Aeroelastic studies of a cable stayed bridge in Talavera de la Reina, Spain

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Abstract

The cable stayed bridge of Talavera de la Reina in central Spain is a signature structure of a high aesthetic quality, the main span is 318 m long and the single pylon reaches 164 m of height. Due to the relevance of the bridge a study of the wind characteristics at the location and a set of aerodynamic studies both experimental and computational were carried out to anticipate the future response of the bridge under wind flow. They comprised a boundary layer wind tunnel test of the stand alone pylon and the full bridge, a test of a reduced model of a segment of the deck in an aerodynamics wind tunnel and computational analysis to obtain the flutter speed of the bridge. This paper describes the bridge and the results of the mentioned studies that allowed us to conclude that the bridge could perform safely while subjected to the expected wind speed at its location.
Keywords: cable stayed bridges, boundary layer wind tunnel, aeroelastic studies, flutter speed.

1 The Talavera de la Reina bridge

This cable stayed bridge is located south of the Talavera de la Reina city in central Spain; the purpose of the construction is to provide a crossing over the Tagus river. The structure is a quite singular design with a concrete deck of aerodynamic shape, a main span of 318 m and a single pylon 164 m tall. The cable system is 3-D with the deck supported by two planes of cables and another pair of families of rear cables balancing the pylon and anchored in massive concrete blocks. Figure 1 shows some details of the bridge.

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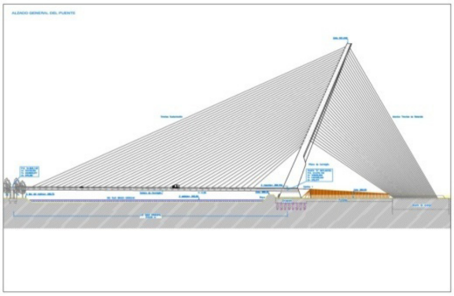
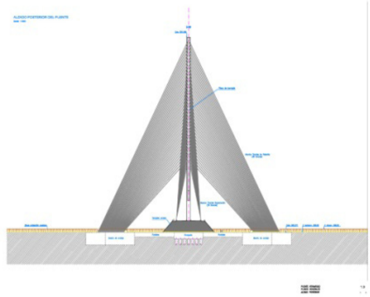
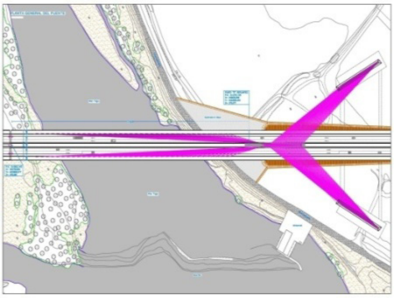


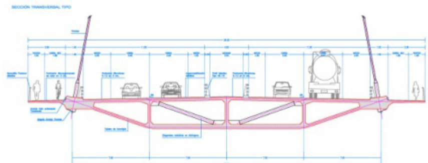
a) Elevation



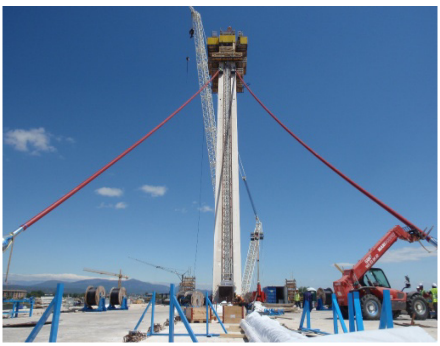
b) Rear view



c) Plan view



d) Bridge deck



e) Pylon view

f) Full bridge view

Figure 1: Geometry of the cable stayed Talavera bridge.

2 Aerodynamic studies

Taking in account the relevance of the bridge several studies related to the influence of the wind in the structural behaviour were carried out they are listed below

a) Wind characteristics at bridge site.
b) Test of stand alone pylon.

c) Aerodynamic coefficients of bridge deck.
d) Evaluation of flutter speed of bridge.

