

Dynamic behaviour of steel footbridges: the “Ponte del Mare” case study

Problematiche di carattere dinamico di passerelle in acciaio: il caso studio del “Ponte del Mare”

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Slender steel footbridges are very sensitive to dynamic phenomena induced by the wind and pedestrians. This article presents the dynamic analysis of the “Ponte del Mare” footbridge in Pescara. Both the aeroelastic stability and the dynamic pedestrian effect are investigated. A numerical wind analysis based on wind tunnel tests is reported. The advantages of employing a three dimensional (3D) finite element (FE) model, inclusive of flutter derivatives, are presented. The need to install dampers to avoid a galloping phenomenon is highlighted. Finally, the numerical study of the pedestrian dynamic effect investigates conditions that might induce pedestrian discomfort while crossing the bridge.

Le passerelle pedonali in acciaio, per loro natura snelle, sono molto sensibili a fenomeni dinamici indotti dalle azioni del vento e dei pedoni. L'articolo presenta le analisi dinamiche della passerella ciclo-pedonale “Ponte del Mare” situata a Pescara. Sono investigati gli aspetti fondamentali riguardanti la stabilità aeroelastica e l'effetto dinamico dei pedoni. Si riporta quindi l'analisi sotto forzamento eolico basata su risultati di galleria del vento, presentando i vantaggi di un modello tridimensionale agli elementi finiti (EF) che include le derivate di flutter. Inoltre si evidenzia la necessità di installare dei dispositivi di smorzamento a causa di un fenomeno di galloping. Infine, lo studio numerico dell'effetto dinamico dei pedoni verifica che il comfort durante l'attraversamento della passerella sia garantito.

1. INTRODUCTION

In the last years, the evolution of the construction techniques and of the structural design criteria has led to consider the architectural and aesthetic aspects as key factors within the design process. In this respect, footbridges represent the best example: by the nature of the vertical live loads, i.e. pedestrians, the choice of geometries and construction solutions is less bound to practical considerations. Moreover, by exploiting the steel properties the designers have conceived lighter and more slender structures, improving the aesthetic and often even reaching artwork perceptions. This attitude has led to decrease the environmental impact but at the same time it has highlighted issues related to dynamic phenomena. In fact, such structures, like footbridges, cable-stayed bridges and suspension bridges, associated to large spans, are characterised by low damping capabilities that can undermine both the safety and the comfort of the bridge. Wind is one of the main causes of dangerous unwanted effects, such as aeroelastic instability, and the Tacoma Narrows Bridge is the most famous example of structural failure due to a flutter phenomenon [1]. In that case, the wind-structure interaction, source of aeroelastic phenomena, caused increasing self-excited oscillations that brought the structure to collapse; basically the bridge was unable to entirely dissipate the energy introduced by the wind. In this respect, it is clear that the wind-structure interaction analysis assumes fundamental importance for the aforementioned structural typologies and both sophisticated theoretical and experimental studies are being performed. From a theoretical viewpoint the need of an accurate structural modelling integrated with a reliable simulation of the wind action, seen as a natural, dynamic and stochastic phenomenon, is essential. For this reason, in condition of high turbulence, complex Computational Fluid-Dynamics (CFD) analyses are also exploited. When it becomes necessary to evaluate an accurate estimate of the wind forces acting on a structure of particular interest, wind tunnel tests are carried out. Generally, scaled models are used and the boundary conditions that include the topography of the terrain are also reproduced.

The dynamic pedestrian loading may also cause undesirable effects related to excessive oscillations and loss of comfort and functionality, without however undermining the safety. For this reason it is associated to a serviceability limit state. The London Millennium Bridge is the most famous example [2]. Bridges result to be pedestrian loading sensitive if some transverse modes are comprised in the 0.8-1.2 Hz range or some of the vertical or torsional modes are within the 1.6-2.4 Hz frequency range. The first step of the analysis consists of the mathematical modelling of the single pedestrian effect, whilst the successive step is the accurate modelling of the synchronised process, called lock-in, that entails the pedestrians to move in phase with the structural displacements, thus increasing vibrations. The various modelling proposals are based on experimental parameters that are most of the times suitable only for the structure being in-

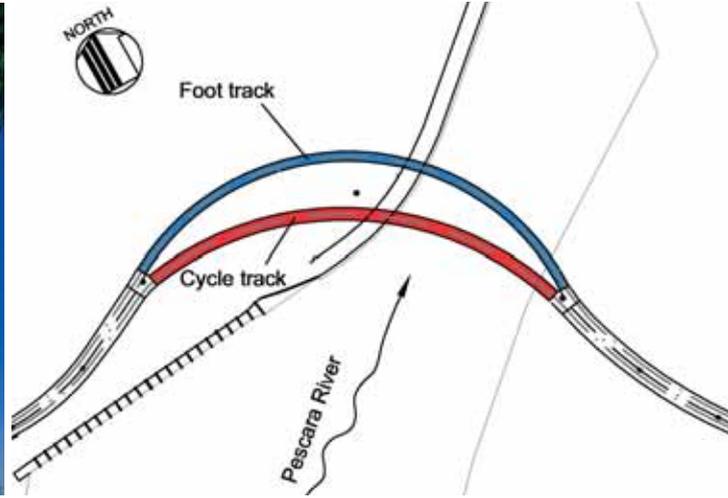


Fig. 1 - The "Ponte del Mare" footbridge

investigated. So far only the French SETRA guidelines [3] provide a general method applicable to every bridge typology. For both the wind and the pedestrian action if an unsatisfactory structural behaviour occurs, damping devices can be designed and installed on the bridge in order to increase the total damping. The article is organised as follows: the description of the "Ponte del Mare" footbridge and the FE model are presented in section 2, whilst in section 3 the numerical wind analysis developed by means a 3D FE model that includes the flutter derivatives is described. The analysis of the dynamic pedestrian loading is reported in section 4, while the conclusions are drawn in section 5.

