DOI: http://dx.doi.org/10.12989/sem.2008.29.2.171

Modal analysis and ambient vibration measurements on Mila-Algeria cable stayed bridge

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(Received December 4, 2006, Accepted March 22, 2008)

Abstract. The seismic response analysis of an existing bridge needs a mathematical model that can be calibrated with measured dynamic characteristics. These characteristics are the periods and the associated mode shapes of vibration and the modal damping coefficients. This paper deals with the measurements and the interpretation of the results of ambient vibration tests done on a newly erected cable stayed bridge across the Oued Dib River at Mila city in Algeria. The signal analysis of ambient vibration records will permit to determine the dynamic characteristics of the bridge. On the other hand, a 3-D model of the bridge is developed in order to assess the frequencies and the associated modes of vibration. This information will be necessary in the planning of the test on the site (locations of the sensors, frequencies to be measured and the associated mode shapes of vibration). The frequencies predicted by the finite element model are compared with those measured during full-scale ambient vibration measurements of the bridge. In the same way, the modal damping coefficients obtained by the random decrement method are compared to those of similar bridges.

Keywords: ambient vibration; cable stayed bridge; dynamic characteristics; numerical model; random decrement method.

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1. Introduction

The previous earthquakes such as the Loma Prieta (USA, 1989), Kobe (Japan, 1995), Izmit (Turkey, 1999), Chi-chi (Taiwan, 1999) confirmed that the bridges could be very vulnerable structures under dynamic loading. The subject of dynamic response of cable-stayed bridges has received increasing attention in recent years as this type of bridge becomes ever more popular (Wilson and Liu 1991, Pridham and Wilson 2005). Cable-stayed bridges are complex and their dynamic behavior is relatively difficult to predict due to the vibration coupling of their dynamic characteristics.

This paper is a contribution to the elaboration of an appropriate numerical model of the cable-stayed bridge erected over Dib-River in Mila Prefecture (Algeria). As a first step, a three-dimension-numerical analysis was carried out. The preliminary results of the elaborated numerical model of the bridge were checked with the results of the ambient vibration tests.

The results of the modal analysis of this model, in terms of natural frequencies of the bridge and their corresponding mode shapes, help to focus on the actual frequency range during the vibration tests, narrow the frequency-band-search that would be recorded and helped us to find the adequate locations of the sensors during measurements.

The first contribution to the engineering of cable-stayed bridges in Algeria, was initiated in 2004 when an ambient vibration records was conducted on the Mila Bridge, a newly constructed cable stayed bridge crossing the Oued Dib River at Mila city in Algeria (Kibboua 2006).

The recorded frequencies from the ambient vibration tests were compared to those obtained from the 3D finite element numerical model. In the same way, the modal damping coefficients obtained from ambient vibration records were compared to previous results found by other researchers for similar bridges that have more or less the same total length, number of spans and shape of the deck.

2. Bridge description

The Mila Bridge was opened in January 2001. It is a two-lane cable-stayed bridge. Until now, it is the unique and the only one built in Algeria. It is classified as a strategic bridge and links the two main cities of Constantine and Jijel. It has a total length of 502 m and it is composed of three spans: the centre span has 280 m and two equal side spans of 111 m. The deck, made of prestressed concrete, is composed of prefabricated elements (voussoirs) with a 0.20 m upper slab stiffened by three webs of 0.22 m thick. The deck has a total width of 13.30 m and a total height of 2 m

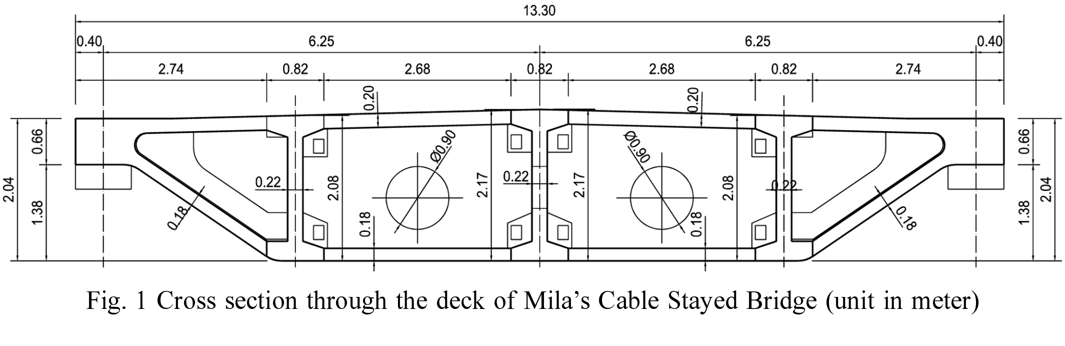

Fig. 1 Cross section through the deck of Mila’s Cable Stayed Bridge (unit in meter)

Fig. 2 General view of Mila's Cable Stayed Bridge

(Fig. 1). The bridge consists of two H-shaped concrete towers, double-plane fan type cables. It is composed of piers and towers, respectively, below and above the deck as illustrated in Fig. 2.

