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Computational analysis of VIV of a pedestrian suspension bridge with a span of 250 m combined with wind tunnel test data

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SUMMARY

Cable-supported bridges built in recent decades have increased the importance of studying the VIV phenomenon. Most research is based on experimental or computational studies intended to identify the presence of lock-in in sectional model tests of bridge decks or in CFD simulations, using this information to infer the response of the real structure. The aim of this presentation is to introduce a procedure that allows the transition from the sectional model level to the full bridge geometry and to evaluate the magnitude of the responses during VIV. The process consists of two steps: an experimental step to identify the wind loads and aerodynamic damping of the deck using wind tunnel tests, and a computational step involving a dynamic analysis of the complete bridge in the frequency domain to obtain the RMS of the structural responses. The procedure was applied to a pedestrian suspension bridge between Spain and Portugal.

Keywords: *VIV, wind tunnel test, aerodynamic damping, dynamic analysis, bridge between Spain and Portugal.*

1. INTRODUCTION

VIV is an important aerodynamic phenomenon whose relevance has increased due to the increasingly slender designs of cable-supported bridges constructed in recent decades. Vickery and Basu (1983) proposed a pioneering approach widely used to describe the main features of this phenomenon, including the self-limiting values of bridge deck displacements. Their formulation is based on the existence of an aerodynamic damping that reduces structural damping near the wind velocities that generate vortices with frequencies close to the natural frequencies of the bridge. Other authors, Larsen (1995) and Lupi (2017) have also contributed to the study of VIV in bridges and wind turbine towers, introducing modifications to the formulation of aerodynamic damping.

Most research conducted to date has been experimental, based on wind tunnel tests of sectional models of bridge decks or other line-like structures, with the objective of detecting lock-in and extrapolating this behaviour to real bridges. The objective of this paper is to extend the study from the sectional model to the full bridge. The approach requires two steps: the first relies on sectional model tests to identify the Strouhal number, wind load spectra, and aerodynamic damping; the second is a computational analysis of the full bridge using the wind load spectra obtained experimentally. The procedure is described in the next section and is applied to a 250 m pedestrian and cyclist suspension bridge between Spain and Portugal (Figure 1).

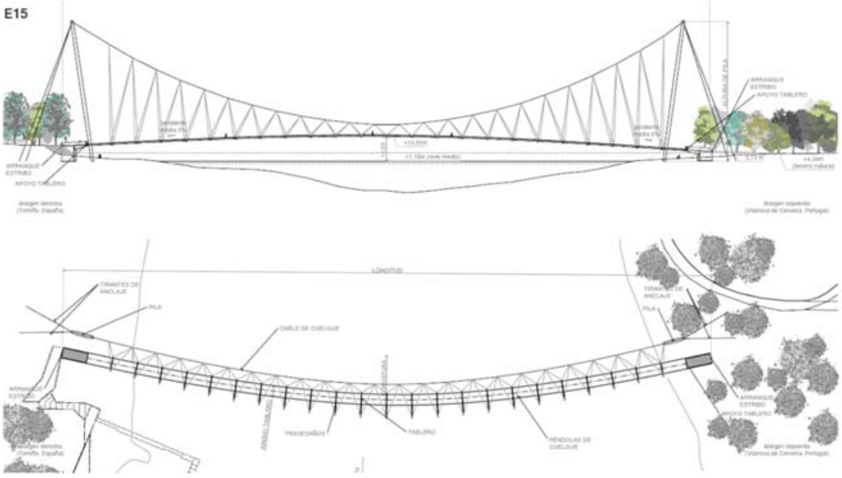


Figure 1: Suspension bridge between Spain and Portugal.

