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Research Article

Identification of Modal Parameters of Bridges Using Ambient Vibration Measurements

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The paper provides an overview of ambient vibration tests and numerical analysis performed in the framework of Project NATO SfP 983828. The aim of the research is the definition of the dynamic characteristics of bridges on the examples. The paper considers three case studies: two older existing bridges and one newly constructed bridge. A comparative analysis of natural frequencies and mode shapes, obtained by ambient vibration measurements (AVM) and mathematical models (AMs), was carried with the aim to demonstrate the usefulness of ambient vibration tests for identification of the modal parameters of the tested bridge structure. Agreement between AVM and AMs results is very good. The mode shapes are very similar. Some differences between computed and measured frequencies were obtained, which can be attributed to the real nature of the boundary conditions, the uncertainty in the material properties of structure elements, and the mathematical models assumptions.

1. Introduction

A maintaining of bridges is particularly interesting because they represent the most sensitive point of a traffic routes. In order to ensure their reliability, and especially their stability and serviceability, it is important to analyze the bridge structure loaded by static and dynamic excitation. According to existing regulations, compliance of structures performance in real with the design structure performance defines with bridge test load (static and dynamic test load). However, loading of the bridge is often expensive. It is necessary to consider that traffic interruption during bridge test, even occasionally, can have significant consequences because the bridges are often the vital point in the transport network.

In order to ensure a design service life, it is important to monitor the bridge structure behaviour. Major threats to bridges primarily consist of the aging of the structural elements, earthquake-induced shaking, and standing waves generated by windstorms. For both, newly constructed bridges and older existing bridges, it is desirable to measure the dynamic properties, resonant frequencies, mode shapes, and modal damping of the bridges to understand better their dynamic behaviour under normal traffic loads as well as extreme loads such as those caused by seismic events or high winds. Trend of recent and future research in this area is the development of new methods of bridges assessment through the monitoring of deformation states. Current nondestructive testing methods for the monitoring and the diagnosis of structures, such as acoustic, ultrasonic, electromagnetic, and radiographic methods, are very useful for the evaluation of the state of condition of structures but sometimes are unsuited for continuous monitoring. They are considered as being local methods since they require detailed inspection of small parts of the structure and assume that the damaged zones are a priori known. The need for more global methods of damage diagnosis led to the development of dynamic evaluation methods based on vibration measurements.

In recent years, techniques based on ambient vibration recordings have become a popular tool for characterizing the seismic response and state-of-health of strategic civil

infrastructure. For civil engineering structures, ambient vibration tests are preferred over forced vibration ones because the artificial excitation of large structures having low natural frequencies is quite difficult and expensive. For very important traffic routes dynamic behaviour of bridges can be obtained by a series of field tests. For example, the Lion-Head River Bridge [1], during its construction stage, was processed by several types of tests, including an ambient vibration test, a force vibration test, a free vibration test, and a truck test. The ambient vibration test can detect the natural frequency of bridges in three directions. The paper [2] concludes that the measurement of ambient vibrations is accurate and rational technique for the determination of modal parameters of large structures. The forced vibration test can detect the vibration frequency, the damping ratio, and the vibration modes. A free vibration test is able to detect the frequency, the damping ratio, and the vibration mode in transverse direction.

In the ambient vibration tests, operation disturbances can be avoided and the measured response is representative of the actual operating conditions of the structures which vibrate due to natural excitation. Such techniques enable to detect possible degradations which can be internal, be invisible, and be localized anywhere in the structure [3]. The techniques rely on the fact that damage modifies the rigidity, the mass or the property of energy dissipation which influences the dynamic response of the structure. The inherent uncertainties in ambient vibration data have been recognized as one of the main barriers against the application of vibration-based damage identification techniques on real-life bridges. Therefore, a statistical damage identification procedure for bridge health monitoring is necessary. The procedure is presented in [4].

In areas less exposed to seismic hazard, vibration-based techniques also represent important tools for civil engineers, for instance, if they have to deal with structures exposed to heavy operational demands for extended periods of time and whose structural integrity might be in question or at risk. A continuous monitoring of such structures allows for an identification of their fundamental response characteristics and the changes of these over time, the latter representing indicators for potential structural degradation. A good example for such a case is the Adolphe Bridge in Luxembourg City, presented in [5]. Under normal operating conditions knowledge of the dynamic properties can be used to assess the effects of traffic loading on the fatigue life of the structure and to determine dynamic load factors for these structures [6].

