Wind loads analysis at the anchorages of the Talavera de la Reina cable stayed bridge

L. Rosa [a,*], G. Squicciarini [b], M. Belloli [a], A. Collina [a], F. Nieto [c], J.A. Jurado [c]

[a] *Politecnico di Milano, Department of Mechanical Engineering, via G. La Masa 1, 20156 Milan, Italy*
[b] *Institute of Sound and Vibration Research, University of Southampton, Highfield, Southampton SO17 1BJ, UK*
[c] *School of Civil Engineering, University of La Coruña, Campus de Elviña s/n, La Coruña, Spain*

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Introduction

Cable stayed bridges are a feasible choice for middle length spans and can be also an alternative to suspension bridges in the range of 1000 m spans. In the last three years, cable stayed bridges having more than 1 km span have been built in China (Sutong Bridge, 1088 m and Stonecutters Bridge, 1018 km). The increasing in length implies higher aeroelastic interaction with wind action to be considered in the design stage.

The present paper deals with the wind response of a cable stayed bridge, the Talavera de la Reina Bridge (Spain), by means of both sectional model and full bridge aeroelastic wind tunnel tests. Talavera de la Reina is a middle-size city in Central Spain. A new highway has been recently built by-passing the city at its Southern limit and a landmark cable-stayed bridge has been designed spanning the river Tagus. The bridge has a single span of 318 m which is supported by a single inclined pylon 164 m high. The pylon angle of inclination with respect to the vertical direction is 22 deg. Two planes of cables connect the deck and the pylon which is back-stayed by another two planes of cables. In Fig. 1 the general geometric definition of the bridge along with the deck cross-section can be perused. The deck is a multi-cell prestressed concrete box with overhangs at the sides for pedestrians. The total width of the deck is 36 m while the depth is 2.77 m which results in a width to depth ratio close to 13. The bridge enjoys a careful aesthetic design, adequate for structures located in urban areas. The particular design of the bridge makes it noteworthy and its wind response worth to be investigated in wind tunnel at design stage.

Aims of the tests were the evaluation of the deck force coefficients and, by means of full bridge aeroelastic wind tunnel tests, the evaluation of the wind-structure interaction in terms of aeroelastic stability and analysis of the response to turbulent wind, as a function of the wind angle of exposure. Several authors already pointed out the importance of confirming sectional model tests results with full aeroelastic model tests. This is particularly true in the case of cable-stayed bridges,

* Corresponding author. Tel.: +39 0223998076.
E-mail address: lorenzo.rosa@polimi.it (L. Rosa).

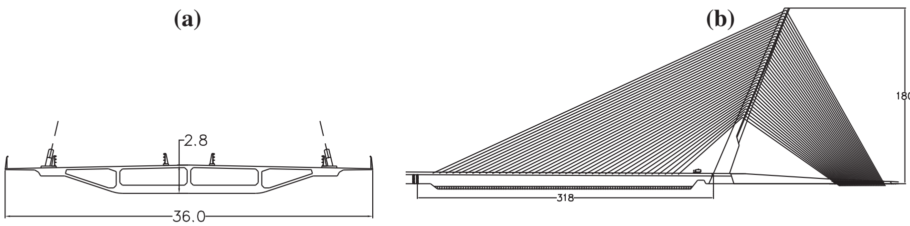

Fig. 1. (a) The cross section of the bridge deck. (b) A lateral view of the bridge. The measures are in *m* full scale.

where the tower-girder-cables interaction can have a positive or negative effect on the wind response depending on the bridge design and on the type of wind excitation, e.g. vortex shedding or buffeting [8,6,7,9].

Sectional model tests

A sectional model of the Talavera de la Reina Bridge deck was tested at the aerodynamic wind tunnel facilities of the University of La Coruña. This experimental campaign aimed to provide an initial assessment of the aerodynamic performance of the proposed design. The set of sectional model tests comprised the evaluation of the deck force coefficients along with the evaluation of the complete set of 18 flutter derivatives required to obtain the critical flutter speed for the bridge. The rigid sectional model of the deck was made at 1/100 geometric scaled, taking special care to reproduce the geometric details such as the handrails or the stay anchorages at the deck. In Fig. 2 an image of the sectional model inside the wind tunnel test section is presented.

The static force coefficients of the deck have been evaluated in agreement with the following expression:

$$C_{D,L} = \frac{F_{D,L}}{\bar{q}_h B}, \quad C_M = \frac{M}{\bar{q}_h B^2} \tag{1}$$

where the wind pressure is defined as $\bar{q}_h = \frac{1}{2}\rho\overline{U}^2$, B is the deck width and $F_{D,L}$ and M represent the drag force, the lift force and the moment, respectively. The coefficients and the associated sign convention are shown in Fig. 3. The positive slope of both lift and moment coefficients and their low values anticipate a good response in terms of high critical flutter speed. Beside this, the low values of the drag coefficients at the studied angles of attack are consistent with the low width to depth ratio of the deck.