2. DESCRIPTION OF THE "PONTE DEL MARE" FOOTBRIDGE AND THE FE MODEL

The "Ponte del Mare" footbridge is located at the mouth of the Pescara river, that divides the city of Pescara into two parts. The structure is conceived into a programme that aims at improving the urban area and it allows to connect the northern to the southern side of the town. The architectural project was conceived by arch. Walter Pichler of the design office Walter Pichler & Partner – Bolzano; the preliminary structural design was per-

formed by DELTA Ingegneria – Trento, while Mario de Miranda, of the consulting office de Miranda & Associates – Milano, was in charge of the final design.

The footbridge is a cable-stayed structure and it consists of two curved decks sustained by cables connected to a tilted mast, as illustrated in fig. 1. The outer deck is dedicated to pedestrians, whilst the inner one is for cyclists; they have constant radius, approximately 80 m and 100 m, respectively, and their length is 173 m and 148 m. The two decks are constituted by a spatial steel-concrete composite truss connected to two prestressed concrete access ramps. The two sections of the bridge are reported in fig. 2. The mast is made of steel and its inclination is about 11° with respect to the vertical axis. Two anchorage cables connect the top of the mast to the ground in order to reduce the displacements. In sum, the complexity of the structure is evident, just note that the bottom chord of the foot-track deck is in tension, whilst the one of the cycle-track deck is conversely in compression because of the eccentricity of the cable attachments and of the curve orientation with respect to the mast position. Thus, considering also the structural slenderness and high flexibility associated to low damping values, both the Università di Trento and the Politecnico di Milano

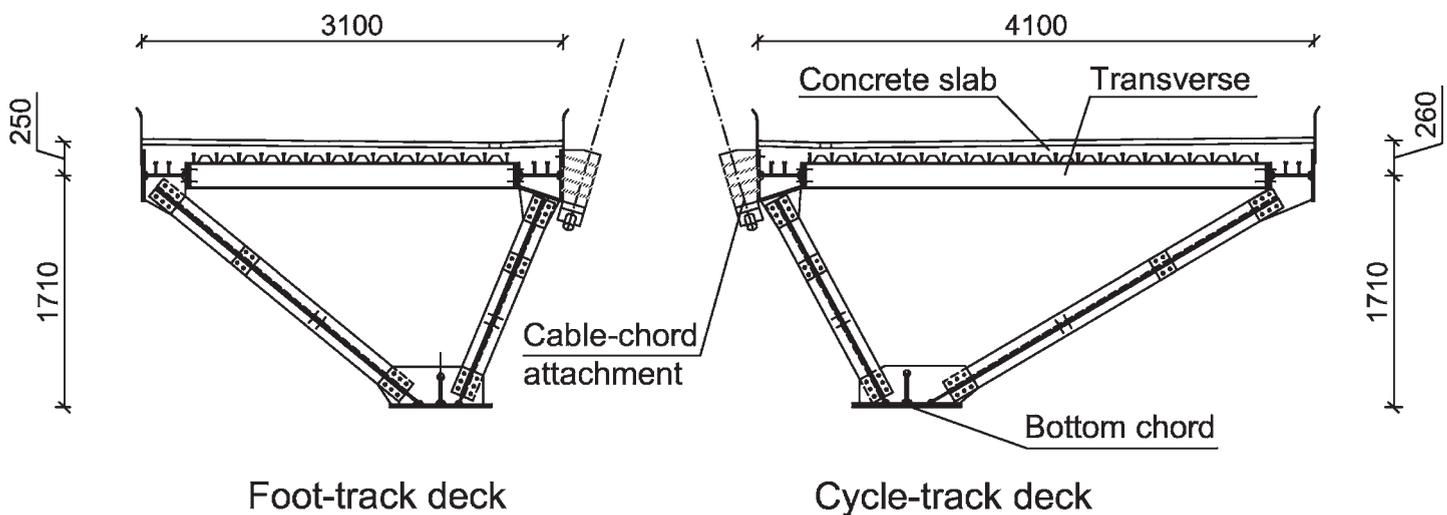


Fig. 2 - Deck cross-sections. Dimensions in mm.

intervened to provide consulting support during the dynamic behaviour study [4]. In detail, the Università di Trento performed numerical analyses to understand the wind effect based on a full 3D FE model that relies on experimental data provided by wind tunnel tests carried out at the Politecnico di Milano. Moreover, the Università di Trento was also in charge of the pedestrian loading analysis as well as a monitoring phase of one-year duration.