The bridge is suspended laterally with a semi-fan shape. 88 cables compose the suspension system, where 44 cables sustain the central span while 22 other cables sustain each edge span. The cables, composed of steel bars of 7 mm diameter, have different sections varying from 22.5 to 55.5 cm^2.

3. Structural model

The complex geometric shape of the bridge structure is modeled and analyzed using the finite elements program SAP 2000 (1997) and its incorporated bridge library. The deck was modeled by shell elements; the pylons (piers and towers) were modeled by frame elements and the cables were modeled as linear elastic truss elements (Gimsing 1983), having a stiffness that depends only on the modulus of elasticity of the cable E_s, the cross sectional A_c, and the length of the cable L_c.

The piers were assumed to be fixed at their bases, neglecting any interaction with the surrounding soil-layers or with water in case when the dam will be filled. The connection between the deck and piers was considered rigid with a fixed link.

The following mechanical characteristics were considered for the analysis in the Table 1.

Table 1 Mechanical characteristics of the bridge

Mechanical characteristics	Deck	Cables	Pylons (towers and piers)
Young's modulus (kN/m)	36×10^6	190×10^6	36×10^6
Poisson's ratio	0.2	0.3	0.2
Weight density (kN/m^3)	25	80	25

4. Modal analysis

Engineering investigations of both the linear and non-linear behavior of cable stayed-bridges (predominately geometric non-linearity due to cable sag) have been presented by Fleming and Egeseli (1980) and Nazmy and Abdel-Ghaffar (1990). Results of their investigations indicated that linear and non-linear dynamic analyses yield to similar results and that linear dynamic analysis is generally acceptable. In our case, a linear analysis of the model shown in Fig. 3 was carried out.

4.1 Modal frequencies

The natural frequencies obtained from the 3D finite element analysis are summarized in Table 2. It can be observed that the predominant mode is vertical. We should also note that the frequencies of the first modes are close to each other, that indicates the existence of coupling modes.

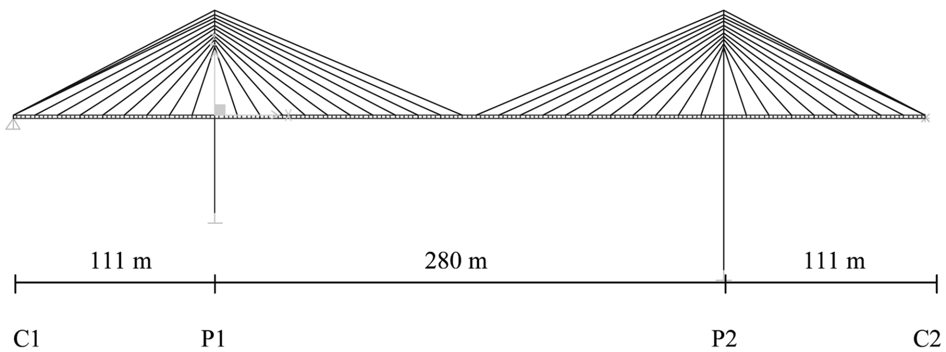

Fig. 3 Finite element model view of the bridge

Table 2 Frequency of each modes

Mode	Frequency (Hz)	Direction
1	0.34	
2	0.38	Lateral
3	0.44	Vertical
4	0.56	Lateral
5	0.58	Lateral
6	0.67	Vertical
7	0.77	Vertical
8	0.86	Vertical
9	0.92	Longitudinal
10	0.97	Lateral
11	1.08	Longitudinal
12	1.20	Vertical

Fig. 4 First vertical bending mode at 0.34 Hz

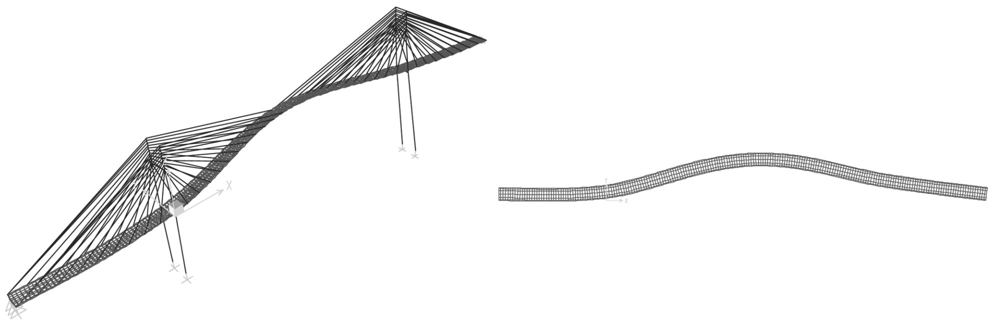

Fig. 5 First lateral mode at 0.38 Hz

4.2 Vibration mode shapes

Fig. 4 shows the fundamental mode of the bridge at a frequency of 0.34 Hz (period of 2.94 sec) which is a vertical motion of the deck. Fig. 5 shows the second computed mode at 0.38 Hz (period of 2.63 sec) having lateral motion of the deck.