Also, it is important to distinguish abnormal changes in dynamic characteristics caused by structural damage from normal changes due to environmental fluctuations. Identifying features that can accurately distinguish a damaged structure from an undamaged one is the focus of most Structural Health Monitoring (SHM) techniques. In [7] the modelling of temperature effects on modal frequencies is presented.

This paper presents some parts of experimental research in the framework of the NATO Project "Seismic Upgrading of Bridges in South-East Europe by Innovative Technologies," with ambient vibration tests of selected bridge prototypes. The aim of the tests was evaluation of dynamic characteristics. Measurements in the frame of the working package WP 1.4, preliminary presented during the Project in [8–11], were performed with the aim to collect input data to make numerical model of bridge structure real behaviour. Three case studies are presented in the paper. Frequency and vibration shapes of surveyed bridges are determined by measuring the ambient vibrations. The projected numerical models of surveyed bridges were done along with experimental studies. The paper includes a comparison of the dynamic properties (frequency and vibration forms) that were determined by measuring the ambient vibrations and using numerical models.

2. Ambient Vibration Tests and Numerical Models

In order to calculate the modal parameters from ambient noise recordings, many techniques are available. In [12] the modal parameters of structures are extracted from output-data only using subspace methods and the wavelet transform. Identification of modal vibration properties of the superstructures using the Enhanced Frequency Domain Decomposition (EFDD) technique is presented in [13]. The Enhanced FDD technique allows us to extract the resonance frequency and the damping of a particular mode by computing the auto and cross-correlation functions. The Single Degree of Freedom (SDOF) Power Spectral Density function, identified around a peak of resonance, is taken back to the time domain using the Inverse Discrete Fourier Transform. The resonance frequency is obtained by determining the zero crossing times and the damping by the logarithmic decrement of the corresponding SDOF normalized auto correlation function. Using the Stochastic Subspace Identification (SSI) method is presented in [14]. SSI algorithms identify a stochastic state-space model of the structure. The resulting model can then be translated into a more convenient structural model form for engineering interpretation of the results. The state-space model can be related to both modal model and finite element (FE) model formulations. The dynamic behaviour of civil engineering structural systems is traditionally modelled through discrete finite element approximations. System identification of discretely sampled responses reformulates the FE model into a discrete-time state space model form. Comparative analysis of the experimental dynamic characteristics obtained by EFDD and SSI methods was performed in the papers [15, 16]. The identification of modal parameters with the Covariance driven Stochastic Subspace Method (SSI-COV), a time domain method, and the poly-reference Least Squares Frequency Domain Method (p-LSCF) is presented in [17]. The complementary peak picking (PP) method in the frequency domain and more advanced SSI method in the time domain were implemented for modal parameter identification in the paper [18].

The experimental modal analysis, presented in the case studies (task 3), was carried out by using two output-only procedures: the PP and EFDD, implemented in the commercial computer program ARTeMIS. The implementation of the methods for newly erected bridge and historic bridge was presented in the papers [19, 20]. The PP method is the simplest known method for identifying the modal parameters

of civil engineering structures subjected to ambient vibration loading. The method is initially based on the fact that the frequency response functions (FRF) goes through extreme values around the natural frequencies. The frequency at which this extreme value occurs is a good estimate for the frequency of the system. In the context of ambient vibration measurements the FRF is only replaced by the auto spectra of the ambient outputs. In such a way the natural frequencies are determined from the observation of the peaks on the graphs of the averaged normalized power spectral densities (ANPSDs). The ANPSDs are basically obtained by converting the measured accelerations into the frequency domain by a discrete Fourier transform. For tower-like structures the PP may become problematic since the bending modes along any of the 2 principal axes and/or any of the torsion modes are likely to have closely-spaced frequencies. For bridge structures this seems to be less the case [21]. The EFDD technique is a refinement of the frequency domain decomposition (FDD) technique. Both the PP and the FDD/EFDD methods are based on the evaluation of the spectral matrix in the frequency domain.