A full 3D FE model was developed in ANSYS, as shown in fig. 3. Beam, shell and solid elements were used to accurately model both the main steel decks and the access ramps, the latter having however little influence on the dynamic behaviour because of the presence of Gerber hinges. The model was used both for the wind and pedestrian loading analyses. Moreover, in order to model the aeroelastic forces, an user-defined Matrix27 element was implemented [5,6]. It is a 6 degree-of-freedom two-node element whose behaviour depends on the mass, stiffness and damping matrices. In this way, it was possible to implement the aeroelastic forces by formulating the aeroelastic stiffness and damping matrices that are function of the flutter derivatives of the footbridge sections. The flutter derivatives are obtained experimentally and depend on the wind attack angle. In this way, the structure spatiality was considered and this gave the possibility to take into account the variability of the wind attack angle along the deck length owing to the static deformed shape. This wind modelling technique is rarely employed in the common practice, but despite being computational demanding, it becomes very useful for a better and more complete understanding of the behaviour of complex structural typologies under wind action [4].

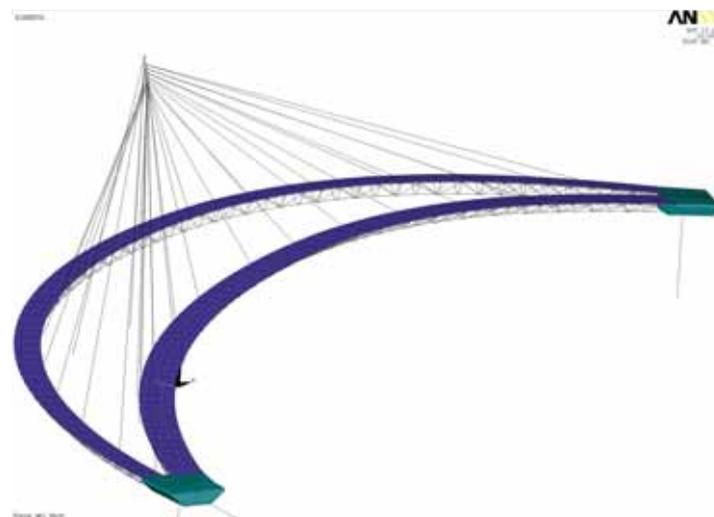


Fig. 3 - 3D FE model

3. WIND ANALYSIS

First of all, in order to obtain the dynamic characteristics of the footbridge, an eigenvalue analysis taking into account both the prestressing effects due to dead loads and the deformed shape resulting from their application was performed. The modal frequencies and periods for the first 12 modes are reported in tab. 1, while the relative mode shapes are depicted in fig. 4.

The wind analysis focussed on the aeroelastic response, because an aeroelastic instability can undermine the structural safety owing to the structure incapability to dissipate the energy introduced by the wind flow at a certain velocity. The lift and moment aeroelastic forces per unit length, by neglecting the drag components and according to the Zasso [7] representation convention, can be described by the following formulae with respect to the sign agreement shown in fig. 5.