5. Experimental study

As a second part of the investigation, the response of the bridge to the ambient vibrations excitations is considered. This method was adopted due to many reasons: 1) simplicity in the procedure, 2) relatively fast, 3) low-cost consuming and 4) no disturbance of the traffic when carrying out the tests. Consequently, the measurements at one location or at different locations on the tested structure can be easily repeated many times.

The procedure consists in the installation of sensors at different locations on the bridge where the response is recorded through a mobile measurement station (Fig. 6). The used station and sensors are very sensitive. They are able to record the vibration response and allow the determination of the dynamic characteristics of the bridge in the small deformation domain (Farsi and Bard 1998). Actually, the structural response is linear-elastic and the very small amplitudes of the vibrations are in the range of 10^{-6} to 10^{-4} g in term of acceleration.

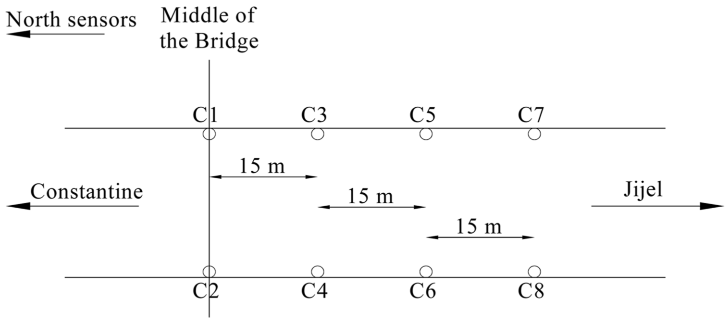

Fig. 6 Ambient vibration measurements -location of sensors on Jijel side-

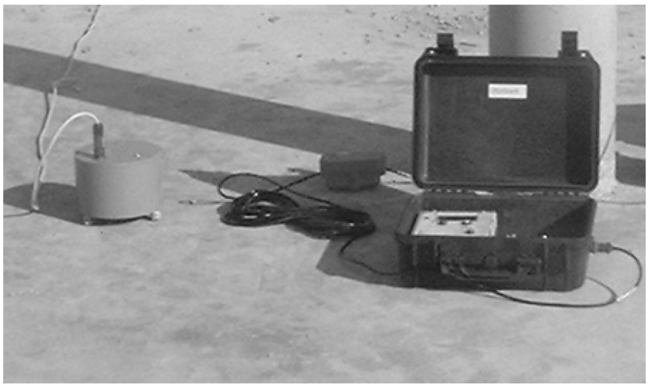

Fig. 7 Equipment for ambient vibration measurements

5.1 Measurement station

The equipment used during the ambient vibration tests is composed of seismometers with three-components (vertical, NS and EW) and CityShark measurement stations as shown in Fig. 7.

- **Seismometers**

The used sensors can record velocities and are of Lennartz 5 second type with three components (Le3D-5s). They can record frequencies of motions due to wind, traffic, etc. They are mainly used in the civil infrastructures and particularly for soil records.

- **CityShark station**

There are two types of measurement station CityShark. The station CityShark I can be connected to only one sensor with three components. The station CityShark II can be connected to 6 sensors with three components each. Different way and recording modes can be achieved, manually or automatically (synchronic measurement: start to record at the same time). The sampling frequency may vary from 10 to 1000 Hz, according to the selected rate. The recorded data are saved in flash cards (Chatelain *et al.* 2000).

5.2 Spectra records

The ambient vibrations are random and the analysis of the response of the bridge to such actions consists of the computation of Fourier Spectra of different windows taken from the response signal, compute their mean and evaluate the standard deviation. However, to do so and to be able to analyze the dynamic response, the selected windows, from the total recorded signal, must approach as much as possible the characteristic of a white noise record. Consequently, not all windows on the record can be used. As an example, Fig. 8 shows the windows, colored backgrounds, which were selected for treatment.

The spectral amplitude for each window is computed through a Fourier transform. Then, all computed spectra are smoothed through a sliding window, which the form and the width depends on the frequency (Konno and Ohmachi 1998). Finally, the obtained spectra are averaged and their standard deviation determined (Figs. 9 and 10).

The Fourier transform of the recorded signals enable the assessment of the natural frequencies of the bridge and are extracted simply by locating the peaks corresponding to the maximum responses. The mode frequencies of the tested bridge obtained through the records at different location points are illustrated in Figs. 11(a), 11(b) and 11(c).

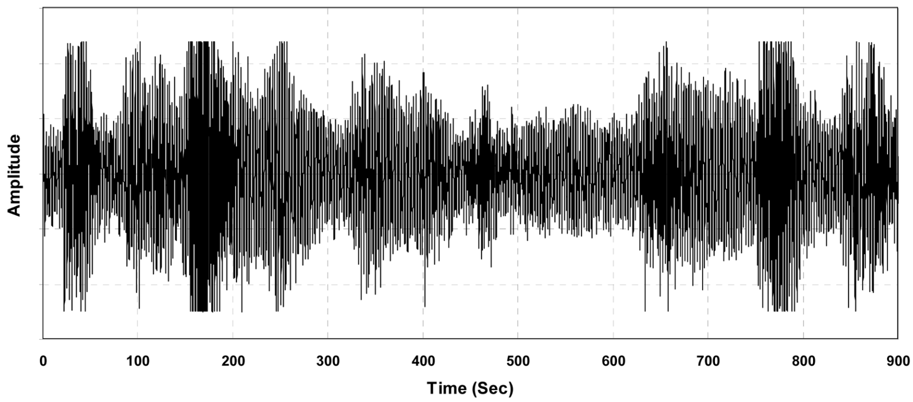

Fig. 8 Recording of ambient vibration noise on the bridge in the vertical direction at location C1