Mathematical modelling and analysis of bridges include many assumptions from the mechanical properties of materials to boundary conditions and analysis options. Therefore, the accuracy of the finite element (FE) model depends on the experimental validation of the numerical results. Update of 3D FEM model of bridges through comparison of mathematical and experimental dynamic characteristic is studied in [1, 15–20].

Ambient vibration measurements on location of bridges in Sarajevo, Tuzla, and Skopje were performed in the NATO Project. Geophones (bridges in Sarajevo and Skopje) and accelerometers (bridge in Tuzla) were used for measurements. Ambient vibration measurements were performed using Instantel Blastmate Pro4 vibration monitor with Sample Rate 1024 to 16384 S/s per channel and Quantum MX840A Universal Amplifier. The measurements were made using Three-Axial Geophone Sensors and 0.5 g Accelerometer Sensors. Used Three-Axial Geophone has range of measurement up to 254 mm/s, with resolution 0.127 mm/s or 0.0159 mm/s, and accuracy ± 5% or 0.5 mm/s. Frequency range is from 0 to 315 Hz. Operating frequency range of used accelerometer is 0.5 to 500 Hz, with acceleration range 0.5 g and resolution 0.00025 g. The measurements are done with time-synchronized geophones complemented with a special trigger for geophones, triggering simultaneous five geophones in 1/1000 s. For identification the frequencies of the structure, the signals collected in the time domain were transformed to the frequency domain by using Fast Fourier Transform (FFT). Many peaks appeared in the Fourier spectrum diagram due to the disturbance and noise in the field. The Power Spectrum method [22] is used to filter out the disturbance and noise and to clarify the structural frequencies.

Ambient vibration tests in the case studies have been used in the scope of the identification of the modal parameters of structures, namely, the natural frequencies and mode shapes. Assessment of the natural frequencies and mode shapes of the bridges is performed using mathematical and experimental method. Mathematical modelling is carried out using the finite element method (FEM) and the experimental modelling is performed using the ambient vibration test (AVT).

3. Case Study 1: The Bridge over River Bosnia in Sarajevo, Bosnia, and Herzegovina

3.1. Description of the Bridge and Measurements Setup. The bridge over the river Bosnia, shown in Figures 1 and 2, is a 45-year-old bridge located on the route M05, section Lasva-Stup. Its overall length is 117 m and is comprised of two spans of 21 m and three spans of 25 m. Width of the bridge is 10.40 m. Each span is built with four precast posttensioned I beams of 1.30 m height with recessed deck. The five spans of the deck are interconnected through a 16 cm thick continuity slab over the piers. The superstructure is supported by rectangular laminated elastomeric bearings at the two abutments (1 and 6) and four piers (2, 3, 4, and 5).

Arrangement of measuring points is shown in Figure 3. Ambient vibration test was made with five geophones. One geophone was used for the reference point (stationary point) 6R, while the remaining 4 geophones moved to individual measuring points 1–17, according to the arrangement. With the use of trigger nine triggering is made with five geophones, one test measurement, and 8 verified measurements (M1–M8). Table 1 shows the different stations distribution of each setup.

During all tests normal traffic flow was permitted. Testing started in the right lane (1R–11R). Each setup yielded 3 base channels (6R) and 12 moveable channels. The ambient vibration was simultaneously recorded for 120 s resulting in 491,520 data points per data set. Test equipment is shown in Figure 4.

3.2. Ambient Vibration Measurements. Velocity of vibration was measured on the individual measuring points in real time. Then frequency spectrums were obtained by using the FFT. In Figures 5–10 results of M1 measurement are presented. The dominant frequencies measured at measuring points are given in Table 2.

From the results presented in Table 2 it can be seen that the dominant frequencies of vibration of the superstructure measuring points, except for a few results, are in range from 3.81 to 7.95 Hz, depending on position of the measuring point, over the support or in the middle of the field. The obtained results of measurements (dominant frequency) were within the limits of test results on similar concrete bridges. There is a difference between the dominant frequency of vibration abutments and piers on either coast, which will be the subject to further analysis by defining the soil-structure interaction.

