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VIBRATIONS TESTS OF A BAILEY-TYPE SUSPENSION BRIDGE OVER THE JABLANICA LAKE

Amina Karavelić-Hadžimejlić, Senad Medić
University of Sarajevo, Faculty of Civil Engineering, Bosnia and Herzegovina

Abstract

This paper presents results of vibration tests conducted on a recently reconstructed suspension bridge in Ostrožac, Bosnia and Herzegovina. The Ostrožac bridge spans 200 m over the Jablanica Lake and represents a case of exceptional engineering. However, it is also a typical example of ageing infrastructure characterized by a lack of adequate maintenance and severe degradation by corrosion. The main girders are a complex steel portable Bailey-type trusses suspended by Ø30 mm steel hangers and 6Ø52 mm primary load-bearing cables. The curved cables support the road pass over 18 m high reinforced concrete pylons. Six 44 m long straight inclined back-stay cables transfer the tensile force using threaded rods to the reinforced concrete block anchorages at both ends. The dynamic parameters were obtained by on-site measurements using Digitex xWave triaxial accelerometers and the output-only operational modal analysis which does not require knowledge of the input excitation. The vibrations were induced by wind and by the passage of cars. The structure vibrates spatially even though the nature of excitation is dominantly vertical and the adjacent structural segments of the Bailey truss oscillate almost independently. Power spectrum, damping and Scruton number were determined from acceleration and displacement measurements of the inclined cables. Natural frequency was employed for estimation of the cable axial force. Even though the inclined cables have solid dynamic parameters, notable wind-induced vibrations were visually detected implying the need for structural strengthening intervention.

Keywords: suspension bridge, Bailey truss, vibration testing

1 Introduction

The suspension bridge over the Jablanica Lake, spanning 200 meters and with a width of 4 meters, was constructed by Hidrogradnja Sarajevo in 1952 for one-way traffic with limited load capacity and duration (Fig. 1). Like any temporary solution, the bridge has endured and served its purpose for 70 years [1]. The bridge is located on regional road R437 at km 51+350 in the settlement of Ostrožac. The investor does not possess the project documentation for the bridge, and based on available information, the structural design was carried out by the Military Technical Institute - Belgrade. The bridge was calculated according to the relevant regulations at that time for applying PTP 5.

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Figure 1 View of the suspension bridge in Ostrožac

The end sections at the lake shores, each 25 m long, are reinforced concrete structures with three arched 5 m openings for the passage of water. Reinforced concrete 18 m high pylons support the main curved cables of the central span and they are anchored using inclined back-stay cables. The pylons with form an A longitudinally and a portal transversely. Main 80/80 cm columns are braced with transverse beams of 40/80 cm. The central bridge span is formed with 65 double, single-height, strengthened Bailey-type steel prefabricated trusses. They are suspended on a 6Ø52 mm main cable (closed spiral type and galvanized) using Ø30 mm Č0361 steel hangers at each 6.1 m [2].

2 Back-stay cables

The inclined stays, approximately 44 m long (from the foundation to the top of the pylon), consist of 6 cables with a diameter of approximately 50 mm (Fig. 2).

Figure 2 Back-stay cables instrumented with LVDT and an accelerometer

The quality and mechanical characteristics of the material from which the cables are made are unknown. It is presumed that they are formed with closed spiral strands with several layers of wires of different shapes. On the outer side, there are likely wires of ‘z’ cross-section providing good mechanical protection against corrosion. The dynamic characteristics of the inclined stays were tested on two cables (second and sixth) on the south side of the lake shore towards Jablanica. Based on the measured accelerations and displacements of the cables, the spectrum of vibration power (natural frequencies), damping, and Scruton number were determined. Additionally, based on the natural frequency, the normal force in the cables was estimated.

The following measuring equipment was employed for testing: HBM WA 100 displacement transducers, HBM B12 accelerometers, Spider 8 amplifier and Catman 5 software data acquisition. Based on occasional observations of cable behaviour while on the bridge, it was noticed that the cables on the south shore exhibit more pronounced vibrations with larger amplitudes compared to the remaining stays. Measurements of acceleration and displacement were performed on two cables, as shown in the following diagrams (Fig. 3). The excitation was caused by hammer impact, and the first frequency was equal to 2.39 Hz (cable 2) and 2.44 Hz (cable 6). Accelerometers and displacement transducers were placed approximately 2 meters above the ground where the impact was induced, which may result in a slightly stiffer response of the structure. It would be desirable to place measuring instruments at the midspan.

