

# Aerodynamic Interference Between Two Closely Spaced Cable Supported Bridges

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ABSTRACT: This paper presents the results of wind-tunnel tests conducted to evaluate the effects of wind on a new cable-stayed bridge and to identify possible aerodynamic interaction effects between the new bridge and an existing suspension bridge. The bridges are spanning the Ozama River in the capital of the Dominican Republic, Santo Domingo.

## 1 INTRODUCTION

A cable-stayed bridge with a main span of 180 m is planned for the Dominican Republic. The new structure will relieve an existing suspension bridge, Puente Juan Pablo Duarte, open for traffic in 1955. To share the road infrastructure, the two bridges will be situated in parallel, 41.8 m apart (centre line to centre line) with less than 16 m from edge to edge.

In the aerodynamic study of the new bridge, possible interference effects between the two bridges were of concern to the local road authority and the designers. In storm winds, the possibility that the much lighter and flexible suspension bridge be excited when in the wake of the new bridge had to be investigated. Experience had shown that such interaction can be important (see Grillaud *et al.*, 1991) and could force the installation of aerodynamic appendages on a bridge deck that would otherwise not have been necessary. The designers of the new bridge had also to evaluate the wind loading along the span of the new structure for the particular flow field created by the old bridge.

The Danish Maritime Institute (DMI) was commissioned to perform the wind-tunnel study by Carlos Fernandez Casado S.L., Madrid, Spain. The study included static and dynamic section model tests where both bridge decks were modelled at the same time in the wind tunnel. To simplify the experiments, only one bridge at a time was modelled dynamically. This allowed to select an adequate velocity scaling for each of them since

the old bridge has fundamental frequencies approximately 50% lower than the new bridge. This approach was selected to indicate if major adverse interference effects existed. If so, a full 3-D aeroelastic representation of both structures would have been advisable since motions of the upstream bridge could impose organised oscillation of the flow approaching the downwind structure.

## 2 FULL-SCALE STRUCTURES

The new cable-stayed bridge spanning the Ozama River in Santo Domingo is presently under construction and will have a main span of 180 m and two cable-supported side spans of each 83.45 m, both with an intermediate pier support, for a total length of cable-supported structure of 346.90 m. The bridge has two vertical cable planes with a distance (plane-to-plane) of 32.6 m, and the cable configuration is of the modified fan type, see figure 1.

The bridge will carry four lanes of vehicles, two rail tracks and two sidewalks. The bridge deck is a 33.5 m wide double-box concrete girder as depicted in figure 2, and the deck height is 3.6 m, including the central jersey barrier.

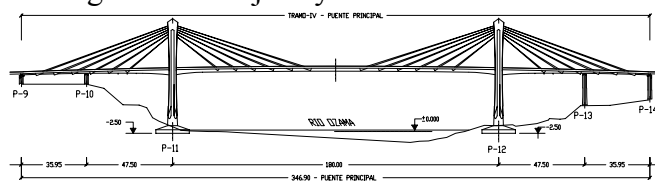


Figure 1: View of the new cable-stayed bridge

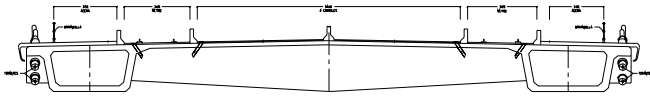


Figure 2: Cross section of the new cable-stayed bridge

The existing bridge (Puente Juan Pablo Duarte) is a suspension bridge with a main span of 176 m and side spans of 74 m and 101 m, respectively. The side spans are not cable-supported but are supported by one or two piers as shown on figure 3. The stiffening girder is a steel truss girder. For the main span the truss girder is 17.8 m wide and 2.6 m high, and carries a two-lane dual carriageway and two sidewalks, see figure 4.

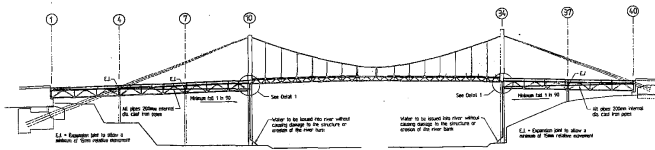


Figure 3: View of existing suspension bridge

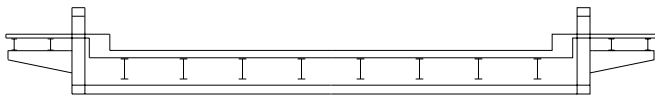


Figure 4: Cross-section of the existing bridge

The existing bridge was constructed in 1955 and has recently undergone extensive rehabilitation. Table 1 presents the dynamic characteristics of both bridges.