3 Wind characteristics at bridge site

A study of wind properties at bridge site was very necessary [1]. It had to provide the maximum speed values expected for the return period required. The vertical profile of wind velocity was also relevant as the tall pylon was to stand alone during several months during the construction works. Additionally the identification of the wind rose was very useful to find out the wind flow orientations that were move probe to occur.

Daily data of wind speed data came from a weather station nearby of bridge location and they were recorded at a high of 10 m. A threshold value of 30 km/h was defined and only events of higher speed were considered in the study. Hence, up to 101 data in a time period of 11 years were considered and they appear in table 1.

Table 1: Vertical wind profile for different return periods.

T = 200 years		*T* = 50 years		*T* = 5 years	
z (m)	U (km/h)	z (m)	U (km/h)	z (m)	U (km/h)
0	0.00	0	0.00	0	0.00
5	108.73	5	96.87	5	77.12
10	122.88	10	109.49	10	87.16
15	131.18	15	116.88	15	93.04
20	137.07	20	122.13	20	97.22
25	141.64	25	126.20	25	100.46
30	145.37	30	129.53	30	103.11
35	148.53	35	132.34	35	105.35
40	151.27	40	134.78	40	107.29
45	153.68	45	136.93	45	109.00
50	155.84	50	138.85	50	110.53
55	157.79	55	140.59	55	111.92
60	159.58	60	142.18	60	113.18
65	161.22	65	143.64	65	114.35
70	162.73	70	145.00	70	115.42
75	164.15	75	146.26	75	116.43
80	165.47	80	147.43	80	117.37
85	166.71	85	148.54	85	118.25
90	167.89	90	149.59	90	119.08
95	168.99	95	150.57	95	119.86
100	170.05	100	151.51	100	120.61
105	171.05	105	152.40	105	121.32
110	172.00	110	153.25	110	121.99
115	172.91	115	154.06	115	122.64
120	173.78	120	154.84	120	123.26
125	174.62	125	155.59	125	123.85
130	175.42	130	156.30	130	124.42
135	176.20	135	156.99	135	124.97
140	176.94	140	157.65	140	125.50
145	177.66	145	158.30	145	126.01
150	178.36	150	158.91	150	126.50
155	179.03	155	159.51	155	126.98
160	179.68	160	160.09	160	127.44
165	180.31	165	160.66	165	127.89
170	180.92	170	161.20	170	128.32
175	181.52	175	161.73	175	128.75
180	182.09	180	162.24	180	129.15
185	182.65	185	162.75	185	129.55
190	183.20	190	163.23	190	129.94
195	183.73	195	163.71	195	130.32
200	184.25	200	164.17	200	130.69

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Wind speed U at the reference altitude from the ground of 10 m was fitted to a Gumbel distribution defined as

$$F(U) = e^{\frac{-U-a}{b}} \tag{1}$$

Data fitting was obtained with a value of correlation coefficient of $R^2 = 0.99$ Parameters a, b. turn out

$$a = 50.292 \qquad b = 0.653 \tag{2}$$

Having obtained the parameters of the expression of $F(U)$ in (1) the wind speed U_T corresponding to a return period T can be written as

$$U_T = a - b\ln(-\ln\left(1 - \frac{n}{NT}\right) \tag{3}$$

where n = 11 is the number of the years included in the study, N the number of events and T the years of the return period.

Vertical profile of wind was defined by the expression

$$U(z) = \frac{U^*}{k}\ln\frac{z}{z_0} \tag{4}$$

where k = 0.4 is the well known Von Karman constants, z_0 is roughness length for which a value of z_0= 0.025 m was selected according with the topographical conditions.

The values of $U(z)$ at z^*= 10 m for any return period T are obtained in expression (3). Therefore the value of U_T^* can be identified easily as

$$U_T^* = \frac{kU_T(z^*)}{\ln\frac{z^*}{z_0}} \tag{5}$$

Entering with U_T^* in expression (4) the vertical profile of wind speed for each return period T can be evaluated. In table 2 values for several cases are shown.

Table 2: RMS of accelerations at pylon top.