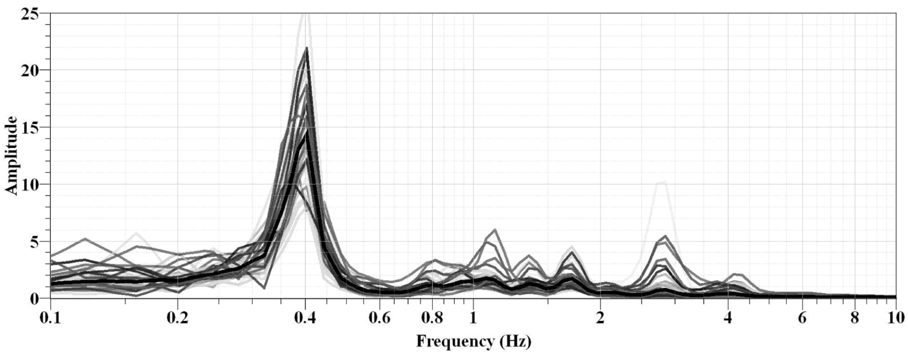

Fig. 9 Spectra on the various windows of the recording of ambient vibration noise on the bridge in the vertical direction at location C1

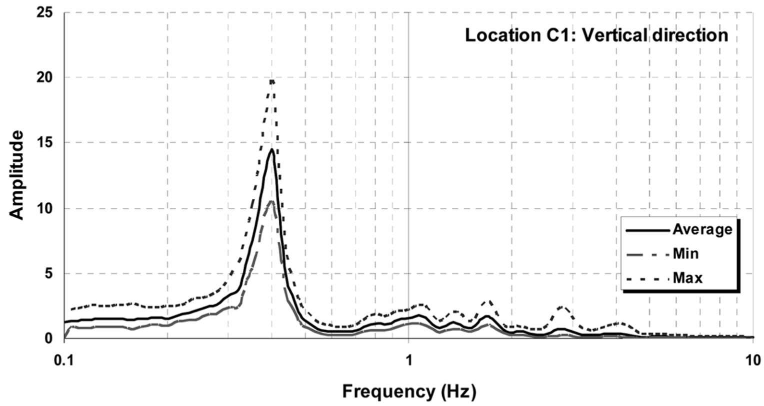

Fig. 10 Spectra of the recording of ambient vibration noise on the bridge in the vertical direction with the standard deviation at location C1. The continuous feature is the average of the spectra calculated on the various windows and the discontinuous features represent the standard deviation

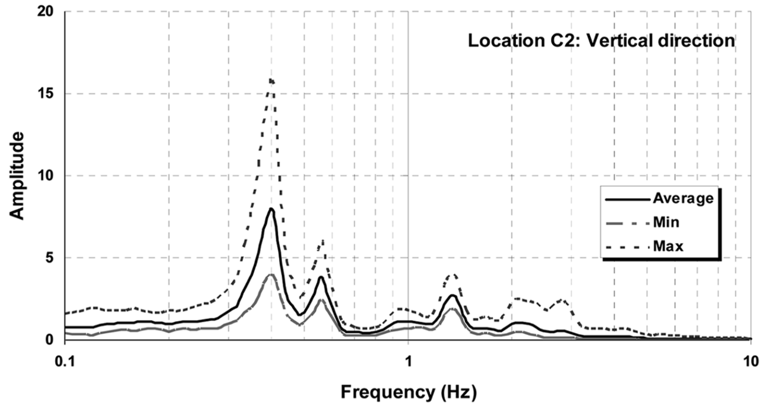

Fig. 11(a) Frequencies of the bridge in the vertical direction with the standard deviation at location C2

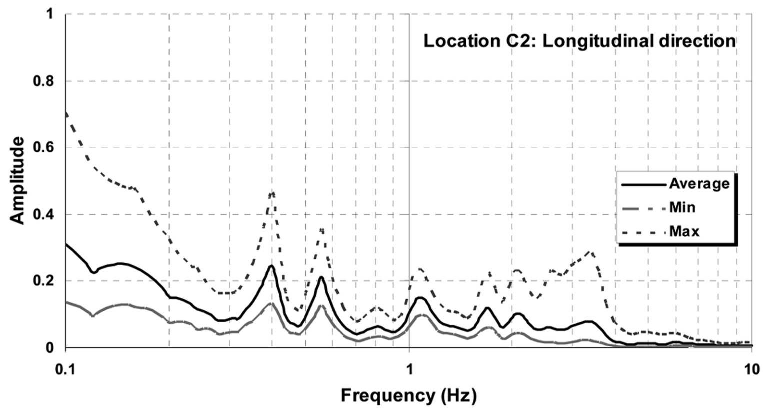

Fig. 11(b) Frequencies of the bridge in the longitudinal direction with the standard deviation at location C2