	New bridge	Existing bridge
$f_{\text{vertical}}$ (Hz)	0.630	0.370
$f_{\text{torsion}}$ (Hz)	0.883	0.478
$f_{\text{torsion}}/f_{\text{vertical}}$	1.40	1.29
Mass per unit length (kg/m)	61,570	13,680
Mass moment of inertia per unit length (kg m <sup>2</sup> /m)	$7.779 \times 10^6$	$0.656 \times 10^6$

Table 1: Dynamic properties of prototype bridges

### 3 TEST OBJECTIVES

The main objectives of the tests were twofold: first to assess the aerodynamic performance of the new cable-stayed bridge, secondly to quantify the interference effects of the new bridge on the existing bridge and vice-versa. The following properties were investigated: 1) the static load coefficients of

the new bridge; 2) oscillations due to vortex shedding on both structures; 3) the flutter stability of the two structures; 4) the response to wind buffeting of both structures.

For each case the influence of the other structure being up-wind and down-wind, respectively, was included.

## 4 SECTION MODELS AND TEST ARRANGEMENT

The section models were constructed at geometric scale of 1:75 and had a length of 1.30 m. The tests were performed in DMI's 2.6 m wide Boundary-Layer Wind-Tunnel. The test rig for the static tests was equipped with a double 3-component force balance. For the dynamic tests, the section model investigated was spring suspended in a dynamic rig and the other model was mounted statically up-wind or down-wind. For the dynamic tests the velocity scale was about 1:5. A photograph of the test set-up is shown in figure 5.

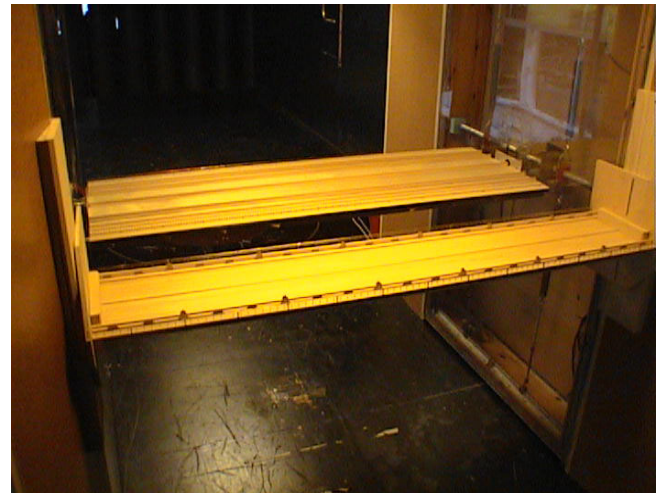


Figure 5: Section models in DMI's 2.6 m wide Boundary layer Wind Tunnel.

## 5 STATIC LOAD COEFFICIENTS

The static load coefficients of the new bridge deck were determined for two configurations: for the new bridge deck alone and for the new bridge deck with the deck of the existing bridge up-wind. The results are shown in figure 6.