	Return period T= 5 years	Return period T= 500 years
Stage 1	0.46 m/s^2	1 m/s^2
Stage 2	0.338 m/s^2	0.975 m/s^2

The monthly wind rose describing the angular distribution of wind flow appears in figure 2. Yearly wind rose and bridge position are in figure 3 and it shows that the most frequent wind orientation is almost perpendicular to the bridge.

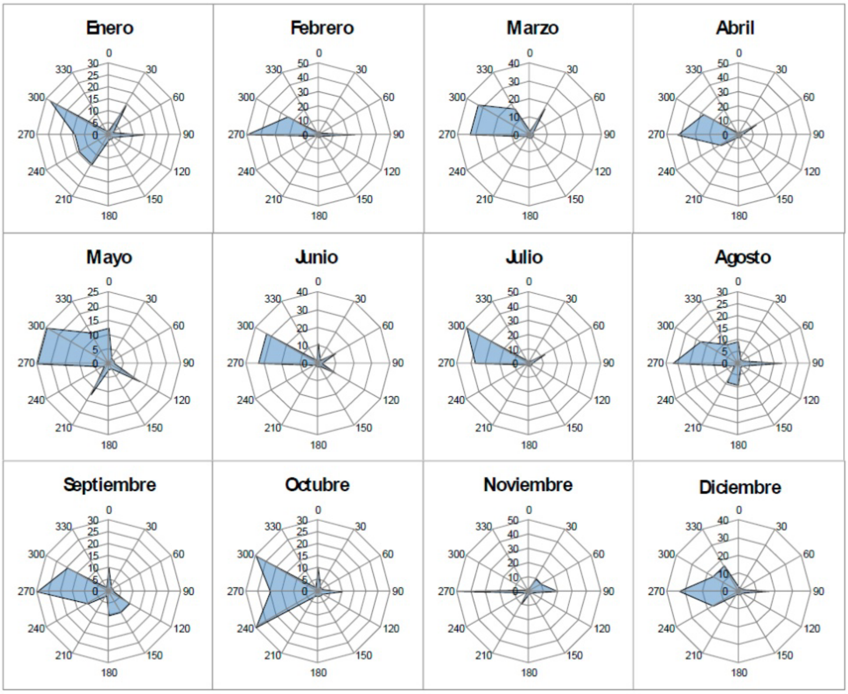
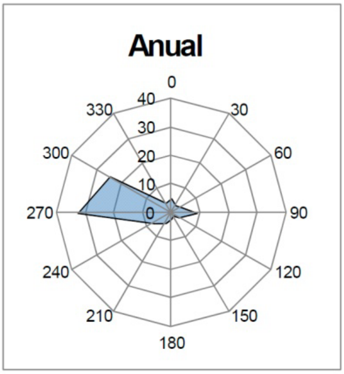
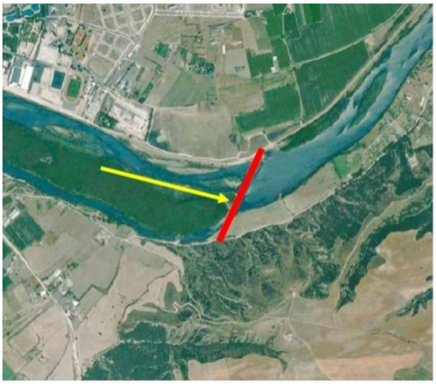


Figure 2: Monthly wind rose.



a) Yearly wind rose



b) Bridge location

Figure 3: Yearly wind rose and bridge location.

4 Test of stand alone pylon

The construction procedure of the cable stayed bridge contained phases in which the pylon is partially built up to a height of 71.44 m and is not connected to any cable, thus it behaves as a vertical cantilever under wind flow. Afterwards, as the pylon construction progresses some cables are placed in the pylon and connected to the bridge deck, so the pylon stiffness is increased. A study of the erection steps of the pylon showed that the two critical situations where when it reached the maximum cantilever height and also the next phase when, after building a new pylon module, and therefore its height mounted up to 79.46 m the pylon was connected with two cables to bridge deck. Both situations are presented in figure 4.


