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STRUCTURAL VIBRATION CONTROL BY TMD'S USE

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ABSTRACT

The paper refers the use of Tuned Mass Dumper (TMD) in different constructions recently built in Italy to improve their serviceability structural performances. In two footbridges and a new tall building the dynamic behaviors are analyzed in the two conditions: without and with TMD. The structural analysis show the improvements in the Serviceability Limit State (SLS).

In the footbridge cases the use of the TMD allows the control of the deck vibrancy, either in the daily use and in extreme overcrowding case; in the tall building case the improvements are appreciated in relation to a greater comfort for the occupants due a better wind structural response.

Keywords: vibration control, TMD, tall building, footbridges

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1. INTRODUCTION

Some TMD solutions to mitigate the effects of lateral loads and to control the vibrations are analyzed for two kind of constructions very different themselves. In two new footbridges the TMD allows the control of the vibrancy under the pedestrian and crowd loads.

For a new slender tall building case the TMD is used to increase the comfort for the occupants mitigating the accelerations due to the wind (in the SLS), but its application is analyzed also for the Ultimate Limit State (ULS) although the Italian Structural Code doesn't treat the TMD's use for the ULS. In fact, the structural resistance and ductility of the structure for the ULS, in the Italian code has to be entirely attributed to the structure.

2. FOOTBRIDGES CASES

For two different footbridges (in the following called F1 and F2) a vibrancy analysis is carried out considering the pedestrian and the overcrowding loads.

The structural and vibrancy analysis are carried out by the finite element models (FEMs) implementation and for both the necessary eigenvalue analysis are conducted for different loads combinations in the cases without and with TMD.

For the vibrancy analysis under pedestrian load, it is possible to consider some methods summarized in Table 1.

The forces – representing the pedestrian load - to apply in the FEMs (Table 1) are described by time varying functions; their effects are evaluated in terms of acceleration.

The methods applied in the cases here discussed are Bachmann, BS5400 and Allen-Murray. The accelerations due to the application of the forces are valued and compared to the limit accelerations (a_{lim}) having different value for BS-5400, the ONT83 and in the ISO 10137/200.

Table 1: Some pedestrian load evaluations.

methods	function
Bachmann	$F_p(t) = G + \sum_{i=1}^{n} G \cdot \alpha_i \cdot sen(2\pi_i f_p t - \phi_i)$
BS5400	$F_p(t) = A \cdot \sin(2\pi \, \mathrm{f_v} \, t)$
Allen-Murray	$F_p(t) = P(1 + \sum \alpha_i \cdot \cos 2\pi \cdot i \cdot f \cdot t)$
AIJ	$F_p(t) = 11.78 \cdot sen(78.5)$ $F_p(t) = 70.69 \cdot sen(78.5)$

For Table 1, the following notes are useful:

- in Bachmann: G = 800 N, α_i is i-harmonic coefficient, f_p is the “activity” frequency and ϕ_i is the the i-harmonic phase respect the first harmonic;
- in BS500: A = 180 N, velocity v is constant and $v = 0.9 \cdot f_v$ (f_v is the vertical frequency);
- in Allen-Murray: P = 700 N, f is the walking frequency and α_i is the dynamic coefficient of the i-harmonic;
- in AIJ: the walking case is represented by a force of 0.3 kgf·sec applied for 0.04s; the running case is represented by a force of 1.8 kgf·sec applied for 0.04s.

The a_{lim} values, in the considered case, are obtained by the following relations:

$$a_{lim-BS5400} = 0.5 \cdot (f)^{0.5}$$

$$a_{lim-ONT83} = 0.25 \cdot (f)^{0.78}$$

$$a_{lim-ISO10137} = 60 \cdot \sqrt{2} \cdot 0.005$$

By the proposed methods the structural behaviour of the footbridges under the crowd load is studied in terms of frequency. The vertical and horizontal vibration mode shapes are valued and the related frequencies are calculated. Subsequently the frequencies are compared to the limit values proposed like acceptable or in Setra [1] or in some structural international codes. In fact it’s possible to associate the risk of structural resonance (R) to the footbridge’s frequency (f).

For the vertical frequency, Setra proposes:

- an high R value for f = 1.7÷2.1 Hz;
- a medium R for f = 0.9÷1.7 Hz and f = 2.1÷2.6 Hz;
- a low R for f = 2.6÷5.0 Hz;
- a very low R for f = 0÷0.9 Hz and f > 5 Hz.