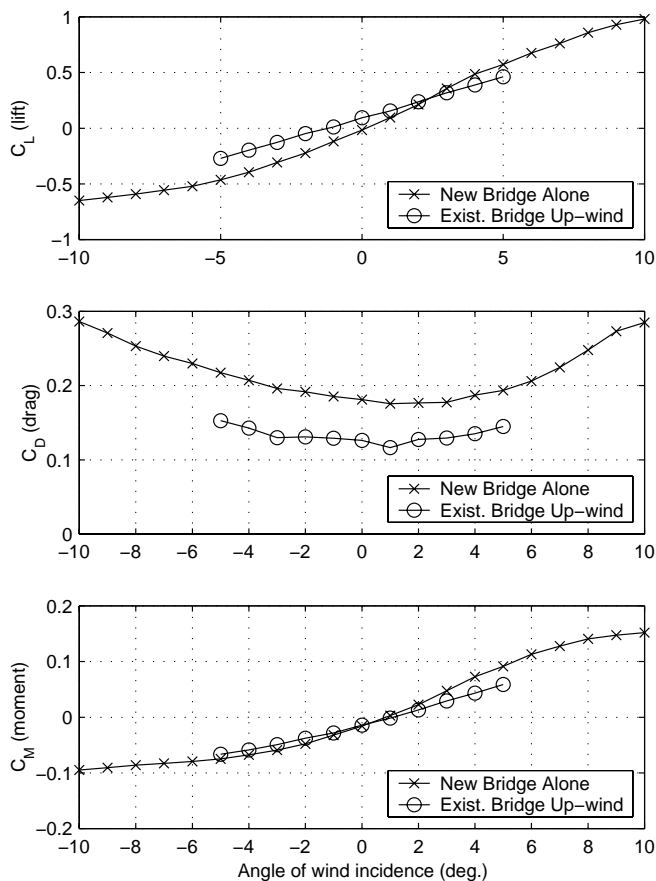


Figure 6: Static load coefficients (based on deck with  $B=33.5\text{m}$ )

The test results underlined that it is necessary to include the up-wind structures when placed in the vicinity of the bridge in question to obtain representative static properties.

The drag coefficients were dramatically reduced when the new bridge was shielded by the existing bridge, and also the rate of change of the moment and lift coefficients with angle of attack were reduced by the presence of the up-wind structure. These effects are generally beneficial.

## 6 VORTEX SHEDDING

As for the static coefficients it was found that the outcome of the tests depended strongly on whether the bridge was placed up-wind or down-wind of the neighbouring structure. In one case the up-wind structure removed all motion induced by vortex shedding (figure 7), in another case the up-wind structure seemed to induce (minor) torsional motion (figure 8). Remarkably, the torsional oscillation observed for both bridges occurred at the same reduced velocity defining a constant Strouhal number and coincidentally the oscillations had the same rms response (0.3 deg.).

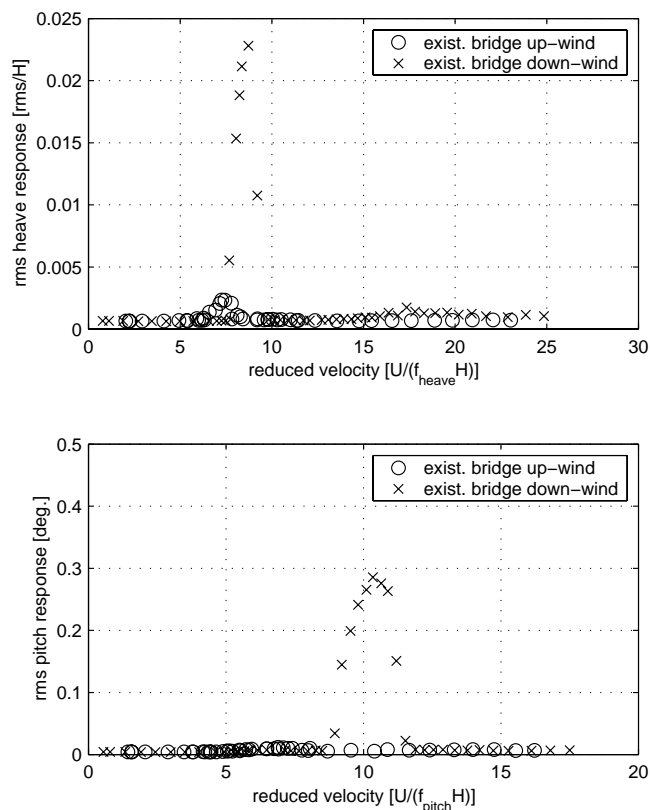


Figure 7: Vortex shedding induced response for the new bridge ( $H=3.6\text{ m}$ )

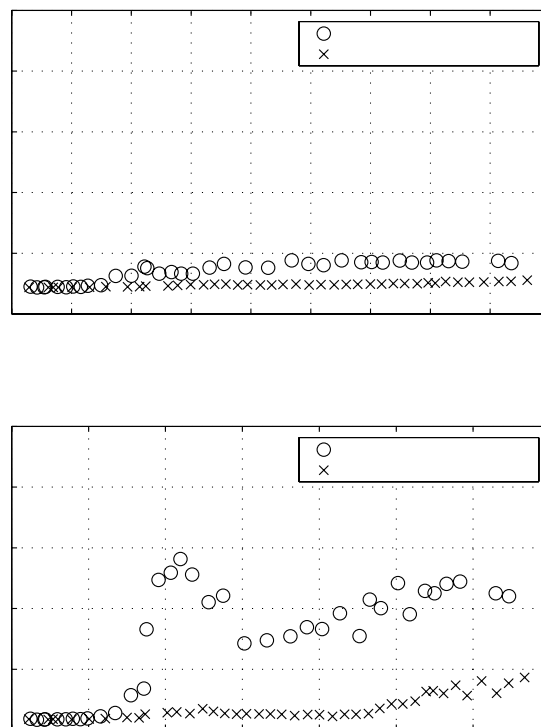


Figure 8: Vortex shedding induced response for existing bridge ( $H=2.8\text{ m}$ )

## 7. AERODYNAMIC STABILITY

For the modern bridge designers it is seldom a problem to design a structure that possesses a critical wind speed with respect to coupled flutter or

torsional instability that is sufficiently high. For the present case it is little more complicated.