Full bridge aeroelastic model tests

Experimental set-up

The tests on the full aeroelastic model were carried out in the boundary layer wind tunnel of Politecnico di Milano, Italy, which is 4 m high, 14 m; wide and 36 m long. The model was manufactured using a geometric scale $\lambda_L = \frac{1}{64}$. This value is a compromise between having large models while keeping low blockage values. The defined geometric scale is compatible with the scaled simulation of the boundary layer and the aeroelastic characteristic of the bridge. The flexibility of the

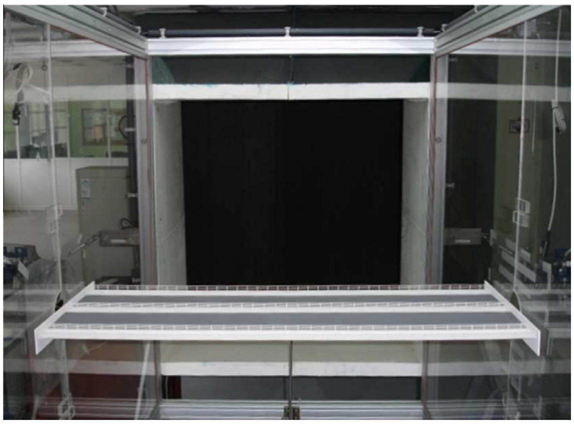

Fig. 2. The sectional model inside the wind tunnel test facilities of the University of La Coruña.

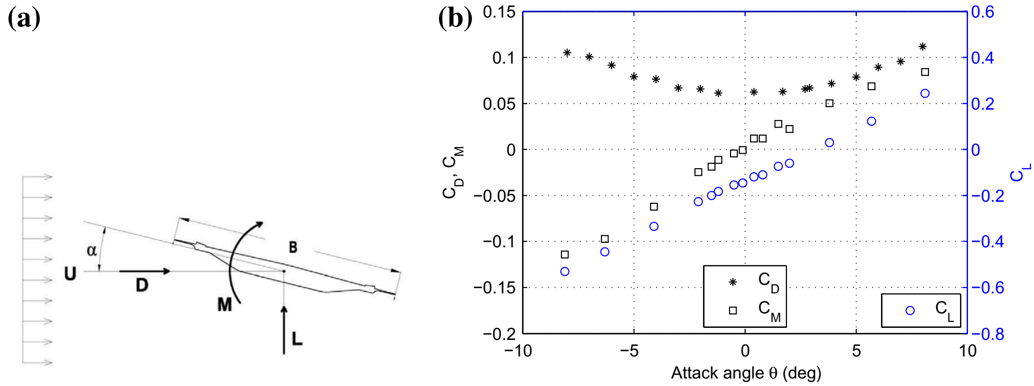

Fig. 3. Deck experimental aerodynamic characteristics [5]: (a) reference system, (b) aerodynamic coefficients.

structure required the use of Froude similitude criteria since this condition permits to consider the influence of gravitational forces in aeroelastic phenomena, leading to a factor $\lambda_F = 8$ for frequencies scaling ($\lambda = model/real$) and leading to a factor $\lambda_V = 1/8$ for scaling the velocities. The other main quantities are scaled accordingly, see e.g. [3]. This criteria was chosen considering an optimum compromise among the geometric scale adopted, the range of wind speeds available (maximum allowed velocity is 14 m/s) and advantages in the manufacture and tuning of the aeroelastic model. In addition this lead to have a 1 : 1 scale factor for the accelerations measured on the model to the real scale structure.

The nominal data of the bridge enabled to build a finite element model of the real bridge from which the frequency and the modal parameters were calculated. These, opportunely scaled, were the target values for the aeroelastic scaled model. In order to help its design, a finite element schematization of the aeroelastic model was also used considering that the structure of the model is different from the one of the real bridge in full scale. In particular deck and tower flexural and torsional stiffness, was modeled using a central steel spine. Mass was added to match the density, and an external cover was adopted in order to reproduce the shape. Boundary conditions were set through proper connection for deck ends and for the tower base. In particular the deck is clamped at the tower side while its rotation and translation in the longitudinal direction at the opposite end are set free. Each stay cable of the model reproduce two stays of the real bridge, except for the longest ones. The axial stiffness of the cables was lumped at cables' end with an helical spring on the deck side and on the ground side for the rear stays. A tensioning device for each stay enabled to regulate the tension. The tension could be controlled through the elongation of the spring at cables' end. Masses were added also on each cable in order to reproduce both the mass distribution and the drag forces in the reduced scale model. Fig. 4 shows the aeroelastic model mounted in the test chamber. The model was designed to simulate also two erection stages of the bridge having different tower heights. It is known (see e.g., [8,1]) that the response in this condition might be more critical than in the full bridge; however these tests will not be discussed furthermore in this paper, more details can be found in [2].