Again for the vertical frequency, the risk of structural resonance R is associated to other ranges indicated in the following codes and methods:

- EC2, 1.6÷2.4 Hz;
- EC 5, 0÷5 Hz;
- BS5400, 0÷5 Hz,
- Japan code AIJ, 1.5÷2.3 Hz;
- ISO/DIS Standard 10137, 1.7÷2.3 Hz;
- CEB 209 Bulletin, 1.65÷2.35 Hz;
- Bachman, 1.6÷2.4 Hz).

For the horizontal frequency, Setra proposes are:

- an high R value for f = 0.5÷1.1 Hz;
- a medium R for f=0.3÷0.5 Hz and f=1.1÷1.3 Hz;
- a low R for f = 1.3÷2.5 Hz;
- a very low R for f=0÷0.3 Hz and f > 2.5 Hz.

Moreover, EC1 suggests to evaluate the vertical and horizontal footbridge frequencies limiting to indicate acceptable ranges, precisely:

- for the vertical frequency the range is 1÷3 Hz;
- for the horizontal frequency the range is 0.5÷1.5 Hz.

Others practical methods for the vibrancy analysis exist, for example could be mention: the Reither-Mesiter-Lenzen (RML) [2] or the Arcelormittal (AR) [3] methods.

By these two methods it is possible to estimate if the deck vibrancy is acceptable for the occupants; in fact, for the SLS, the vibrational response is defined acceptable by using a correlation between the frequency deck and the maximum vertical displacements under the pedestrian load. It is important to underline the RML method is specifically valid for floor deck vibrancy study, however it is here conduced to evaluate the structural behaviour in a potential overcrowding situation in footbridge case.

In the cases in the following described (called F1 and F2) the behaviours under the pedestrian and overcrowd loads are studied and if the mentioned acceptable limits are not respected (or in terms of acceleration or acceptable vibrancy range), TMD solutions are studied.

For the damper characteristics Den Hartog [4] formulations are used. Estimating the ratio (μ) between the TMD mass (m) and the mass of the main structure involved in the main mode shape (m_i), the optimal damping ratio (ξ_{opt}) and optimal frequency ratio (f_{opt}) are evaluated by the following relations:

$$\mu = \frac{m_{tmd}}{m_i}$$

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

$$f_{opt} = \frac{f_i}{1+\mu}$$

F1 - case

The F1 footbridge connects two buildings renovated for EXPO 2015; it presents a S355 steel structures and its architectural design presents a Vierendeel configurations (30m total length, 5.10m width, 4m roof level height).

The footbridge has two levels: the walkway level and the roof level. The footbridge could be classified III class by the SETRA; for a III class the crowd load is defined by the 0.5 pedestrian/m^2 density.

In the structural analysis, the F1 is implemented by beam and plate elements; in particular the plate elements are used for the lower and upper decks characterized by a trapezoidal sheet with a 4 cm concrete thickness layer.

The footbridge is supported by n.°4 general links; they connect the n.°4 extremity nodes (n.°2 for each sides) of the lower deck to fixed nodes positioned at a distance equal to the supports height (in vertical direction from the deck).

The supports on the left footbridge side are fixed and multi-directional, instead the supports on the right side are movable along the longitudinal footbridge axis and multi-directional.

The dead and live loads are combined with the seismic action in according to NTC 2008. The elastic site spectrum with the ductility structure factor q = 2 is considered. The eigenvalue analysis is carried out in order to obtain the vibration mode shapes and the related involved percentage of the mass (Table 2 and Figure 1).

Table 2: FA - main vibration modes.

mode	frequency [Hz]	mass [%]	main direction [/]
1	2.16	79.85	Z global, vertical
5	6.22	32.79	Y global, horizontal
7	7.40	77.42	X global, horizontal