2. COMPUTATIONAL FORMULATION

This phase of the process consists on establishing the dynamic equation of the bridge under wind flow as expressed in (2.1).

$$\mathbf{M}\ddot{\mathbf{u}} + \left(\mathbf{C}_{\mathrm{s}} - \mathbf{C}_{\mathrm{a}}\right)\dot{\mathbf{u}} + \mathrm{K}\mathbf{u} = \mathbf{f}_{\mathrm{w}}\left(\mathrm{t}\right) \tag{2.1}$$

Where **M**, $\mathbf{C}_s$, **K** are the mass, structural damping and stiffness matrices, **u**, **u**, $\ddot{\mathbf{u}}$ are the displacements, velocities and acceleration of the bridge, $\mathbf{f}_w$ represents the wind forces and $\mathbf{C}_a$ is the aerodynamic damping that appears during VIV. Using modal decomposition and mass-normalized eigenvectors the expression (2.1) becomes

$$\mathbf{I}\ddot{\mathbf{q}} + \left(\bar{\mathbf{C}}_{\mathrm{sR}} - \mathbf{\Phi}^{\mathrm{T}}\mathbf{C}_{\mathrm{a}}\mathbf{\Phi}\right)\dot{\mathbf{q}} + \bar{\mathbf{K}}_{\mathrm{R}}\mathbf{q} = \mathbf{\Phi}^{\mathrm{T}}\mathbf{f}_{\mathrm{w}} \tag{2.2}$$

Where **I** is the identity matrix, $\bar{\mathbf{C}}_{\mathrm{sR}}$ and $\bar{\mathbf{K}}_{\mathrm{R}}$ are diagonal matrices with $\bar{\mathbf{C}}_{\mathrm{sRii}} = 2\zeta_{\mathrm{i}}\omega_{\mathrm{i}}$ and $\bar{\mathbf{K}}_{\mathrm{Rii}} = \omega_{\mathrm{i}}^2$ respectively, ζ_{i} is the modal damping ratio. Taking the Fourier transform of expression (2.2)

$$\left(\mathbf{K}_{\mathrm{R}} - \omega^2\mathbf{I} + \mathrm{i}\omega\left(\mathbf{C}_{\mathrm{R}} - \mathbf{\Phi}^{\mathrm{T}}\mathbf{C}_{\mathrm{a}}\mathbf{\Phi}\right)\right)\tilde{\mathbf{q}}\left(\omega\right) = \mathbf{V}\left(\omega\right) = \mathbf{\Phi}^{\mathrm{T}}\tilde{\mathbf{f}}_{\mathrm{w}} \tag{2.3}$$

Where $\tilde{\mathbf{q}}$ and $\tilde{\mathbf{f}}_{\mathrm{w}}$ are the Fourier transform of vectors **q** and $\mathbf{f}_w$. Thus

$$\tilde{\mathbf{q}}\left(\omega\right) = \mathbf{V}\left(\omega\right)^{-1}\mathbf{\Phi}^{\mathrm{T}}\tilde{\mathbf{f}}_{\mathrm{w}}\left(\omega\right) = \mathbf{H}\left(\omega\right)\mathbf{\Phi}^{\mathrm{T}}\tilde{\mathbf{f}}_{\mathrm{w}}\left(\omega\right) \tag{2.4}$$

Given the nature of wind loads, the dynamic analysis is carried out using the PSD of the wind forces $\mathbf{S}_{fw}$, which allows computation of the PSD of the structural responses $\mathbf{S}_u$ using the following expression

$$\mathbf{S}_{\mathrm{u}} = \mathbf{\Phi}\mathbf{H}\left(\mathbf{\Phi}^{\mathrm{T}}\mathbf{S}_{\mathrm{fw}}\mathbf{\Phi}\right)\mathbf{H}^{*\mathrm{T}}\mathbf{\Phi}^{\mathrm{T}} \tag{2.5}$$

Where $\mathbf{H}^{*T}$ is he complex conjugate of matrix **H**. The RMS values of displacements, velocities and accelerations of bridge deck are then obtained using

$$\sigma_{ui} = \sqrt{\int_0^\infty S_{uii} df}; \qquad \sigma_{\dot{u}i} = \sqrt{\int_0^\infty 4\pi^2 f^2 S_{uii} df}; \qquad \sigma_{\ddot{u}i} = \sqrt{\int_0^\infty 16\pi^4 f^4 S_{uii} df}; \tag{2.6}$$