a) Maximum cantilever height

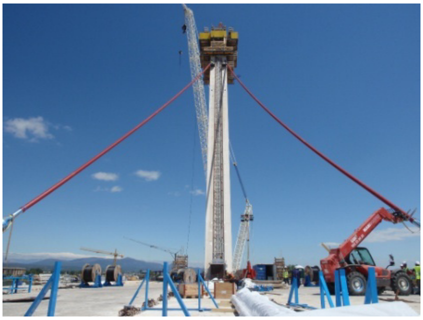
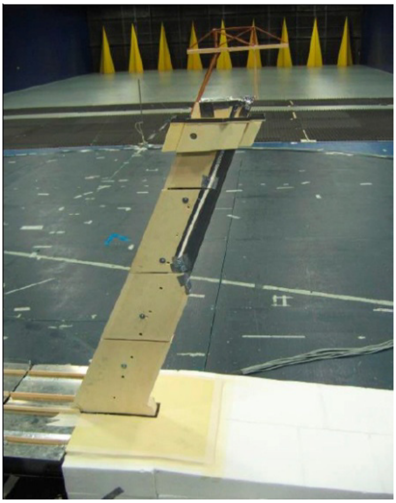
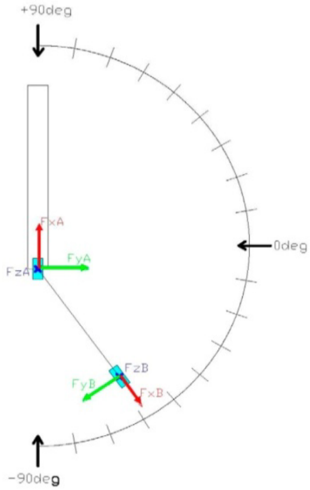

b) Pylon connected with two cables

Figure 4: Critical phases of pylon construction.

Considering this information a test of the pylon was carried out in the boundary layer wind tunnel of the Politecnico de Milano [2] using a reduced model with a geometrical scale of 1/64. Figure 5a) shows a detail of the reduced


a) Pylon with formwork and crane



b) Convention signs in experiment

Figure 5: Test of the pylon in the wind tunnel.

model of the pylon that incorporates details of the equipment existing at the top; also the convention sign adopted in the experiment is shown in figure 5b).

The reduced model was tested in two configurations entitled Stage 1 (stand alone pylon of 71.44 m) and Stage 2 (pylon of 79.46 m connected with two cables). Wind speeds in the experiment corresponded to those of 5 years and 500 years return period for actual wind. The pylon exhibited vibrations under both flows being larger when wind was in *x* direction and vibrations occurred in the perpendicular axis, namely on *y* directions according to the sign convention.

The RMS of accelerations at pylon top appears in table 3.

Table 3: Data of reduced model and springs.

	SECTIONAL MODEL MASS			3.2 kg		
	SECTIONAL MODEL INERTIA			5.1 kgm^2		
	MODEL WIDTH			0.36 m		
	DISTANCE d			0.47 m		
	VERTICAL SPRINGS		HORIZONTAL	MODEL FREQUENCIES (rad/s)		
	TOP	BOTTOM	SPRING	LATERAL	VERTICAL	TORSIONAL
TEST 1	130 N/m	96 N/m	102 N/m	13.6	16.6	35.7
TEST 2	600 N/m		300 N/m	21.1	26.1	67.9

No proper regulation exists to define the allowable upper value of this type of accelerations but it is quite common to accept a maximum value of 0.5 m/s^2. In that regards it looks that the pylon is safe for both construction stages for the five years return period wind and it is not for the T =500 wind speed. The test showed that the critical situation is when the pylon is at its maximum stand alone height.

It may be reminded that the values of accelerations in table 3 corresponded to wind in direction *x* and the wind rose presented in figure 3.a indicates that this is not the most frequent wind direction. Additionally, the idea of checking the behaviour of the isolated pylon under a wind speed of return period T=500 years looks too much protective taken in account that the pylon construction only lasts a few months. It seems that a return period of T = 5 years is more sensible.

5 Evaluation of flutter speed of bridge

Evaluation of bridge behaviour under laminar flow and the wind speed leading to instability was carried out by two different procedures.

