapes change; in fact, by the TMcS, the first four modes involve only the TMCs mass and, from the fifth mode, the chimney's structure begins to be involved; so, respect of the first vibration mode, the resonance curve of the chimney is modified by the TMcS vibration modes introduction.

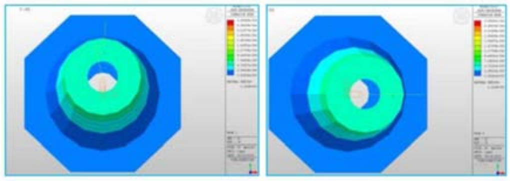

*Figure 6. I and II Vibration Mode Shapes (chimney and TMD)*

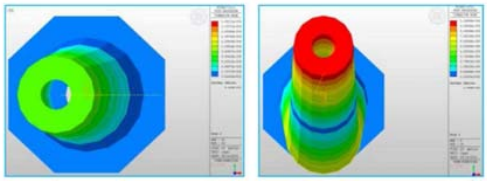

*Figure 7. IV and V Vibration Mode Shapes (chimney and TMD)*

In the preliminary structural verifies in terms of compression stress, it's fundamental to consider the no tensile resistance of the masonry; thus the verification base section is partialized. In this case (Figure 8), because the section having a symmetry axis, to evaluate the $\sigma_{max}$, if the seismic action gives a center of the pressure (C) located on one of the symmetry axis, the neutral axis (n) direction is noted; so the distance (X) - between C and n - has to be determined. X value has to nullify the following relation where: $S_x$ is the no partialized section static moment (respect to n) and $J_x$ is no partialized section inertia moment (respect to n)

$$S_x \cdot X - J_x = 0. \qquad (5)$$

Estimated n - from n it's possible to measure the maximum distance ($d_{max}$) to the generic compression resistance fiber-, the $\sigma_{max}$ is given by the following relation (where N is the axial load):

$$\sigma_{max} = (N/S_x) \cdot d_{max} . \qquad (6)$$

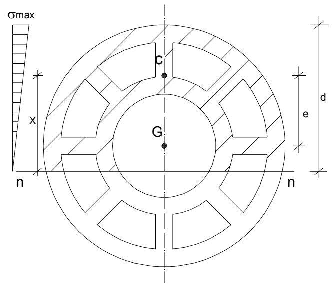


*Figure 8. base verify section*

Moreover, to confirm the improvements of the FEM_1, the second FEM_2 is implemented using solid elements for the chimney structure, with beam and plate elements for the TMcS.

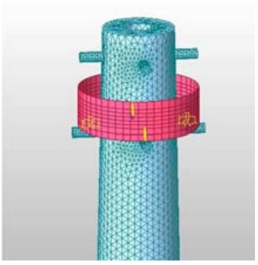

*Figure 9. TMcS system in FEM_2*

*Table 4. $\sigma_{max}$ in FEM_1 and FEM_2*

| | FEM_1 No TMD **[MPa]** | FEM_1 with TMD **[MPa]** | FEM_2 with TMD **[MPa]** |
|---|---|---|---|
| q=1 | 5.1 | 3.4 | 3.3 |
| q=2 | 4.3 | 2.8 | 2.5 |

The seismic action is applied by time history and the masonry is characterized by a non-linear behavior. The effect of nonlinearity to the behavior of masonry must be accurately taken into account analyzing the ultimate behavior of masonry chimney. The main concept of the nonlinear masonry model, adopted in the FEM implemented by the software MIDAS GEN, is based on the theory of J.S. Lee & G. N. Pande. In FEM_2 the maximum compression stress (extracted from the stress time history for the solid elements close the base) is very similar to the maximum value obtained using the FEM_1.

A results comparison using FEM_1 and FEM_2, is summarized in the previous Table 4.

Furthermore it's necessary evaluate the mass acceleration under the seismic action. In the acceleration time history obtained from the spectrum seismic compatible analysis: the maximum value $a_{max}$ is $a_{max}$ = 6.45m/s$^2$ (= 0.65g), lower than the acceptable value $a_{lim}$, normally fixed in $a_{lim}$ = 1g.

To practical realize the TMcS, it's useful underline the TMcS could be install external the chimney. In a steel chimney it's very easy to realize the oscillating mass and its positioning is easy too, for example using a welded steel profiles system externally welded to the chimney's body; in a concrete or masonry chimneys, the dimensions of the oscillating mass could lead to large cross-section beams not easy to assemble. That is especially true in masonry chimneys where it's necessary not affect the actual stresses state of the masonry. The mass support system could be characterized by a steel structure to realize on two levels: the first (upper) level working for the positioning of the ropes to which the circular steel mass of the damping system is attached; the second (lower) level supporting the viscous-dampers.