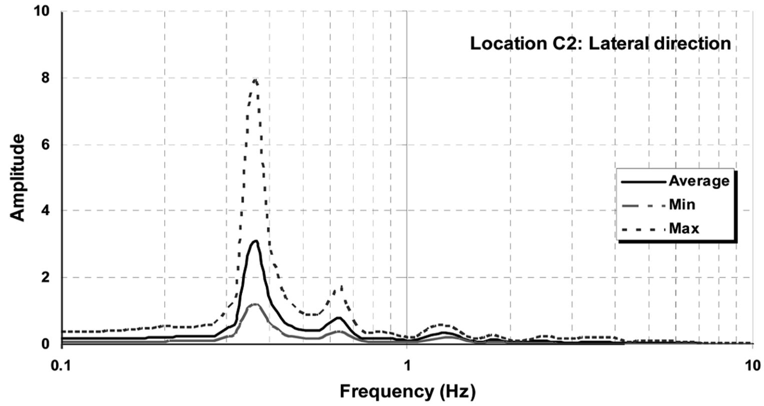

Fig. 11(c) Frequencies of the bridge in the lateral direction with the standard deviation at location C2

Table 3 Identified experimental modal frequencies of the bridge

Mode	Frequencies (Hz)	Direction
1	0.36	Vertical (Z)
2	0.38	Longitudinal (NS)
3	0.39	Lateral (EW)
4	0.56	Lateral (EW)
5	0.55	Longitudinal (NS)
6	0.63	Vertical (Z)

5.3 Experimental modal frequencies

A total of 6 vibration modes of Mila's Bridge were identified from the records in the range of 0 to 1 Hz. The first natural mode is a vertical mode with a frequency of 0.36 Hz. The first lateral mode was found to be occurring with a frequency of 0.39 Hz, while the first longitudinal mode appeared at a frequency of 0.38 Hz. Cable-stayed bridges are complex and their dynamic behavior is relatively difficult to predict due to the vibration coupling of their dynamic characteristics, which may explain the closed frequencies of some modes.

The main frequencies recorded on the bridge are illustrated in Table 3.

5.4 Comparison between analytical and experimental frequencies

Table 4 shows a comparison beteen the analytical, f_{cal}, and the measured, f_{meas}, frequencies of the bridge.

The frequencies obtained from the analytical model and those from the tests in the vertical and lateral directions are relatively comparable for the first two modes. However, in the longitudinal direction, the finite element model in the analysis seems very stiff. The adopted model could be improved by considering the realistic nature of these boundary conditions as well as the permissible degrees of freedom of the deck-tower bearings.

Table 4 Comparison of the analytical and test results

Direction	Mode	f_{meas} (Hz)	f_{cal} (Hz)	f_{cal}/f_{meas}
Vertical	1	0.36	0.34	0.94
	2	0.63	0.44	0.70
Lateral	1	0.39	0.38	0.97
	2	0.55	0.56	1.02
Longitudinal	1	0.38	0.92	2.42
	2	0.56	1.08	1.93

6. Estimation of the modal damping

Damping is a characteristic that quantifies the energy dissipated by a loaded structure. The quantification of damping is complex because internal frictions within and/or at connections are difficult to model. On the other hand, the experimental methods such as the ambient vibration test make it possible to evaluate the damping coefficient by analyzing the recorded response of the structure to a dynamic input. The random decrement method was adopted to estimate the modal damping coefficients of the bridge. A brief introduction of this method is introduced hereafter.

6.1 Determination of the damping coefficient by the random decrement method

The random decrement method (Dunand *et al.* 2002) is based on the fact that the response of a dynamic system (e.g., a simple oscillator) to a random vibration can be broken up into two parts. The first part corresponds to the impulse response of the system and the second part to its forced response. The idea is to remove the random component to reveal the impulse response by doing the summation of a great number of windows of the signal (Fig. 12) which all have the same initial

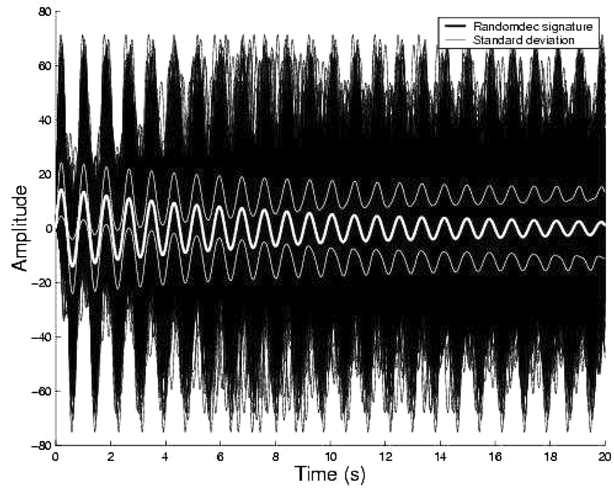

Fig. 12 Average (large white line) and standard deviation (thin white line) of 2440 windows (black lines) obtained with the Random decrement method on the Millikan Library in the lateral direction (Dunand *et al.* 2002)