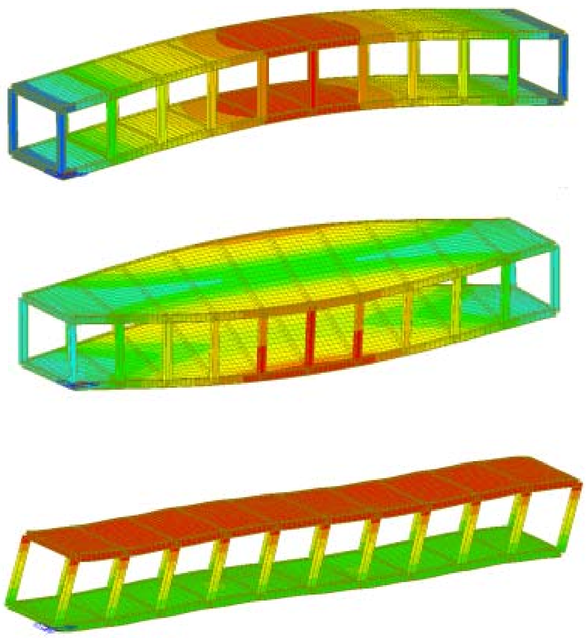

Figure 1. FA footbridge: 1-5-7 vibration mode shapes.

For F1, the analysis under the pedestrian load is carried out by the following methods: Bachman, BS5400 and Allen-Murray.

In the Bachman case, the time history force is applied along the central nodes of the longitudinal axis; in BS5400 and Allen Murray cases, the time history forces are applied in the node with the maximum vertical displacement (under the dead and life loads).

The BS5400 and Allen-Murray [5] forces applied in the FEM are showed in the next Figure 2.

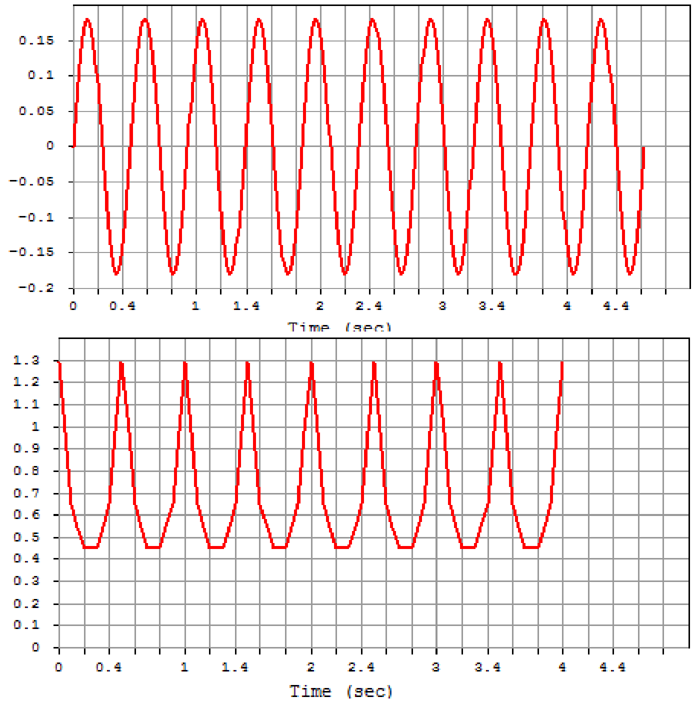

Figure 2. (on the left) Time History force for BS5400; (on the right) time history force for Allen Murray

The accelerations (a_{max}) due to the application of the forces are evaluated and compared to the limit value (a_{lim}). The verifies are positive because a_{max} = 0.25 m/s^2 is minor than: $a_{lim\text{-}BS5400}$ = 0.73 m/s^2, $a_{lim\text{-}ONT83}$ = 0.49m/s^2 and $a_{lim\text{-}ISO1037}$ = 0.42 m/s^2. At the same time, the accelerations calculated for FA by the application of the forces described in BS500 and Alley-Murray are minor than the limit accelerations.

For the the vibrancy under the crowd load, the RML and AR methods are here applied.

By usying RML, dynamic nodal loads (descripted in [2]) are applied on the nodes having a mutual distance of 0.70 m. The maximum displacement (δ_{max}) is δ_{max} = 0.93 mm and the first walkway deck frequency is T_f = 1.08 Hz. From the correlation of δ_{max} and T_f the footbridge shows a very perceptible oscillating move. Thus, to reduce the oscillating effect the TMD's introduction could be valid.

On the contrary, for the vibrancy under the crowd load, different result is obtained by the application of AR method. Considering 1% of structural damping and combining T_f = 1.08 Hz to the modal mass (m) of the floor involved (m =42949 kg, equal to 69% of the 62246 kg total mass) it results OS-$_{RMS90}$ = 0.6, included in the recommended range (0.2÷0.8) for the class C floor retail [3].