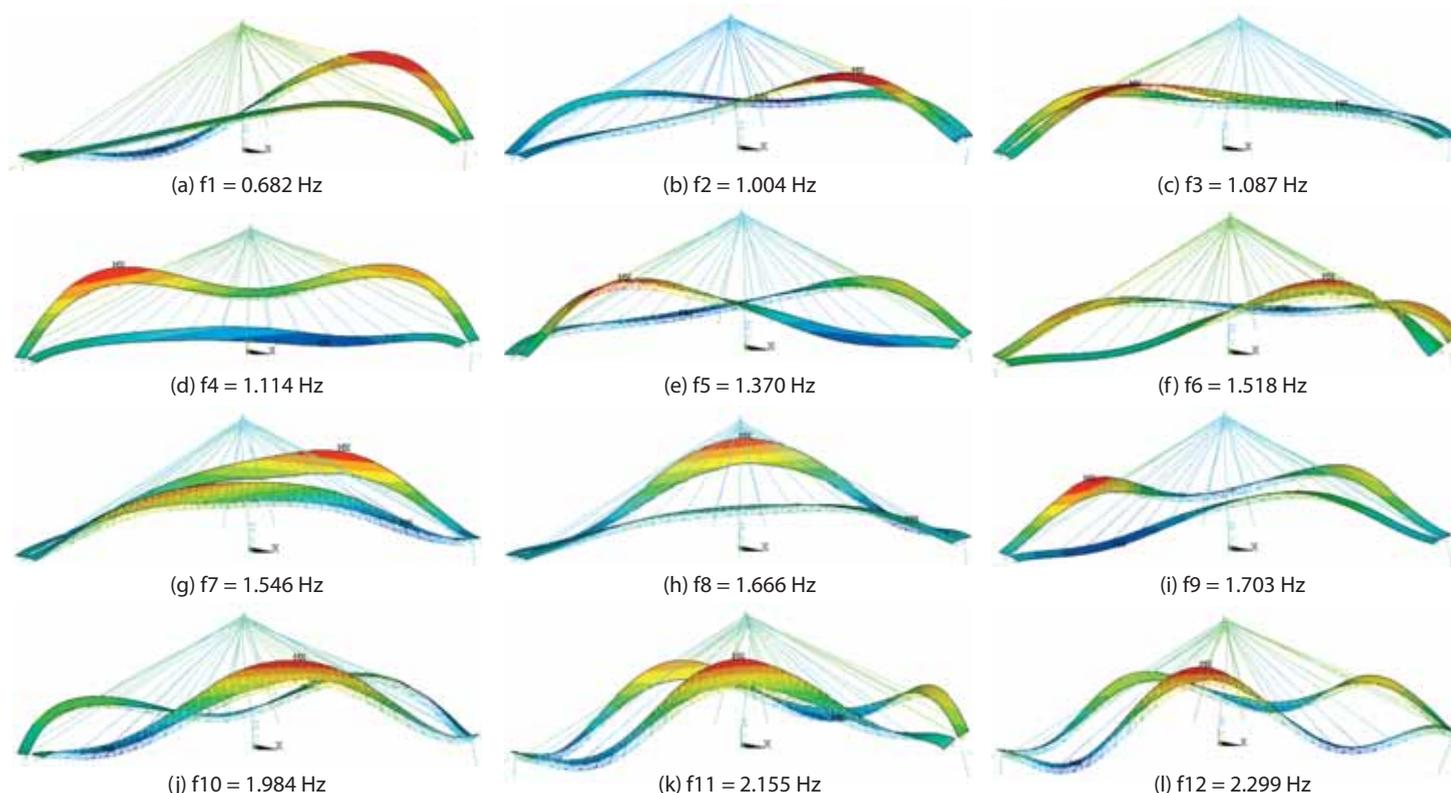


Fig. 4 - Numerical mode shapes

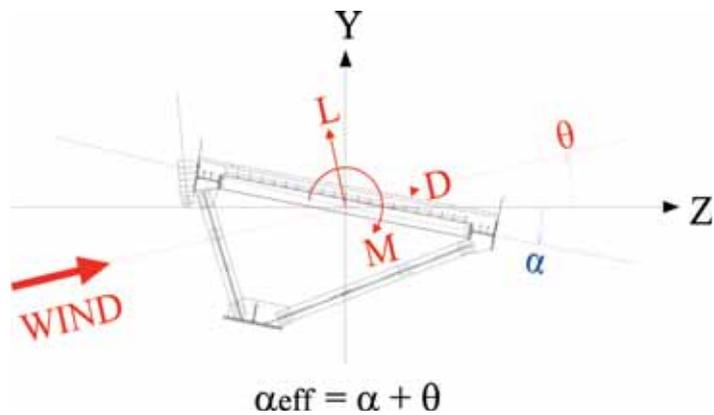


Fig. 5 - Sign convention of the wind forces [7]



Fig. 6 - Foot-track deck scaled sectional model tested in the wind tunnel

Tab. 1 - Numerical modal frequencies and periods

Mode	Frequency [Hz]	Period [s]
1	0.682	1.147
2	1.004	0.996
3	1.087	0.920
4	1.114	0.874
5	1.370	0.730
6	1.518	0.659
7	1.546	0.647
8	1.666	0.600
9	1.703	0.587
10	1.984	0.504
11	2.155	0.464
12	2.299	0.435

$$L(t) = \frac{1}{2} \rho V^2 B \left[-h_1^* \frac{\dot{y}(t)}{V} - h_2^* \frac{B\dot{\alpha}(t)}{V} + h_3^* \alpha(t) + h_4^* \frac{\pi}{2V\omega^2} \frac{y(t)}{B} \right]$$

$$M(t) = \frac{1}{2} \rho V^2 B^2 \left[-a_1^* \frac{\dot{y}(t)}{V} - a_2^* \frac{B\dot{\alpha}(t)}{V} + a_3^* \alpha(t) + a_4^* \frac{\pi}{2V\omega^2} \frac{y(t)}{B} \right] \quad (1.1)$$

where $h_i^*(V^*, \alpha_\sigma)$ and $a_i^*(V^*, \alpha_\sigma)$ ($i=1, \dots, 4$) are the flutter derivatives obtained experimentally by means of wind tunnel tests. They are function of the average attack angle α_σ , the motion angular frequency ω , the flow velocity V and the deck width B through the reduced velocity defined as $V^* = V/fB$ or $V_\omega^* = V/\omega B$. Moreover h_i^* and a_i^* ($i=3, 4$) have a meaning of aerodynamic stiffness, whilst h_i^* and a_i^* ($i=1, 2$) are meant as aerodynamic damping.

As mentioned before, the wind tunnel tests were necessary to obtain both the flutter derivatives and the stationary aerodynamic coefficients - $c_D(\alpha_\sigma)$, $c_L(\alpha_\sigma)$, $c_M(\alpha_\sigma)$ -, so that to fully characterise the aerodynamic properties of the bridge sections. As a result scaled models of the two decks were tested, as illustrated in fig. 6. The data evaluation of the foot-track deck showed, see fig. 7a, negative values of h_1^* for probable average attack angles and for low reduced velocities when the wind came from the sea. This was expected since the lift aerodynamic coefficient c_L exhibits negative