Firstly, it must be demonstrated that the new bridge is not influenced in such a manner by the existing bridge that the critical wind speed is lower than required (and dramatically lower than would be expected for such a short-span structure). Secondly, and perhaps more importantly, the extent to which the new structure changes the flutter characteristics of the existing structure must be investigated.

For the present case none of the configurations tested showed unfavourable influence on the aerodynamic stability limit from one structure to the neighbouring structure.

## 7 BUFFETING RESPONSE

The response due to the buffeting action of the oncoming turbulent flow was measured directly on the section models. The test results were converted to full-scale equivalent static loads and displacements by an advanced spectral approach proposed by Davenport and King, 1984 in which the measured wind loading was corrected for mode-shapes, wind properties, aerodynamic admittance, joint acceptance function and structural damping to reflect the differences between the test conditions and the predicted full-scale conditions. The aerodynamic admittance and joint acceptance function corrections were based on the empirical model presented in Larose, 1997.

This method was chosen for its ability to use the maximum of the testing conditions, in occurrence the buffeting effect of the wake of the upstream bridge on the downwind bridge.

rately and introduced in a frequency domain model the wake buffeting information would have been lost unless a time consuming mapping and spectral characterisation of the wake had been done. A summary of the results of the buffeting analysis is given in Table 2.

## 8 APPLICATION OF RESULTS

As the span of the bridges is only moderate, the designers were only worried by possible aerodynamic interferences between the two bridges. Then vortex shedding and buffeting were the phenomena to be investigated more thoroughly. Nevertheless the static results are interesting since the drag coefficient is found to be significantly larger than the design value, which had been obtained from well-known standards (BS5400 and ENV 1991-2.4). Since transverse seismic forces are much larger than wind forces, the increased drag coefficient has no consequences on the design.

Vortex shedding accelerations in the model have been transformed into prototype accelerations after taking into account not only the different scale but also the increased damping coefficient (0.5%). Such estimation yields a peak acceleration equal to  $0.31\text{m/s}^2$ , which is lower than the admissible value ( $0.40\text{m/s}^2$ ).

The experimental buffeting forces have been compared with the values, which were considered during the design and have been found to be significantly lower. The resulting sum of the static and the buffeting forces are reduced by 31% with respect to the design forces in spite of the already mentioned increase in the drag coefficient.

All these results were satisfactory from the design point of view and they confirmed the advantages of performing a wind tunnel test in terms of an increased knowledge of the bridge behaviour.

Grillaud G., A. Chauvin and J. Bietry: *Comportement Dynamique d'un Pont Haubans dans une Turbulence de Sillage*. 8<sup>th</sup> International Conference on Wind Engineering, London, Canada, July 1991.

Larose, G.L.: *The Dynamic Action of Gusty Winds on Long-Span Bridges*. Ph.D. Thesis, Technical University of Denmark, April 1997.

Davenport A.G. and J.P.C. King: *"Dynamic Wind Forces on Long Span Bridges"*. 12<sup>th</sup> IABSE Congress, Vancouver, Canada, September 1984.

	Existing bridge down-wind (kN/m)		Existing bridge up-wind (kN/m)	
<b>Lateral</b>				
Mean	5.3	$C_x=0.18$	3.7	$C_x=0.13$
Resonant 1	2.0		1.4	
<b>Vertical</b>				
Mean	-0.5	$C_z=-0.02$	2.7	$C_z=0.09$
Resonant 1	26.7	$C_z=6.3$	13.4	$C_z=4.1$
<b>Torsion</b>				
Mean	-15.4	$C_m=-0.02$	-13.7	$C_m=-0.01$
Resonant 1	192	$C_m=1.0$	139	$C_m=0.76$

Table 2: Equivalent static loads for the new bridge

With the conventional buffeting analysis where the aerodynamic properties are determined sepa-