Although the different results, to improve the vibrancy behavior of the walkway deck a TMD are studied and optimized considering the mentioned relations. The characteristics of the TMD in terms of mass, stiffness and damping are: m_{tmd} = 6.4 kN, K_{tmd} = 28.33 kN/m, c_{tmd}= 0.68 kNs/m.

In this case, the optimum ratio of frequencies α_{opt} = 0.97 and the optimal equivalent viscous damping ratio is ξ = 8.02%.

The TMD is simply implemented like a nodal mass linked to the walkway deck by a spring and linear dashpot link; in the FEM with the TMD's implementation the maximum displacement decreases to 0.725 mm and the frequency slightly reduces to f_1 = 1.04 Hz. Combining the new values of the maximum vertical displacements and the frequency the global behaviour of the footbridge improves to a lightly perceptible oscillating move [2]. Moreover, considering the SETRA classification, if only the walkway is implemented its frequency $f_{1\text{-onlywalkway}}$ = 1.04 Hz is in the medium R range but, if all of the footbridge structure is implemented, its frequency the $f_{1\text{-structure}}$ = 2.16 Hz is in the high R range.

F2 - case

The second footbridge F2 is characterized by a 67.6 m length and 7.08 m width; ithas a steel deck and static conceptual scheme showed in the next Figure 3.

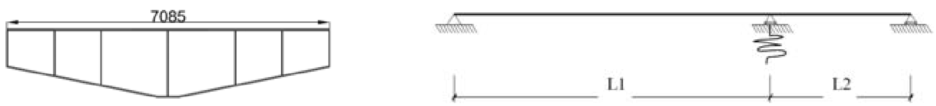

Figure 3. FB section and structural conceptual design (L1 = 45.9 m , L2 = 21.7m).

The F2 connects two urban areas of an important Italian city. By Setra, it is possible classify FB in class III.

Two FEM are implemented in order to deep the F2 structural response: a beam model and a plate model. Comparing the vibration modes by eigenvalue analysis carried out in both FEMs, the results are practically the same in terms of vibration mode shapes and stress effects. As the FA case, here a TMD solution is studied to improve the F2 vibrancy behaviour. The TMD could be installed inside of the deck close the maximum vertical displacement zone. The TMD implementation in the FEM is performed like mentioned in FA case.

In this case, the TMD works for the vertical direction (so the vertical mode shapes has to be considered) because, from the FEM analysis:

- the first frequency is f_1 = 2.11 Hz (in vertical direction) and the mass involved is the 47.30% of the total mass;
- the second frequency is f_2= 6.66 Hz (horizontal direction) and the mass involved is the 71.21% of the total mass.

Known the value of the first frequency, by SETRA the f_1 is included in the high R frequency range for vertical direction shape. That shows the importance of the TMD's application. For F2, the TMD has the following characteristics: m_{tmd} = 18.2kN (corresponding to the 1.25% of the mass involved in the first mode), stiffens K_{tmd} = 312 kN/m and a damping coefficient c_{tmd} = 3.61 kNs/m. The resulting optimum ratio of frequencies is α_{opt} = 0.98 and the optimal equivalent viscous damping ratio is ξ = 7.56%.

Implementing the TMD in the FEM with plate elements, the maximum displacement decreases from δ_{max} = 1.2 mm to δ_{max} = 0.3 mm and the frequency changes from f_1 = 2.11 Hz (without TMD) to f_1= 0.61Hz (with TMD). Thus, referring to RML method, the vibrancy is reduced to a lower lightly perceptible state [7].

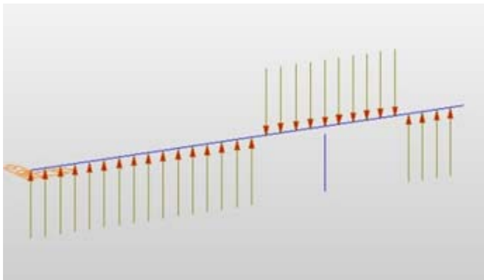

Figure 4. SETRA vertical crowd load application.

Evaluating the structural behaviour under the crowd load, the Setra indications are followed by the application of the crowd load in the direction of the deformations (Figure 4); the value of the load is defined in the following relation:

$$Q = d \cdot (280N) \cdot \cos(2\pi f_v t) \cdot 10.8 \cdot (\xi/n)^{0.5} \cdot \psi$$

where:

- d is evaluated for dense traffic and III class footbridge, d = 0.5 pedestrian/m^2 ;
- ξ is a no-dimensional damping ratio;
- n is the number of people on the footbridge deck.