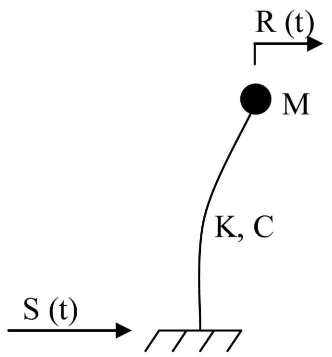

Fig. 13 Simple Oscillator

conditions (e.g., null displacement and positive velocity). The random part of the response becomes weak in front of the impulse response which has a zero average at the end of the signal. Then, starting from the impulse response, it is possible to deduce the frequency f_0 and the damping coefficient ξ by using the logarithmic decrement method (Clough and Penzien 2003).

6.2 Theoretical approach

The method of the random decrement can be explained by the study of the response of a Single Degree Of Freedom, SDOF, oscillator to a random excitation. The simple oscillator with a mass M, a rigidity K, and a damping C, is subjected to a random excitation with imposed displacement $S(t)$ (Fig. 13). Its response in terms of relative displacement $R(t)$ can be deduced from the differential Eq. (1), which can be rewritten in the reduced form given by Eq. (2).

$$M.\ddot{R}(t) + C.\dot{R}(t) + K.R(t) = -M.\ddot{S}(t) \tag{1}$$

$$\ddot{R}(t) + 2\xi\omega_0.\dot{R}(t) + \omega_0^2.\dot{R}(t) = -\ddot{S}(t) \tag{2}$$

By assuming that the oscillator's behavior is linear-elastic, it is possible to apply the principle of superposition. The ambient vibration noise enters within the framework of this assumption from its low level of excitation. In such case, it is possible to break up the excitation into several parts and make summation of the responses to obtain the resultant response. By separating the excitation $S(t)$ into two parts, respectively, $S_1(t)$ and $S_2(t)$ before and after a time t_0, the response, as illustrated in Fig. 14, would be $R_1(t)$ for $S_1(t)$ and $R_2(t)$ for $S_2(t)$:

$$S_1(t) = S(t) \text{ for } t < t_0 \text{ and } S_1(t) = 0 \text{ for } t > t_0$$
$$S_2(t) = 0 \text{ for } t < t_0 \text{ and } S_2(t) = S(t) \text{ for } t > t_0$$

The interested responses of the oscillator after the time t_0, R_1 $(t > t_0)$ and R_2 $(t > t_0)$ would take the following forms:

- The response to the imposed displacement $S_1(t)$:

After the time t_0, the response $R_1(t)$ is a free oscillation of the form of Eq. (3) for an oscillator with one degree of freedom. $R_1(t)$ depends only on the displacement $R_1(t_0)$ and the velocity $\dot{R}_1(t_0)$

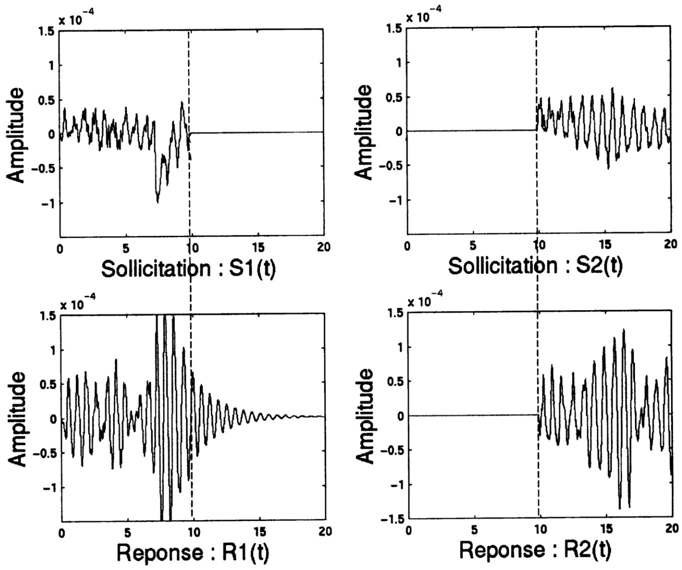

Fig. 14 Separation of the solicitation and the response in two parts

of the oscillator at the moment t_0.

$$R_1(t) = R_1(t_0) + \dot{R}_1(t_0)\frac{1}{\omega_0\sqrt{1-\xi^2}}e^{-\xi\omega_0(t-t_0)}\sin[(t-t_0)\omega_0\sqrt{1-\xi^2}] \tag{3}$$

- The response to the imposed displacement $S_2(t)$:

After the time t_0, $R_2(t)$ is a forced mode which is governed by Eq. (4) for an oscillator with one degree of freedom.

$$R_2(t) = \frac{1}{\omega_0\sqrt{1-\xi^2}}\int_{t_0}^{t} S_2(t-\tau)e^{-\xi\omega_0\tau}\sin(\tau\omega_0\sqrt{1-\xi^2})d\tau \tag{4}$$

The time t_0 can be selected with various ways, Huerta *et al.* (1998) proposed to take t_0 when the response in displacement passes by a threshold value with a positive velocity. Whereas Delome *et al.* (1990) suggested to choose t_0 when the displacement is null and the velocity is positive, thus imposing a null threshold on displacement on the preceding condition. The important parameter is to obtain windows of response with identical initial conditions. In our analysis, Delome *et al.* suggestions were selected. Finally, the summation of all the windows of the response $R(t)$ in respect to the mentioned initial conditions reveals two behaviors according to whether the responses are of the type $R_1(t)$ or of the type $R_2(t)$.