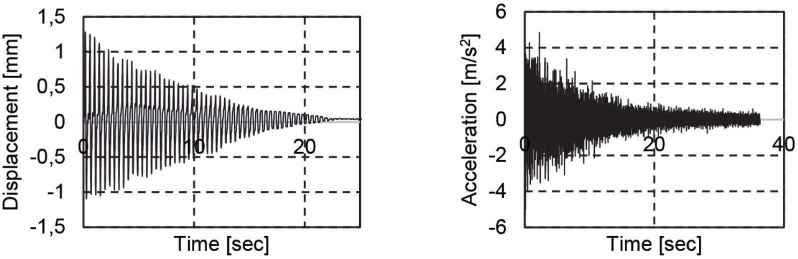

Figure 3 Variation of cable displacement (left) and acceleration (right) of stay no.2 in time due to hammer impact

Power spectra of displacement and acceleration time histories are shown in Fig. 4. The measured frequencies match exactly. The tension force in the cable can be assessed using equation (1) according to [3]. This equation is valid if dealing with a long inextensible cable without bending stiffness. For m = 7850 · 0.0023 = 18.06 kg/m, L = 40 m and f = 2.39 Hz one obtains T = 659 kN (σ = 286 N/mm^2).

$$T = 4 \cdot m \cdot L^2 \cdot f^2 \tag{1}$$

SAP2000 was employed to numerically model the inclined cable [4].

The tensile force was introduced through nonlinear load case with negative temperature t^0 = -168.5 °C obtained from $F = EA\alpha t^0$. Then the modal analysis was executed which yielded dependable results compared experimental investigation (f_1 = 2.36 Hz, f_2 = 4.73 Hz, and f_3 = 7.11 Hz).

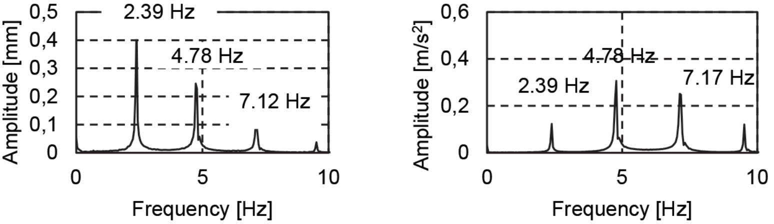

Figure 4 Power spectra of no.2 inclined stay vibrations determined from displacement (left) and acceleration (right) time histories

By comparing experimentally determined values of the damped vibration amplitudes (displacements or accelerations), the damping coefficient can be determined. The relationship between the displacement u_i at time t and the displacement u_{i+1} at time t+T_D does not depend on time and can be expressed using the logarithmic decrement δ given in (2) [5].

If the amplitude decay is slow, it is desirable to compare the values of amplitudes displaced over several cycles (instead of two consecutive ones). After j cycles of displacement decreasing from u_1 to u_{j+1}, the decrement is equal to the expression (3).

$$\delta = \ln \frac{u_i}{u_{i+1}} \approx 2\pi\xi \quad (\xi = c / c_{cr}) \tag{2}$$

$$\delta = \frac{1}{j} \ln \frac{u_1}{u_{j+1}} \approx 2\pi\xi \tag{3}$$

Displacement amplitudes of the inclined stay no. 2 are given in Table 1. Derived from the collected measurements, the period of damped vibrations amounts to T_D = 0.44 sec, and the relative damping ξ = 0.7 %.