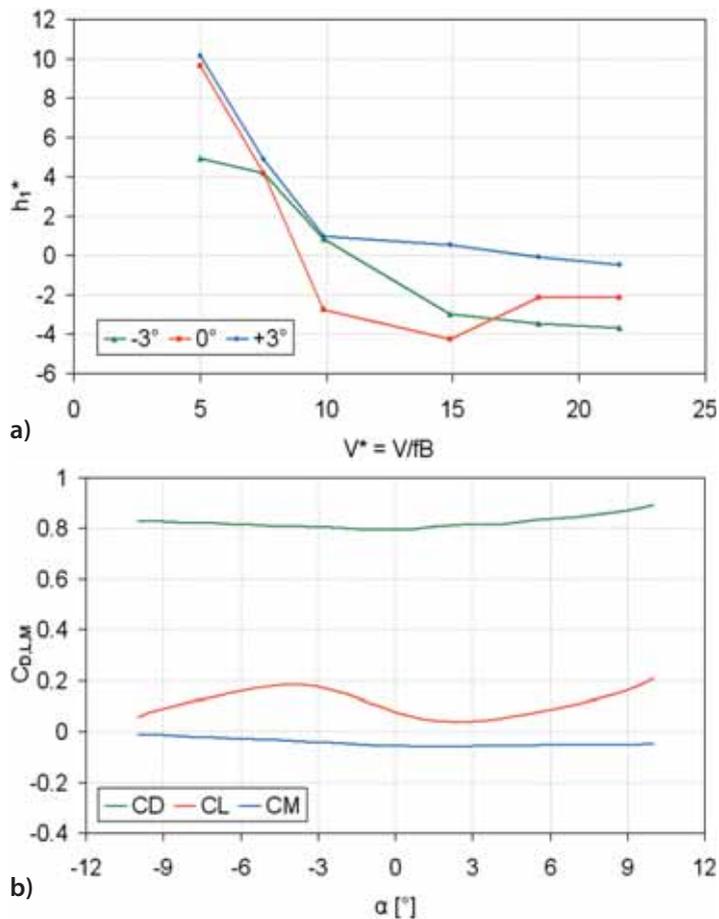


Fig. 7 - a) h_1^* flutter derivative values for different wind attack angles α as a function of the reduced wind velocity V^* and b) stationary aerodynamic coefficients c_D , c_L , c_M

slope in the neighbourhood of the origin, as depicted in fig. 7b. This entails a negative effect on the aeroelastic behaviour of the structure for the vertical degree of freedom if one notes that h_1^* is associated to a damping measure, because in this case self-excited oscillations with increasing amplitude owing to energy absorption are likely to occur for low wind speeds. The wind behaviour of the bridge was ascertained by carrying out several numerical analyses on the 3D FE model developed in ANSYS. In order to investigate in depth the wind effect on the bridge the aeroelastic forces by means of user-defined finite elements were included, as explained before. This technique has the advantage to model in a more re-

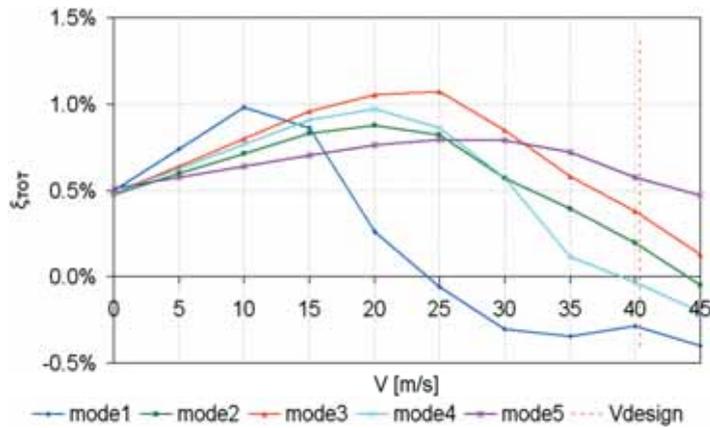


Fig. 8 - Total damping ξ_{tot} as a function of the wind velocity for the structure without dampers

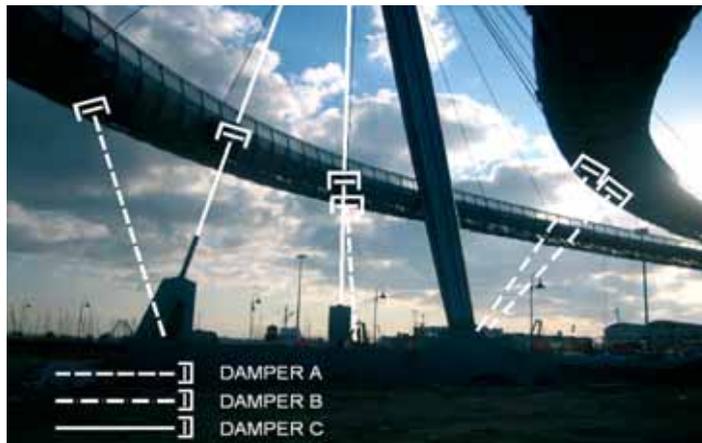


Fig. 9 - Damper location

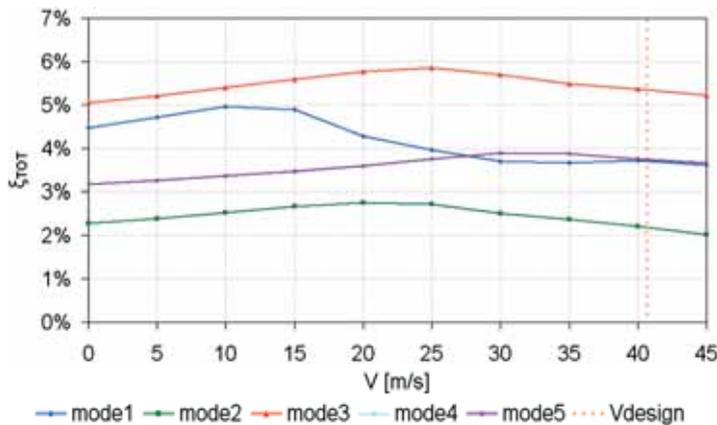


Fig. 10 - Total damping ξ_{tot} as a function of the wind velocity for the structure with dampers