5.1 Evaluation by a hybrid method

In this approach flutter speed was identified by a method consisting in a two step procedure. In the first one a reduced model a segment of bridge deck was tested in the wind tunnel of the University of Coruña. The geometric scale of the model was 1/100 with a span to width ratio of 3 and the Reynolds number of the experiment was Re= 2.16E+0.5. Figure 6 shows a view of the wind tunnel with the test chamber in the forefront and a picture of the reduced model.

The aim of the text was to obtain the complete set of flutter derivatives defined by the Scalan formulation. The reduced model was placed in the test

a) View of the wind tunnel b) View of the reduced model of bridge deck

Figure 6: Wind tunnel test of a segment bridge deck.

chamber hanging on several springs as described in figure 7. Two different set of springs were used in order to allow a wider interval of wind velocities that ranged from 6 m/s to 15 m/s. Data of reduced model and springs appear in table 4.

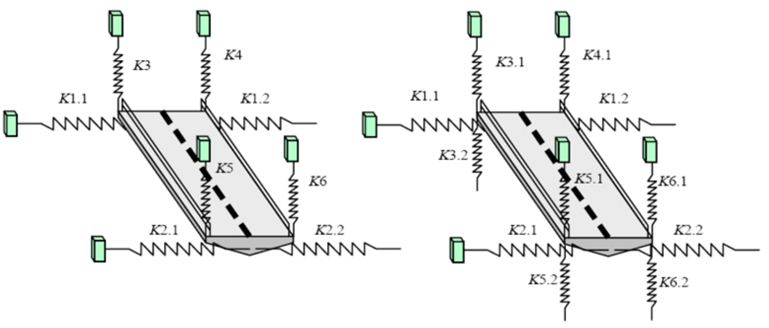


Figure 7: Suspension system of reduced model deck.

Figures 8 to 10 show the results of the $A_i^*, H_i^*, P_i^* (i=1,...6)$ flutter derivatives.

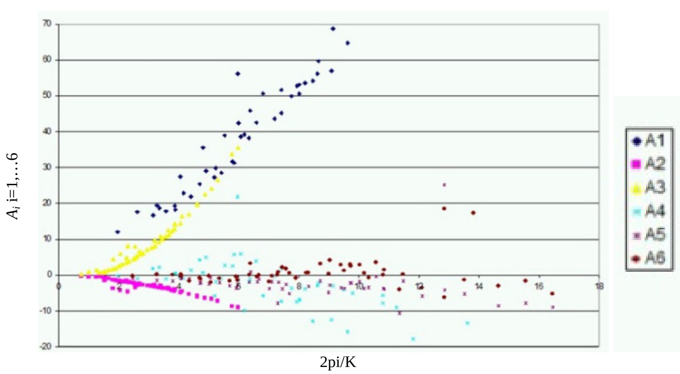


Figure 8: Values of A_i^* $(i=1,...6)$.

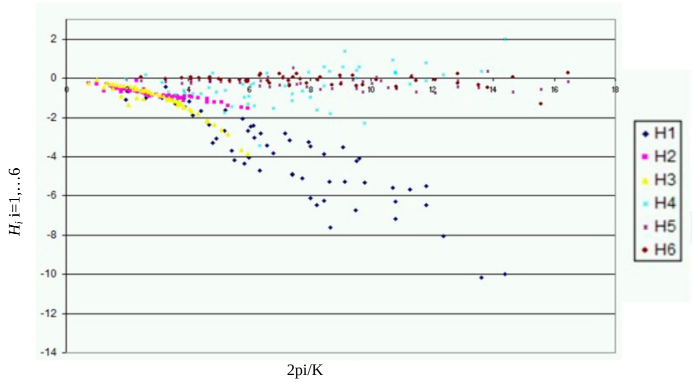


Figure 9: Values of H_i^* $(i=1,\ldots 6)$.