The free oscillations $R_1(t)$ will be added in a constructive way according to Eq. (5).

$$\sum_{t_0} R_1(t) = \sum_{t_0}\left\{R_1(t_0) + \dot{R}_1(t_0)\frac{1}{\omega_0\sqrt{1-\xi^2}}e^{-\xi\omega_0(t-t_0)}\sin[(t-t_0)\omega_0\sqrt{1-\xi^2}]\right\} \tag{5}$$

The sum will be always of the form of a free oscillation. Indeed, all responses are different only by their initial conditions $R_1(t_0)$ and $\dot{R}_1(t_0)$, the displacements and velocities of the oscillator at the time t_0. In this case $R_1(t_0)$ is chosen null, therefore the answers $R_1(t)$ depend only on $\dot{R}_1(t_0)$.

The answers to the random forced mode will be added in a destructive way according to Eq. (6).

$$\sum_{t_0} R_2(t) = \frac{1}{\omega_0\sqrt{1-\xi^2}} \int_{t_0}^{t} \sum_{t_0} [S_2(t-\tau) e^{-\xi\omega_0\tau} \sin(\tau\omega_0\sqrt{1-\xi^2})] d\tau \tag{6}$$

By increasing the number of windows, the random responses can be disappear because their average tends towards zero, and make appear the free oscillation of the oscillator (e.g., Fig. 12). This procedure allows the evaluation of the impulse response of the oscillator, and estimates its frequency and damping coefficient. As the impulse response is of the form of Eq. (3), a pseudo pulsation ω_1 can be extracted, which is equal to $\omega_0\sqrt{1-\xi^2}$.

In practice the damping coefficient ξ_i is low (<10%). This fact allows merging the narrow frequency band by confusion of ω_1 and ω_0. By doing so, it is then possible to determine the period of the free oscillation by an average of the observed periods $T_0 = 2\pi/\omega_0$. The determination of the damping coefficient is done in the following way:

According to Eqs. (3) and (7), between two specified times (t) and ($t + mT_1$) spaced by a number "m" times of pseudo periods $T_1 = 2\pi/\omega_1$, the amplitude $R_1(t)$ and R_1 ($t + mT_1$) of the free oscillation decrease of a factor $e^{\xi\omega_0 mT}$. By taking the logarithm of the ratio of the respective amplitudes, the Eq. (8) shows that it is then possible to determine the value of the damping coefficient ξ (for low values of ξ).

$$R_1(t+mT_1) = \dot{R}_1(t_0)\frac{1}{\omega_0\sqrt{1-\xi^2}} e^{-\xi\omega_0(t-t_0+mT_1)} \sin[(t-t_0+mT_1)\omega_0\sqrt{1-\xi^2}] \tag{7}$$

$$\ln\left(\frac{R_1(t)}{R_1(t+mT_1)}\right) = \xi\omega_0 mT_1 = \xi\omega_0 m\frac{2\pi}{\omega_1} = \frac{\xi 2\pi m}{\sqrt{1-\xi^2}} \approx \xi 2\pi m \tag{8}$$

In order to obtain an amplitude ratio, which represents of all the impulse response, the slope p of the logarithm of the extremes of the impulse response obtained by linear regression is considered. Then the damping coefficient is determined by the Eq. (9).

$$\xi = \frac{-pT}{2\pi} \tag{9}$$

Where $T = 2\pi/\omega_0$ is the period of a cycle and p the slope of the regression.

It is much more difficult to obtain meaningful estimation of the damping from ambient vibration data than to determine modal frequencies. This occurs because the small valued damping ratios are very sensitive to the non-stationary nature of the ambient vibrations, the selections of the frequency

Table 5 Modal frequencies and modal damping of the bridge

Mode	Damped experimental Frequencies (Hz)	Modal damping (%)	Direction
1	0.37	0.40	Vertical
2	0.39	0.24	Lateral
3	0.56	0.34	Lateral
4	0.64	1.25	Vertical

resolution, the averaging time during the process of data analysis and the physical measurements of the bandwidth of the spectral peaks.

Using the random decrement method, the values of the modal damping coefficients and the frequencies of vibrations of the bridge for an ambient vibration noise recording of duration of 15 minutes are given in Table 5.