Tab. 2 - Damper characteristics

PARAMETER	DAMPER A	DAMPER B	DAMPER C
DAMPING CONSTANT [kNs/m]	90	200	900
SPRING STIFFNESS [kN/m]	130	130	-

alistic way the wind effect on the structure. The aeroelastic forces were applied orthogonally to the deck axes and by means of the 3D FE model the structure spatiality and the variability of the attack angle of wind could be taken into account. In fact, both the station-

ary aerodynamic coefficients and the flutter derivatives were set on the basis of the local attack angle that depends on the deformed shape due to dead loads. The wind design velocity was set at 40.4 m/s for the case of wind coming from the sea and it was the most unfavourable case. The structural damping was assumed to be $\xi_{struc} = 0.5\%$. As confirmation of the wind tunnel test findings, the total damping $\xi_{tot} = \xi_{struc} + \xi_{aero}$ became negative for the first mode at a wind speed well below 40.4 m/s, as illustrated in fig. 8. This means an aeroelastic instability related to increasing energy absorption, in this case regarding a galloping phenomenon of the foot-track deck that only involves the vertical degree of freedom. For this reason, the introduction of devices capable to increase damping was deemed necessary. Hence, fluid-viscous dampers were designed in order to add sufficient damping to the structure and to avoid any change of the modal parameters, i.e. frequencies and mode shapes. In detail, two dampers - Dampers A - were conceived to connect the foot-track deck to the ground, while other two - Dampers B - do the same with the cycle track deck; finally a further couple - Dampers C - was envisaged to link the mast anchorage cables to the foot-track deck. Their location is shown in fig. 9, whilst tab. 2 reports the characteristics used in the numerical model.

Briefly, a linear force-velocity relationship was requested and the damping coefficients c_A and c_B were initially estimated according to an equivalent single-degree of freedom system through the $c=2\xi\omega m$ formula and were calibrated in such a way to prevent aeroelastic instability for the first mode; where ξ is the total modal damping ratio set at 2.5% (considered a safety value to satisfy aeroelastic stability), ω is angular frequency for the considered mode and m is the respective modal mass. Then, the steady-state acceleration due to the pedestrian effects, explained in Section 4, is inverse proportional to the damping ratio, if the latter is small. Thus, the additional damping needed on higher modes in order to meet the comfort requirements, expressed in terms of acceleration limits [3], was computed on the basis of the results of numerical simulations carried out on the structure without dampers. Owing to the architectural constraints, the damper positioning was optimised so as to guarantee effectiveness against the modes relevant for the wind and the pedestrian action. To improve the structure aesthetic appeal, the initial idea of employing rigid tubes for connecting Dampers A and B to the ground was discarded. Thus, slender cables were used, and in order to keep them in traction, Dampers A and B were endowed with a spring, whose stiffness was chosen at the lowest possible value in order to avoid changes in modal frequencies and shapes. Conversely, an additional stiffness for Dampers C was not reckoned necessary, since the attachment points were represented by the anchorage cables.

Likewise fig. 8, fig. 10 shows the total damping for the first 5 modes as a function of the wind speed; the total damping related to mode 4 is not represented because always well above 7%. In this con-

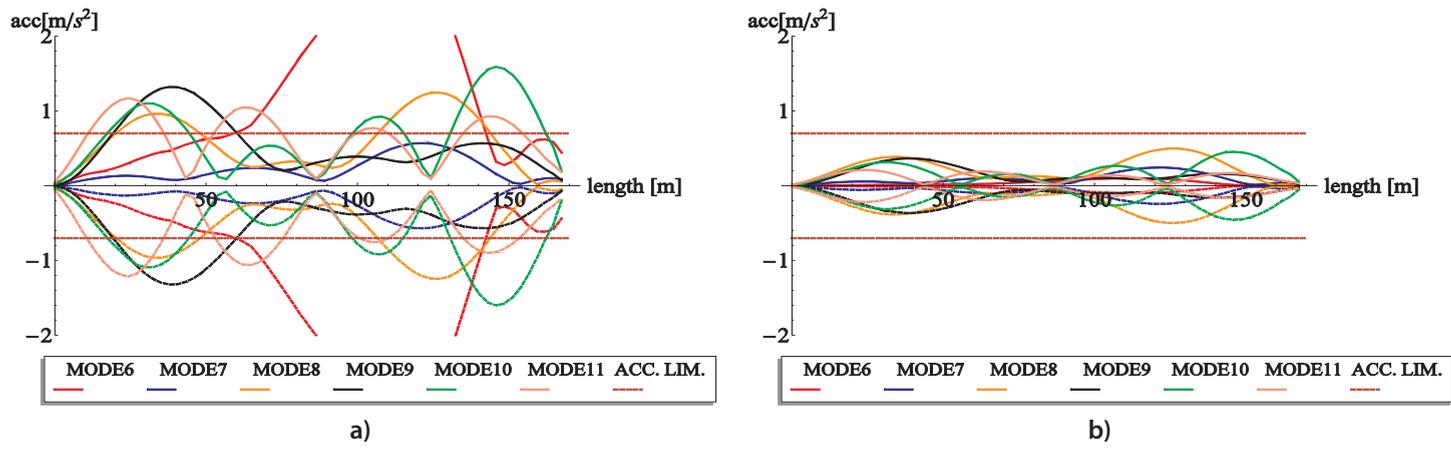


Fig. 11 - Maximum vertical accelerations along the foot-track deck with $d=0.8$ pers/m²: (a) structure without dampers; (b) structure with dampers