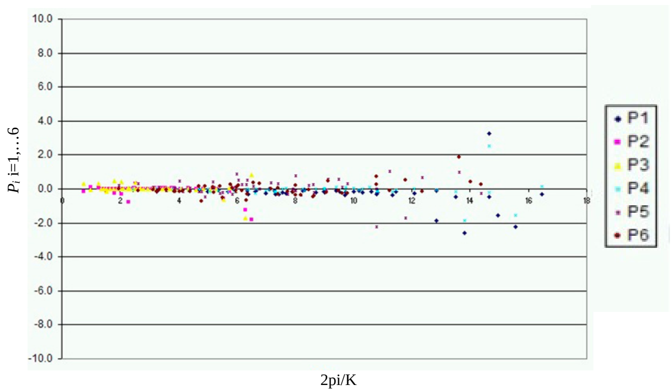


Figure 10: Values of P_i^* $(i=1,\ldots 6)$.

The set of flutter derivatives allows us to formulate the aeroelastic forces according to expression (6).

$$\mathbf{f}_a = \begin{Bmatrix} D_a \\ L_a \\ M_a \end{Bmatrix} = \frac{1}{2}\rho U^2 KB \cdot \begin{pmatrix} P_1^* & -P_5^* & -BP_2^* \\ -H_5^* & H_1^* & BH_2^* \\ -BA_5^* & BA_1^* & B^2A_2^* \end{pmatrix} \begin{Bmatrix} \dot{v} \\ \dot{w} \\ \dot{\varphi}_x \end{Bmatrix} + \frac{1}{2}\rho U^2 K^2 \cdot \begin{pmatrix} P_4^* & -P_6^* & -BP_3^* \\ -H_6^* & H_4^* & BH_3^* \\ -BA_6^* & BA_4^* & B^2A_3^* \end{pmatrix} \begin{Bmatrix} v \\ w \\ \varphi_x \end{Bmatrix} \tag{6}$$

where v, w, φ_x and $\dot{v}, \dot{w}, \dot{\varphi}_x$ are deck displacement and velocities according to the convention showed in figure 11, U is wind speed, ω is the vibration frequency, k is the reduced frequency $k = \omega B/U$ and A_i^*, H_i^*, P_i^* $(i=1,\ldots 6)$ are the flutter derivatives.

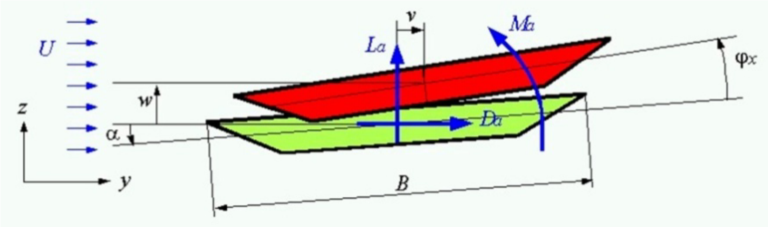


Figure 11: Convention signs for wind tunnel test.

After obtaining the flutter derivatives the dynamic equilibrium of the full bridge under wind flow can be expressed as

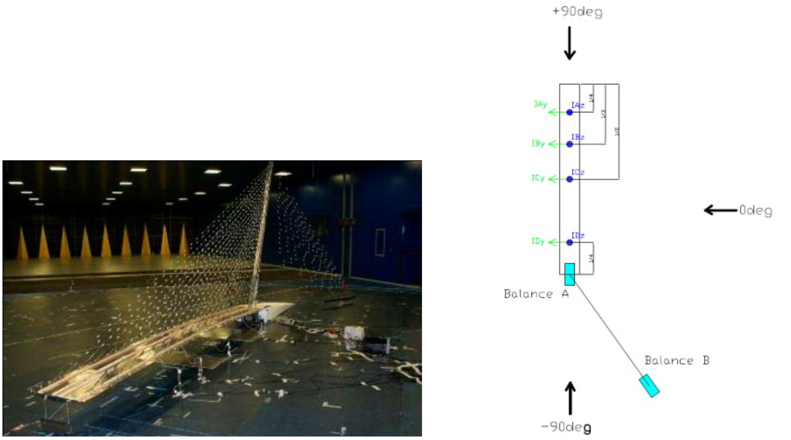
$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{f}_a = \mathbf{C}_a\dot{\mathbf{u}} + \mathbf{K}_a\mathbf{u} \tag{7}$$

Using modal superposition equation (7) leads to a non linear problem that needs to be solved for each wind speed *U*. It is well known that the solution consists on a set of complex eigenvalues whose real and imaginary components represent the damping and frequency of each eigenvector. When a damping becomes zero for a given wind speed the flutter speed is obtained.