Due to the symmetry of the structure of the bridge, a range of angles of exposure of 180 deg was investigated, with steps of 10 deg, Fig. 5. The wind loads were evaluated in turbulent flow. In this configuration the vertical velocity profile expected at the construction site was reproduced (scaled) and the turbulent wind characteristics were simulated, in terms of integral scale length and turbulence index, by means of spire and roughness elements. Particularly the wind was reproduced taking care of the variation of the mean velocity with the height from the ground and its turbulence. The target profile, is defined by a roughness length $z_0 = 0.025$ m and it is representative of an agricultural area. The along wind turbulence at the deck level

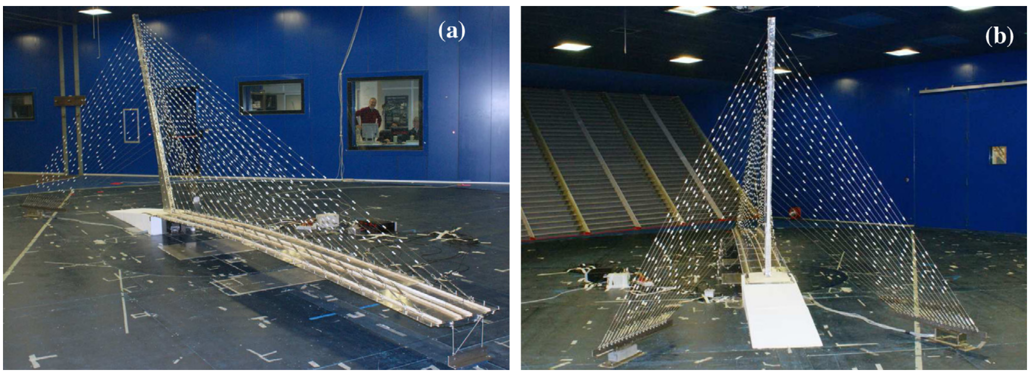

Fig. 4. The aeroelastic model mounted in the test chamber. (a) front view, (b) back view.

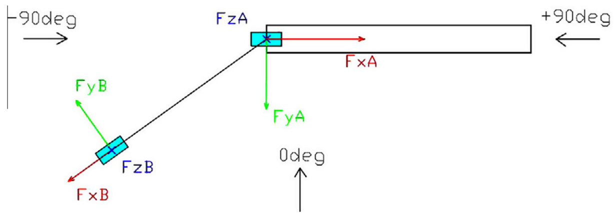

Fig. 5. Exposure angles, reference systems and instrumentations (top view). *FzA* and *FzB* are positive going into the page.

(h) is $I_u(h) = 15\%$ and the turbulence integral scale $L_u^x(h) \cong 1$ m. With the aim of supporting the manufacturer in the designing stage of the bridge the tower and the cable system connection to the ground were equipped with two dynamometric six-components balances, one at the base of the tower (balance A) and the other at one cables ground connection (balance B).

Aeroelastic model results, wind loads

The measured forces on the balances A and B are presented through dimensionless aerodynamic coefficients corresponding to the three force components in the reference directions, defined as follows:

$$C_{Fx,y,z} = \frac{F_{x,y,z}}{\bar{q}_h BH} \tag{2}$$

being $B = 0.27$ m and $H = 2.72$ m the reference dimensions in model scale. As a convention all force coefficients are referred to the mean wind speed at deck height $h = 0.35$ m, corresponding to a wind pressure $\bar{q}_h$ equal to:

$$\bar{q}_h = \frac{1}{2}\rho \overline{U}_h^2 \tag{3}$$

The wind load analysis is carried out in turbulent flow condition.

Forces measured by balance A correspond to the actions applied to the foundation of the tower, while balance B gives the forces at the anchorage of the cables to the ground. Fig. 6(a) shows the mean force coefficients versus the wind exposition angle α. Some comments can be done about the values of the coefficients along the bridge direction CF_{xA} and in the vertical direction CF_{zA}, which show a maximum at $\alpha \cong 0$ deg. This load condition is mainly caused by the forces transmitted by the deck through the cables. The negative lift force acting on the deck (Fig. 3) is higher for this direction and it is partially transferred to the tower by the cables, resulting in a pulling force acting on the tower in the cables direction. The drag force on the stays also contribute to increase their tension. This load configuration is also clear on the balance B, Fig. 7(a), where at $\alpha \cong 0$ deg the maximum negative values of the coefficients CF_{xB} and CF_{zB} indicate a maximum tension of the stay cables.