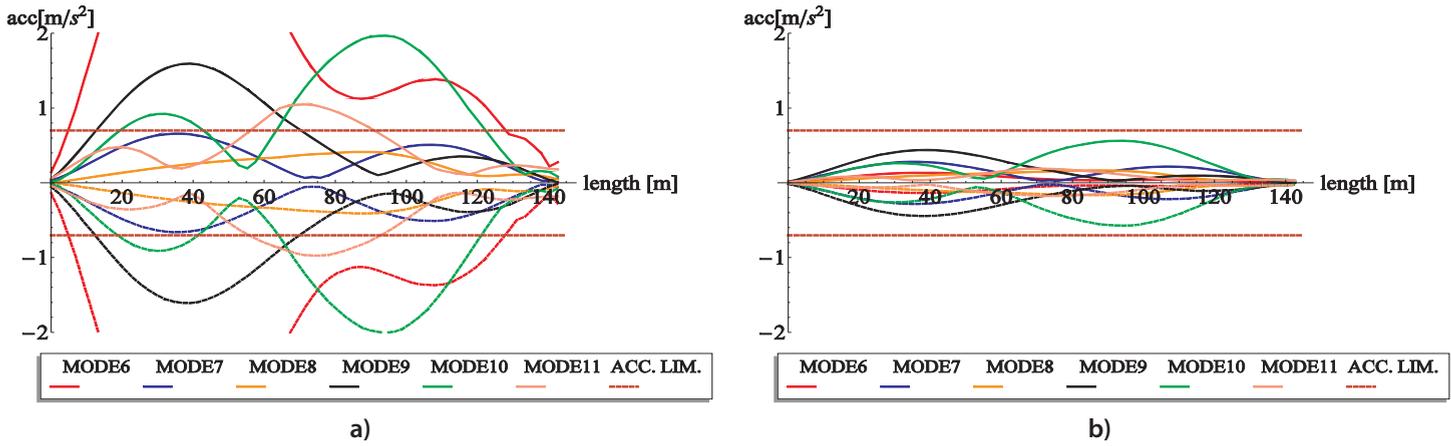


Fig. 12 - Maximum vertical accelerations along the cycle-track deck with $d=0.8$ pers/m²: (a) structure without dampers; (b) structure with dampers

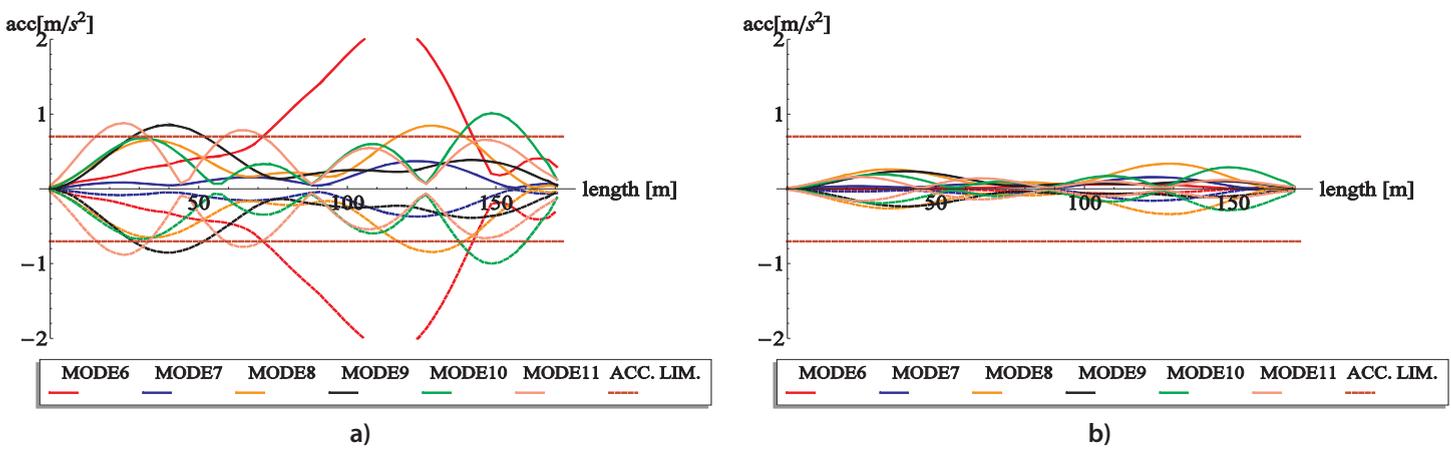


Fig. 13 - Maximum vertical accelerations along the foot-track deck with both decks loaded with 200 people: (a) structure without dampers; (b) structure with dampers