Introducing the TMD, under the mentioned Setra load condition, the acceleration decreases to 0.23 m/s^2: this value is included in the more comfortable range for the III class acceleration in vertical direction.

3. A TALL BUILDING CASE

To improve the structural behaviour under wind loads in a slender tall building the TMD installation could be considered, especially if the shape - and consequently the geometrical proportions - of the building plays a symbolic architectural role.

In these cases, if the geometrical slenderness (λ) [6] can not decrease the building could be characterized by λ>5÷7; it represents a disadvantageous quality because the structural response under the wind action could be afflicted by dynamic effects due to the vortex shedding.

Normally the passive TMD is positioned at the top of the building and its functioning in the Italian code is considered valid only for the SLS to improve the occupants comfort.

On the contrary, the TMD really represents an important solution to improve the wind response also for the ULS like it is explained in the case disucussed in the following.

A FEM of a new slender tall building is implemented by beam and plate elements. The main dimensions of the buildings are: 220m height, 74m x 28m rectangular section (so λ = 7.85).

The structure is characterized by: two lateral cores in high strength reinforced concrete (HSC, C70/85) located at 48m mutual distance, beams and columns in HSC, and 0.25m post tense concrete slab deck. The main frequencies (f) estimated by the eigenvalue analysis are f_1=0.14Hz, f_2=0.15Hz and f_3=0.26 Hz.

In the wind analysis, directional wind velocities (v) come from 16 different directions are considered. The values of the velocities correspond to return period T_R=50 years.

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 (Figure 5).

By wind tunnel tests and the application of the High Frequency Force Balance (HFFB) [6], [7], [8] procedure, the base shear (V) and bending moment (M), with the top acceleration (α_{max}), are valued for each directions.

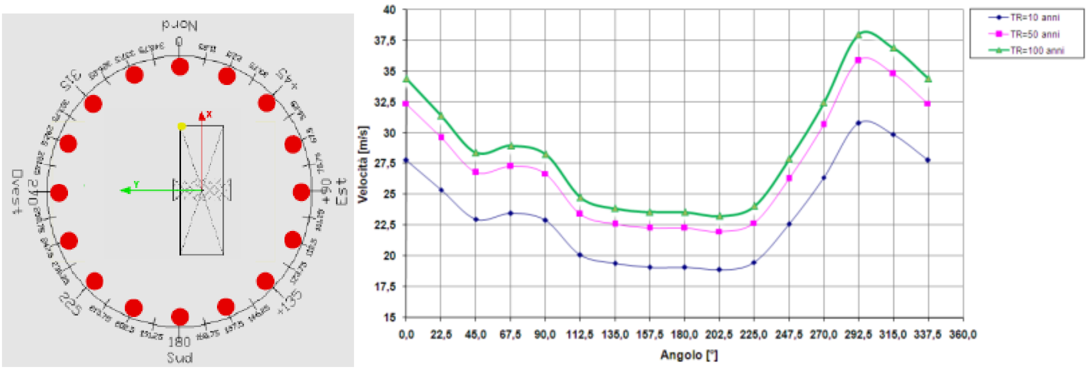

Figure 5. Direction wind velocity.

By the HFFB procedure it is possible to understand the mentioned dynamic effects because the V, M and α_{max} obtained by the HFFB are greater than the values calculated by the application of the CNR DT207 Italian regulation for the wind construction analysis). The differences in terms of base shear are showed in the Table .

Moreover, to not exceed the TMD's limit installation due an excessive mass [6], by the HFFB the α_{max} is evaluated considering two values of damping coefficient (ξ), respectively: ξ = 1% in the case without TMD and ξ = 4% in the case with TMD (this value is the summation of the SLS concrete damping ξ_s=1% and the TMD damping ξ_{tmd} = 3%).

For the SLS comfort building conditions the α_{max} has to be minor than the limit acceleration (α_{lim}) suggested in literature [8]. The α_{lim} in case of office intended use is α_{lim} = 0.25 m/s^2 referring to T_R = 10 years. from the HFFB analysis the α_{lim} for ξ = 1% is α_{max} = 0.47 m/s^2 , whereas for ξ = 4% α_{max} = 0.24 m/s^2.

Table 3: Base shear V comparison.

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

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