The formulation presented requires defining wind loads and aerodynamic damping. The spectra of wind loads per unit length is usually presented by

$$S_{pz} = \frac{\left(\frac{1}{2}\rho V_S^2 B^2\right)^2 \hat{\sigma}_{pz}^2}{\sqrt{\pi}\omega_s} \cdot exp\left(-\left(\frac{1-\frac{\omega}{\omega_s}}{b_z}\right)\right) \qquad S_{p\theta} = \frac{\left(\frac{1}{2}\rho V_S^2 B^2\right)^2 \hat{\sigma}_{p\theta}^2}{\sqrt{\pi}\omega_s} \cdot exp\left(-\left(\frac{1-\frac{\omega}{\omega_s}}{b_\theta}\right)\right) \tag{2.7}$$

Where $$V_s = \frac{\omega_s D}{2\pi S_t} \tag{2.8}$$

S_t is the Strouhal number, B, D are the deck width and depth, $\hat{\sigma}_{pz}^2$ and $\hat{\sigma}_{p\theta}^2$ are the RMS values of the dimensionless aerodynamic coefficients and b_z and b_θ non-dimensional load spectrum bandwith parameters. The aerodynamic damping $\mathbf{C}_a$ associated to vertical displacement and torsional rotation has several formulations, Vickery (1983), Larsen (1995), Lupi et alt. (2017) or Strommsen (2010). In this work:

$$C_{azi} = \frac{\rho B^2 \omega_i K_{az}}{2} \cdot \left(1 - \left(\frac{\sigma_z}{a_z D}\right)^2\right) \qquad C_{a\theta i} = \frac{\rho B^2 \omega_i K_{a\theta}}{2} \cdot \left(1 - \left(\frac{\sigma_\theta}{a_\theta}\right)^2\right) \tag{2.9}$$

Where

$$K_{az} = K_{az0}\left(\frac{V_i}{V}\right)^n \cdot exp\left(-\left(\frac{V_i}{V}\right)^m\right) \qquad K_{a\theta} = K_{a\theta 0}\left(\frac{V_i}{V}\right)^n \cdot exp\left(-\left(\frac{V_i}{V}\right)^m\right) \tag{2.10}$$

V_i and ω_i are related through (2.8) and K_{az0}, $K_{a\theta 0}$, a_z, a_θ, n, m are dimensionless coefficients associated with the deck shape and σ_z, σ_θ are RMS values of vertical displacement and torsional rotation of the bridge deck nodes. The Strouhal number and the parameters required for the wind spectra and the aerodynamic damping are obtained from sectional model tests of bridge deck of CFD simulations; here, the experimental approach is used.

3. APPLICATION TO A SUSPENSION BRIDGE

A sectional model of the bridge in figure 1 built at a geometrical scale of 1/15 was tested in the wind tunnel of the University of Coruña. Figure 2 shows a picture in the test and figure 3 presents the relationship between wind speed and vortex shedding frequency yielding to a value of the Strouhal number of $S_t = 0{,}166$.

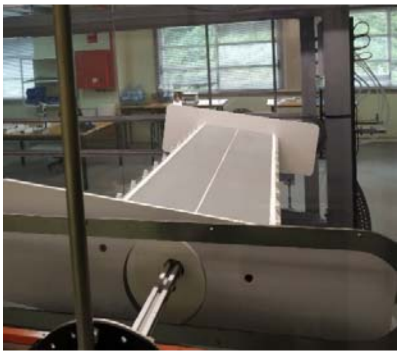


Figure 2: Sectional model in the test chamber.

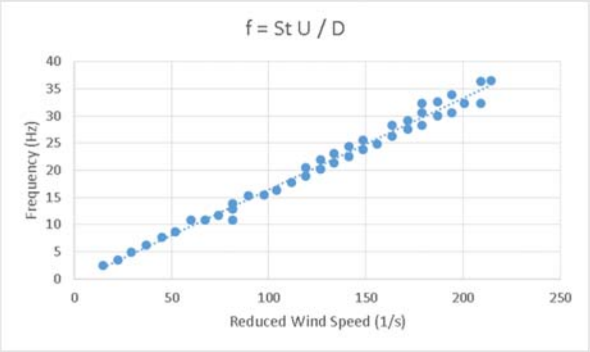


Figure 3: Graph (f, V^*) obtained in the test.