Figs. 6 and 7 indicate that in the range of wind exposure angles between $(-90 < \alpha < -60)$ and $(60 < \alpha < 90)$ the x and z direction mean forces at balances A and B (pylon foundation and back-stays anchorage) tend toward values close to zero. This is due to the fact that at those exposure angles the wind load on the deck and cables decreases as their exposure to wind is low. Hence, the bridge basically under self-weight plus the wind action on the pylon which also offers lower values of wind exposed surface. This explanation is also consistent with the trend of the y direction forces on both balances which are closely related with the drag forces on cables and deck.

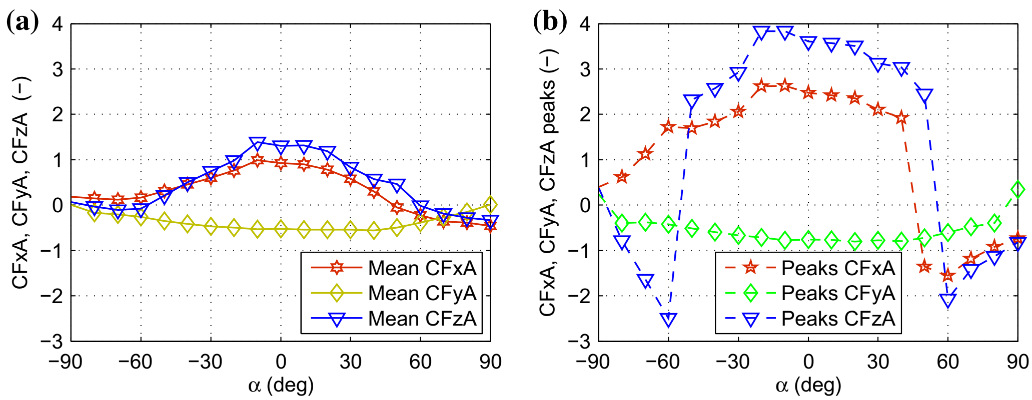

Fig. 6. Balance A. Forces coefficients versus wind exposure angle α, *TF*. (a) Mean values. (b) Peaks values.

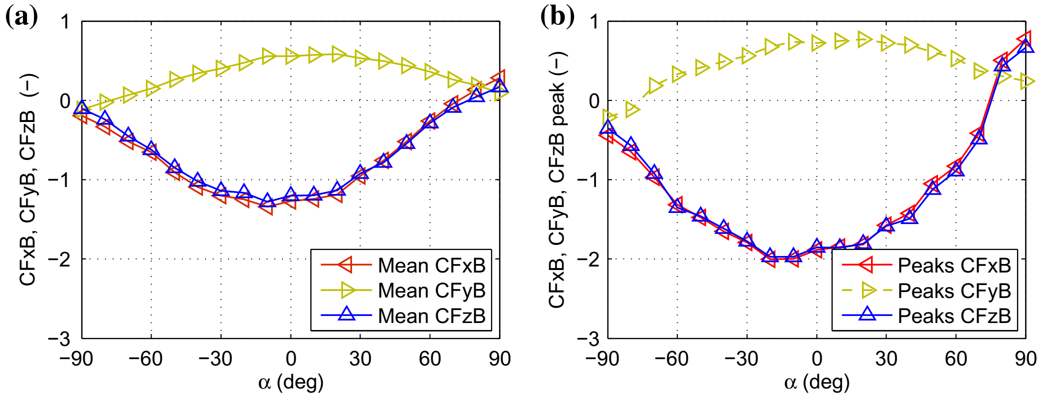

Fig. 7. Balance B. Forces coefficients versus wind exposure angle α, *TF*. (a) Mean values. (b) Peaks values.

Figs. 6(b) and 7(b) shows the peak values of the coefficients evaluated using the method of the peak factor for Gaussian process [4]. Also these values are in agreement with the previous analysis and in particular the peak actions at the base of the tower are fairly greater than the action at the cables anchorages.

Conclusions

A complete investigation on the wind response of the Talavera de la Reina bridge was carried out by means of sectional model and full aeroelastic model wind tunnel tests. This work, part of a more extended study, is focused on the wind loads at the tower foundation and the cable anchorages. The analysis shows the load distributions which can be used by the bridge manufacturer as a support for the bridge design.

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