Using the in-home code, FLAS [3] a flutter speed of 77.17 m/s was find out. This is a very high velocity that means that the bridge is very stable aeroelastically.

5.2 Test of full bridge under laminar flow

A reduced model of the full bridge at a geometric scale 1/64 was tested in the boundary layer wind tunnel of the Politecnico di Milano [2].



a) View of the reduced model b) Location of the accelerometers

Figure 12: Picture of the model and locations of the measurement devices.

The aim of the test in laminar flow was to investigate the damping effect as it usually increases for small wind speed and then this tendency changes for higher wind velocities and eventually it can become zero leading to incipient flutter.

Initial test showed a value of structural damping of 0.85% for the first eigenvector that was a flexural mode as it appears in figure 13.

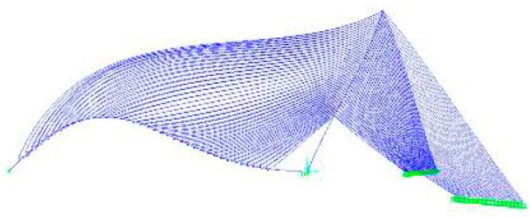


Figure 13: First flexural model of bridge.

Deformation of bridge under different wind speed and orientation was measured at positions A and B that are closer to the point of maximum deformation for the first flexural mode. Figure 14 and 15 shows the values of total damping (structural play aerodynamic) for increasing wind speed up to 7m/s. It must be borne in mind that given the geometrical scale of 1/64 that speed represents a real wind velocity of 56 m/s or 201.6 km/h. It can be observed that for that range of wind speed total damping increases and there is no danger of instability. This result is consistent with the conclusion obtained in the computational calculations and therefore it can be said that the design of the bridge was very safe regarding flutter.

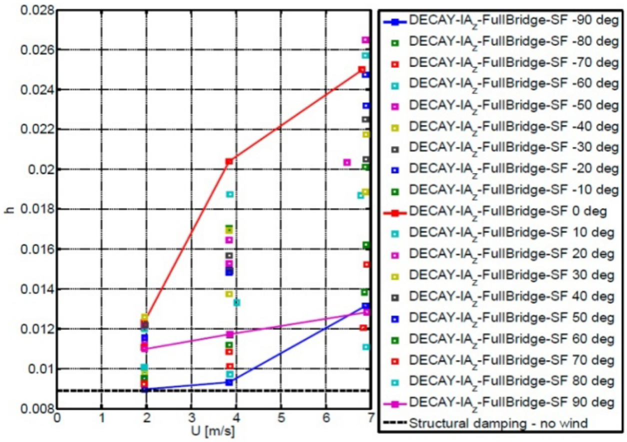


Figure 14: Total damping at location A.

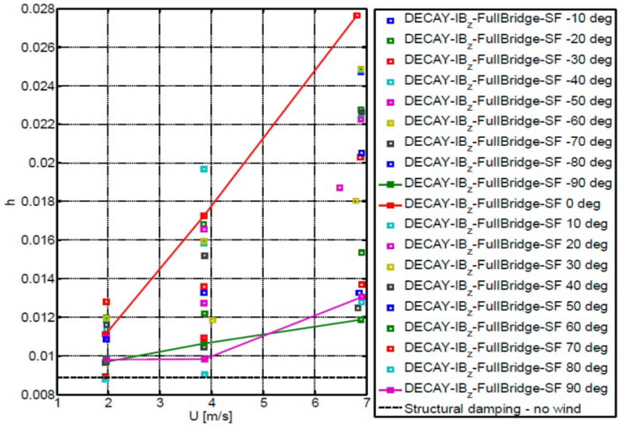


Figure 15: Total damping at location B.

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