Hereafter, some values of the damping coefficients obtained on the Mila bridge are presented and compared with Tampico, Figueira da Foz and Quincy Bayview bridges (Muri-Vila *et al.* 1991, Rodrigues and Campos-Costa 1998, Pridham and Wilson 2005), as shown in Table 6. These bridges were selected based on the geometric similarity, such as shape of the deck and the total length of the bridge. As illustrated in Fig. 15, it can be observed that the damping of the first and the second modes are nearly identical for Mila and Tampico bridges. However, a large gap was observed between the Figueira da Foz, Quincy Bayview and Mila bridges. Figueira and Quincy bridges showed a higher damping for the tree first modes compared to Tampico and Mila bridges. For the first and second modes, nearly the same frequencies were found for Tampico, Quincy and Mila bridges.

Since the four bridges were not exactly similar, a perfect comparison can not be made based only on the few available data on the three bridges.

For the four bridges, the measured damping coefficients are small compared to the values used in calculations for concrete structures and stipulated in many seismic codes to be taken as 5%.

Table 6 Comparison of modal frequencies and modal damping of the Mila Bridge with other bridges

N	Name	Total length (m)	Modal Frequencies and modal damping coefficients							
			f_1 (Hz)	ξ_1 (%)	f_2 (Hz)	ξ_2 (%)	f_3 (Hz)	ξ_3 (%)	f_4 (Hz)	ξ_4 (%)
1	Mila Bridge	502	0.37	0.40	0.39	0.24	0.56	0.34	0.64	1.25
2	Tampico Bridge	640	0.40	0.39	0.45	0.20	0.90	-	-	-
3	Figueira da Foz Bridge	405	0.51	1.60	0.60	1.30	0.73	1.60	0.87	1.00
4	Quincy Bayview Bridge	542	0.37	1.40	0.50	-	0.56	1.10	-	-

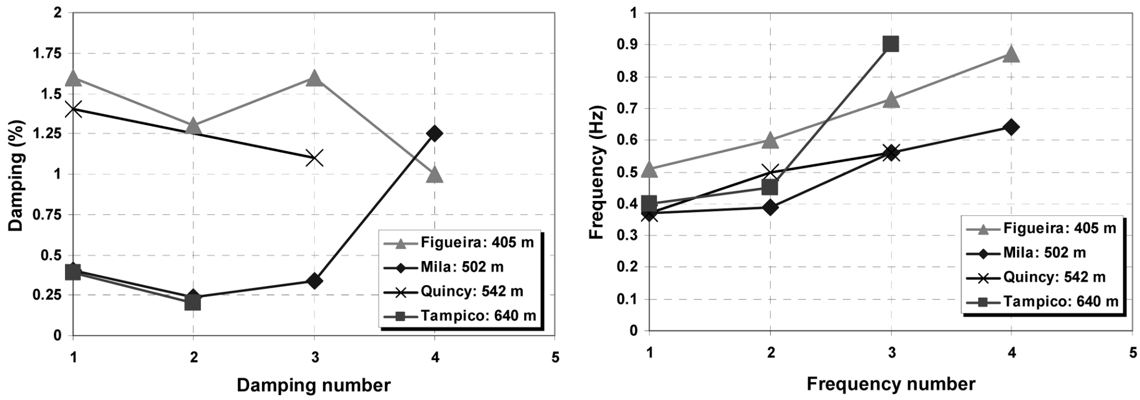

Fig. 15 Variation of the experimental modal damping ratios (left) and frequencies (right)

7. Conclusions

Cable-stayed bridges are complex and their dynamic behavior is relatively difficult to predict due to the vibration coupling of their dynamic characteristics. To evaluate the dynamic response of Mila's (Algeria) bridge a numerical model using finite elements method was suggested, based on the collected details and plans.

A series of ambient vibration tests were carried out on the bridge. The achieved operations were not simple and easy while conducting the tests. This fact was due to the topography at the location of the bridge and the difficulties to reach some places inside and outside the bridge as well as the problem faced when using the equipment during the tests. Also, in order to not disturb the traffic, many hours were necessary to complete the necessary records.

The processing of the recorded data was carried out using specific software in order to extract the dynamic characteristics of the bridge. A good agreement was obtained for the computed and measured vertical and lateral frequencies. However, scattered values, between the computed and the measured frequencies, were obtained for the longitudinal direction. The reason maybe attributed to the real nature of the boundary conditions as well as the permissible degrees of freedom of the deck-tower bearings.

Comparison in term of frequencies and damping ratios with Tampico and Figueira da Foz bridges was presented. It was found that, damping of the first and the second modes were nearly identical for Mila and Tampico bridges. However, a large gap was observed while compared to Figueira da Foz bridge. Since the four bridges were not exactly similar, a perfect comparison can not be made based only on the few available data on the three bridges.

For the four bridges, the measured damping coefficients are small compared to the values used in calculations for concrete structures and stipulated in many seismic codes to be taken as 5%.

Acknowledgements

The authors would like to express their thanks to Ali Nour of Ecole Polytechnique de Montreal of Canada for his assistance in the early stages of the study. Thanks are also extended to Farid Bouriche, Hamid Rezkallah and Rabah Bensalem of CGS for their valuable assistance in conducting the field tests.

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