Table 1 Displacement amplitudes of inclined stay no. 2

Peak	Time [sec]	Amplitude v_i [mm]
1	0.6	1.23
21	9.42	0.51

Problems with vibrations of inclined cable stays of bridges are well-known in literature [6]. An important parameter for analysis is the Scruton number S_c, given by equation (4). To avoid the occurrence of vibrations, it is necessary for the Scruton number to be greater than 10 according to PTI recommendation for cable stays [7].

$$S_c = \frac{m\xi}{\rho D^2} \tag{4}$$

In Eq. (4), m is the distributed mass of the stay, ξ = δ/2π stands for the damping coefficient, D equals the diameter and the air density is described by ρ. The Scruton number determined in this manner amounts to S_c = 35.9 › 10. Table 2. provides a comparison of the dynamic parameters of the bridge in Ostrožac and the bridge over Rijeka Dubrovačka (after rehabilitation and installation of dampers) [6]. It can be concluded that the bridge over Lake Jablanica has solid dynamic parameters despite noticeable oscillations of the inclined stays.

Table 2 Comparison of bridges over Lake Jablanica and Rijeka Dubrovačka

Parameter	**Lake Jablanica**	**Rijeka Dubrovačka**
f_1 [Hz]	2.44	0.647
ξ [/]	0.007	0.0121
S_c [/]	35.9	16.8

3 Main span

The use of experimental tests to gain knowledge about the dynamic response of civil structures is a well-established practice. Assuming that the dynamic behaviour of the structure can be expressed as a combination of modes, whose values depend on geometry, material properties, and boundary conditions, Experimental Modal Analysis (EMA) identifies those parameters from measurements of the applied force and the vibration response.

The identification of the modal parameters by EMA techniques is challenging in the case of civil engineering structures because of their large size and low frequency range. The application of controlled and measurable excitation is often a complex task that requires expensive and heavy devices. For this reason, the community of civil engineers has more recently focused the attention on the opportunities provided by Operational Modal Analysis (OMA). OMA can be defined as the modal testing procedure that allows the experimental estimation of the modal parameters of the structure from measurements of the vibration response only [8, 9]. It is a so-called Output Only method, where it is assumed that the wind, traffic, and human activities can adequately excite a structure. Highly sensitive acceleration sensors are used to record, evaluate, and interpret the vibration behaviour of a structure without forced excitation in all three directions in space. The main assumption of the Output-Only identification methods is that the ambient excitation input is a Gaussian white noise stochastic process in the frequency range of interest. This type of analysis is suitable for monitoring the structural condition due to vibrations induced by natural processes and for detecting damage within the structure [10, 11]. However, it should be emphasized that the method is limited to small displacements and the linear range of material behaviour.

DIGITEX Innovative Systems has developed a framework for structural health monitoring (SHM) using an intelligent and reliable monitoring system termed Sentry System [12]. The employed configuration consists of a central acquisition unit (xPlorer), five triaxial accelerometers (xWave) and two Wi-Fi units (xNet) (Fig. 5). Ambient Response Testing and Modal Identification Software (ARTeMIS) was used for data analysis and visualization [13]. ARTeMIS Modal includes the Frequency Domain Decomposition (FDD) peak picking method. FDD is a modal analysis technique that estimates the modes of a system from the calculated spectral densities for a lightly damped structure in a condition of white noise input.

The dynamic properties of the bridge were determined by measuring the response after excitation caused by wind and traffic loads. A 200 Hz sampling was conducted in 4 phases, with one of the 5 xWave devices used as a reference and placed in the middle of a pedestrian passage, while the remaining 4 devices changed position.

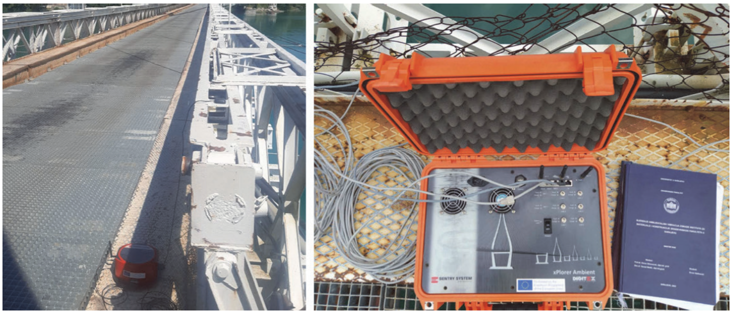

Figure 5 Bridge vibration testing: triaxial xWave accelerometer (left), and central acquisition unit xPlorer (right)

The singular values of spectral densities (natural frequencies) are shown in Fig. 6 using the Artemis Modal program. Considering that the bridge is highly deformable, the diagram below provides a more detailed representation of the low-frequency region. A short 60 s measuring interval was chosen due to time and bridge closure constraints. It was sufficient for a car to approach and cross the bridge, and for the bridge to vibrate after the excitation. Typical accelerogram induced by vertical excitation on the main span in X, Y, and Z directions is shown in Fig. 6 (inset).