The circular steel mass could be hanged on the ropes and, at the same time, supported by the dampers. The number of the dampers is three or four; for example, the two levels could be positioned at about 3.00m mutual distance; it depends by the ropes length (also variable between 0.80-1.40m) and it has to fix by dynamic tests to investigate the real vibration modes of the chimney. The first level of the structure could be characterized by an inner part realized by a commercial cross welded hollow circular profiles (e.g. 193.7mm diameter and 5.4mm thickness). At the end of each cross' arm the hollow circular steel plates are welded. Each steel plate is bolted to another steel circular plate welded to hollow circular profile that reaches a length of 700mm from the external chimney surface. Where the steel profiles cross the masonry square holes, to protect the masonry, in a steel box realized by n.6 4mm thickness steel rectangular plates is previously welded on profiles. To perfectly close the square holes it's necessary use a suitable premixed cement mortar. The second level is equal to the first one but the cross' arms reach the necessary length from the external chimney surface; at the end of each n.°4 arms the viscous-dampers are positioned on a n.°6 steel welded plates system characterized by n.° 5 6mm thickness vertical rectangular plates (sides 230 mm and 238 mm) and n.°1 8mm thickness horizontal plate. The first and second levels are also connected by n.°4 AISI 316 cables linked at the inner hollow circular plate. A 3d representation of the TMCs structure is represented in the next Figure.

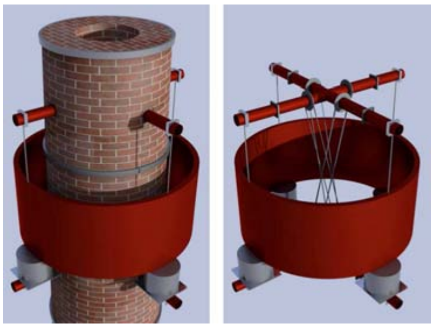

*Figure 10. TMcS hypothesis*

To control the displacement of the mass a suitable set of springs (for example n.°6 for each damper, like shown in the next Figure) and some rubbers is positioned between the inner surface of the circular mass and the external surface of the chimney (for the springs, see also next Figure 11).

*Figure 11. springs set (from Gerb)*

## 4 A footbridge improvement by TMD

The footbridge, length 67.6 meters, is characterized by a steel deck (Figure 12). The conceptual design is represented in Figure (13).

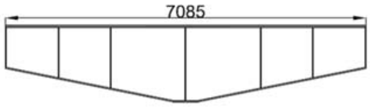


*Figure 12 steel deck section (Unit: mm)*

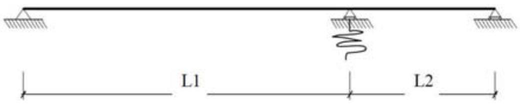


*Figure 13 footbridge conceptual design ($L_1$ = 45.9 m , $L_2$ = 21.7m )*

In this case, the damper is installed inside of the deck near the intermediate pier. The footbridge dynamic behaviour is evaluated by a first simple FEM model: the deck and the intermediate pier are modelled using beam elements; the bearings are represented by linear spring and TMD is a nodal mass connected at the deck with a rigid link.

*Figure 14 footbridge FEM model*

The advantages of TMD use, in terms of stresses inside the deck is very significant because the reduction is approximately 50%.
By the TMD the first and second vibrational modes are modified respect to the only footbridge configuration, involving only the mass dumper (with TMD $T_1$ = 0.95 s, without TMD $T_1$ = 0.31 s)
A vibrancy analysis is conducted to evaluate the footbridge behaviour in the particular overcrowding situation. To carry out the vibrancy analysis is possible observe some methods [10], in this case the Reither-Mesiter-Lenzen (RML) method is used. Noted the vibration mode shapes of the footbridge, in the two cases A and B (A: without TMD; B: with TMD), the horizontal

elements of the second FEM (implemented by plate elements) is loaded by self weight, dead loads and dynamic nodal loads. The dynamic nodal load (descripted by a function having a zero force at time 0sec and force F=0.90 kN at time 1sec) is applied on the mesh nodes having a mutual distance of 0.68m (that could represents a predicted overcrowding situation). In the case A the maximum displacement ($\delta max$)is 0.65mm and in the case B $\delta max$ = 0.29mm (the vibration mode is f = 3.22Hz). So, using the RML diagram, in case A the vibrancy is very perceptible, instead by the TMD, in case B, the vibrancy is reduced to the lower lightly perceptible state.

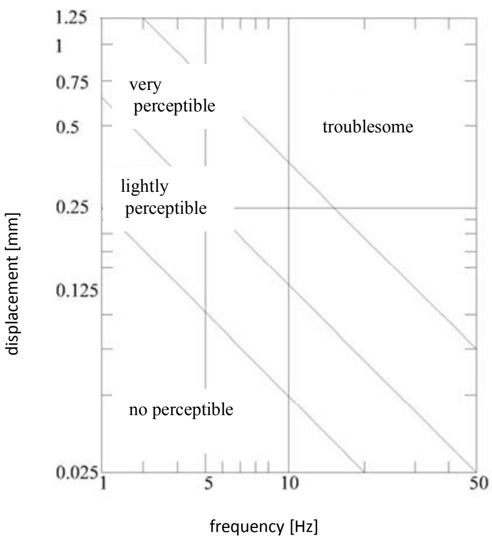


*Figure 15 displacement - frequency*

## 5 Conclusions

In the paper the mitigation of the seismic and wind loads on three symbolic constructions are discussed underlining, for each construction cases, the problems for the serviceability and ultimate limit states. Some solutions are proposed to improve the structural behavior thinking about the real possibility to install the TMD (or TMcS) without affecting the layout (in case of tall building and footbridge) or the symbolism. In the chimney case the solution proposed is interesting because it's removable and reversible, unlike the pure structural solutions (by the application of steel elements and reinforced concrete layer to anchor to the structure).

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