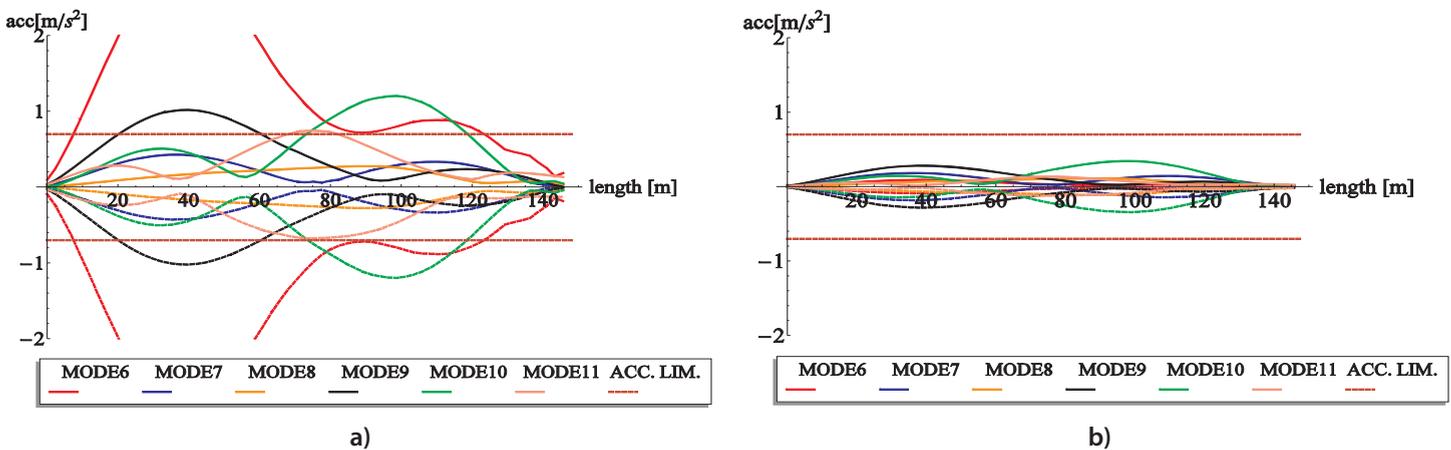


Fig. 14 - Maximum vertical accelerations along the cycle-track deck with both decks loaded with 200 people: (a) structure without dampers; (b) structure with dampers

figuration the total damping clearly results to be always positive, also beyond the design wind speed. Therefore, the structure now exhibits a satisfactory behaviour with respect to the wind action.

4. PEDESTRIAN ANALYSIS

The pedestrian analysis was performed in order to check the comfort during the bridge crossing. The pedestrian force was modelled according to the French SETRA guidelines [3] and the vertical, transverse and longitudinal components were considered. The analyses of the effects induced by the pedestrians were conducted for the modes that are sensitive to this dynamic action, i.e. 6-11 as shown in tab. 1. For each mode the forcing input was considered in resonance and was applied according to the same sign of the mode shape. Two loading conditions were considered: i) both decks loaded with a crowd density of 0.8 pers/m²; ii) both decks loaded with 200 people, that correspond to 0.385 pers/m² for the foot-track deck and 0.342 pers/m² for the cycle-track deck. The weight of one person was set $G_0 = 0.7$ kN [3]. The mathematical model of the pedestrian loading is based on a decomposition in Fourier series. The SETRA guidelines provide the vertical component of the force per unit m² as the sum of a static term and a dynamic term that reads

$$F_{v,i}(t) = F_{st} + \frac{N_{eq} G_0}{A_i} \alpha_v \sin(2\pi f_v t) \Psi_v \quad \text{where } F_{st} = G_0 d \quad \text{with } i = ped \text{ or } cyc \quad (1.2)$$

where N_{eq} is the equivalent number of synchronized pedestrians in resonance with the considered mode; α_v is the reduction factor of the vertical force amplitude; f_v is the frequency of the vertical force equal to the modal frequency of the considered mode; Ψ_v is the reduction factor of the resonance probability; A_i is one of the two deck areas and d is the crowd density. Likewise, for the transverse and longitudinal directions, but with the static term equal to zero and a change of the relevant subscripts, one gets

$$F_{t,i}(t) = \frac{N_{eq} G_0}{A_i} \alpha_t \sin(2\pi f_t t) \Psi_t \quad \text{and} \quad F_{l,i}(t) = \frac{N_{eq} G_0}{A_i} \alpha_l \sin(2\pi f_l t) \Psi_l \quad (1.3)$$

The SETRA guidelines indicate a vertical acceleration limit, based on the footbridge class, that should not be exceeded if the com-

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fort on the bridge during its crossing be ensured. On the basis of this maximum value, the vertical acceleration in any points of the structure should be less than 0.7 m/s². In detail, the pedestrian force components for the considered loading conditions were applied and then the vertical maximum accelerations along the deck length for the modes of interest were monitored. The numerical analyses of the structure without and with dampers are compared and reported in figs. 11-14. From the figures it is possible to observe the effectiveness of the dampers with respect to the dynamic pedestrian loading, being the accelerations lowered below the limits.

5. CONCLUSIONS

Long span steel footbridge are sensitive to dynamic actions, i.e. wind and pedestrians, owing to their intrinsic high flexibility and slenderness. The dynamic analyses of the "Ponte del Mare" in Pescara presented in this article confirm this peculiar issue of such structures. In fact, the wind analysis, performed by including the aeroelastic forces applied to a 3D FE model, shows that the footbridge cannot withstand this action for the specified wind design speed because of aeroelastic instability related to a galloping phenomenon. Thus, it highlights the necessity to introduce viscous dampers to overcome the problem. Moreover, also the dynamic pedestrian effect is analysed by means of the SETRA guidelines and it results to be fundamental to check the comfort during the bridge crossing. Also in this case, the presence of dampers was deemed necessary to comply with the acceleration limits based on the French guidelines. Finally, further studies should be aimed at identifying the actual dynamic characteristics of the footbridge by means of experimental dynamic identification tests. In this way, the FE model will be calibrated so as to improve the analysis accuracy.

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