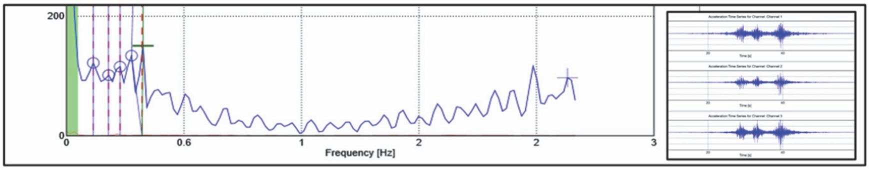

Figure 6 Power spectral density of the low-frequency region and accelerogram induced by vertical excitation on the main span (X, Y, Z) (inset)

It is noted that the structure predominantly vibrates vertically due to the nature of the excitation (passage of vehicles). However, the structure oscillates spatially in all three directions, i.e., longitudinal, and lateral vibrations occur simultaneously with the vertical ones (Fig. 7) for all modes. The obtained dynamic properties are shown in Table 3. In case of very low damping, the program assumes zero values. The third column refers to the complexity of the mode shapes. Modes can be complex due to non-proportional damping, poor estimation of shape parameters, mismatched data due to variable weather conditions. In such situations, the shape animation in the geometry window will show a “traveling wave” because the minimum and maximum values do not occur simultaneously [13].

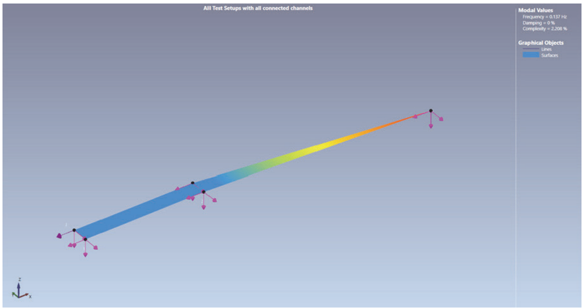

Figure 7 Mode 1 vertical vibrations

Table 3 Dynamic parameters obtained by Artemis Modal

Mode	**Frequency** [Hz]	**Damping** [%]	**Complexity**
1	0.137	0	2.208
2	0.215	0	6.814
3	0.273	0	53.972
4	0.386	2.736	49.023

4 Conclusion

The values of natural frequencies, vibration shapes, and damping were obtained through dynamic testing of the suspension bridge over the Jablanica Lake in Ostrožac using Operational Modal Analysis (OMA) and Frequency Domain Decomposition method. The excitation was caused by wind and the usual passage of vehicles, while traffic over the bridge was maintained during testing. This investigation approach demonstrated exceptional simplicity, cost-effectiveness, and high efficiency. During the testing, each of the 4 phases was recorded for 60 seconds, and the data was filtered using the Artemis Modal software package. Power spectra show approximately the same natural frequencies and modes for each of the test phases.

The expectedly small values of natural frequencies indicate the flexibility of the structure. Animation revealed dominant vertical vibration of the superstructure, but longitudinal and lateral vibrations also occur concurrently with vertical vibrations. It can be concluded that the structure moves spatially in all three directions, as confirmed by the complexity of the shapes given as percentages. It is assumed that such a response, indicating almost independent vibration of individual segments, is a result of the actual condition of the structure, i.e., pin-jointed structure, as well as observed deficiencies, such as the lack of pins and pin locks at certain locations.

Due to the visually noticed significant vibrations of back-stay cables, testing was conducted on two stays, resulting in natural frequencies of cca. 2.4 Hz and a cable force of cca. 660 kN. The Scruton number was calculated to be 35.9, which is higher than the recommended. Increasing the mass and damping of the cable increases the Scruton number and thus reduces the oscillation amplitudes. Connecting the cables and installation of dampers can be considered to reduce vibrations, which has proven to be a good solution in previous studies.

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