More tests are currently underway to identify the spectra of wind loads and aerodynamic damping. A finite element model of the bridge is shown in figure 4. The natural frequencies obtained from the modal analysis are listed in table 1.

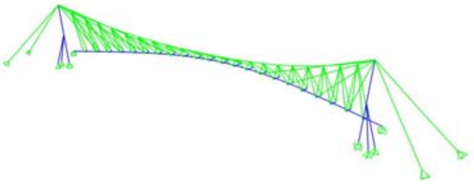
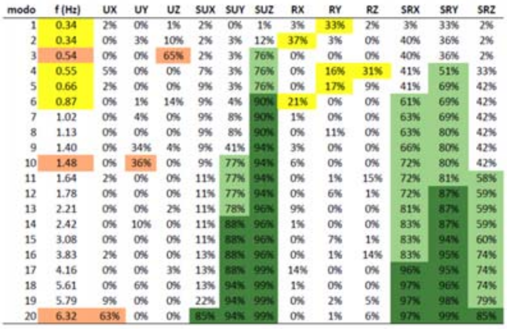


Figure 4: FEM of the suspension bridge.

Table1: Natural frequencies of bridge



modo	f (Hz)	UX	UY	UZ	SUX	SUY	SUZ	RX	RY	RZ	SRX	SRY	SRZ
1	0.34	2%	0%	1%	2%	0%	1%	3%	33%	2%	3%	33%	2%
2	0.34	0%	3%	10%	2%	3%	12%	37%	3%	0%	40%	36%	2%
3	0.54	0%	0%	65%	2%	3%	76%	0%	0%	0%	40%	36%	2%
4	0.55	5%	0%	0%	7%	3%	76%	0%	16%	31%	41%	51%	33%
5	0.66	2%	0%	0%	9%	3%	76%	0%	17%	9%	41%	69%	42%
6	0.87	0%	1%	14%	9%	4%	90%	21%	0%	0%	61%	69%	42%
7	1.02	0%	4%	0%	9%	8%	90%	1%	0%	0%	63%	69%	42%
8	1.13	0%	0%	0%	9%	8%	90%	0%	11%	0%	63%	80%	42%
9	1.40	0%	34%	4%	9%	41%	94%	3%	0%	0%	66%	80%	42%
10	1.48	0%	36%	0%	9%	77%	94%	6%	0%	0%	72%	80%	42%
11	1.64	2%	0%	0%	11%	77%	94%	0%	1%	15%	72%	81%	58%
12	1.78	0%	0%	0%	11%	77%	94%	0%	6%	1%	72%	87%	59%
13	2.21	0%	0%	2%	11%	78%	96%	9%	0%	0%	81%	87%	59%
14	2.42	0%	10%	0%	11%	88%	96%	1%	0%	0%	83%	87%	59%
15	3.08	0%	0%	0%	11%	88%	96%	0%	7%	1%	83%	94%	60%
16	3.83	2%	0%	0%	13%	88%	96%	0%	1%	14%	83%	95%	74%
17	4.16	0%	0%	3%	13%	88%	99%	14%	0%	0%	96%	95%	74%
18	5.61	0%	6%	0%	13%	94%	99%	1%	0%	0%	97%	96%	74%
19	5.79	9%	0%	0%	22%	94%	99%	0%	2%	5%	97%	98%	79%
20	6.32	63%	0%	0%	85%	94%	99%	0%	1%	6%	97%	99%	85%

The ongoing test will provide the values of the coefficients of expressions (2.7), (2.9) and (2.10) and afterwards, expressions (2.5) and (2.6) will be employed to compute the dynamic responses of the bridge deck for different values of wind speed. The complete results will be included in the full paper and presented at the conference.

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