In this study, two different methods, which are EFDD in the frequency domain and SSI in the time domain, are used for modal parameter extraction. Singular values of spectral density matrices, average of auto spectral densities, and stabilization diagram of estimated state space model of the first test setup attained from vibration signals using EFDD

Figure 1: The bridge over river Bosnia, close to Sarajevo.

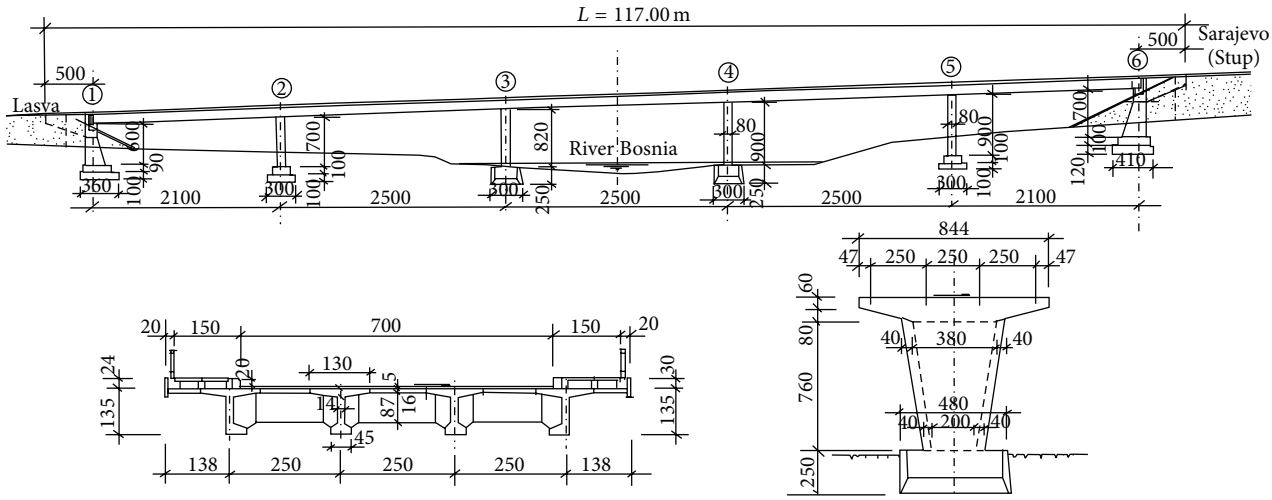


Figure 2: Elevation and cross sections of the bridge.

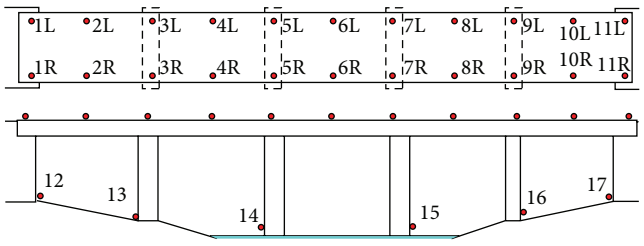


Figure 3: Measurements setup.

Figure 4: Test equipment.

and SSI methods are shown in Figure 11. Natural frequencies and damping ratios obtained from the test setup are given in Figure 12.

Table 1: Setup of stations during the ambient vibration testing.

Setup	Moveable stations	Base stations
M1	4R, 5R, 7R, 8R	6R
M2	2R, 3R, 9R, 10R	6R
M3	1R, 2R, 10R, 11R	6R
M4	4L, 5L, 6L, 7L	6R
M5	3L, 4L, 8L, 9L	6R
M6	1L, 2L, 10L, 11L	6R
M7	12, 13, 16, 17	6R
M8	12, 13, 16, 17	6R

L: left lane; R: right lane.

3.3. Mathematical Models and Comparative Analysis of Experimental and Mathematical Modal Parameters. Two mathematical models (AM1 and AM2) were created using the finite elements. The FEMs were created using SAP2000N. For both models the deck was modelled as a shell element and girders as elastic beam elements, as this approach provides effective stiffness and mass distribution characteristics of the bridge. The bridge superstructure itself is expected to remain essentially elastic during earthquake ground motions.

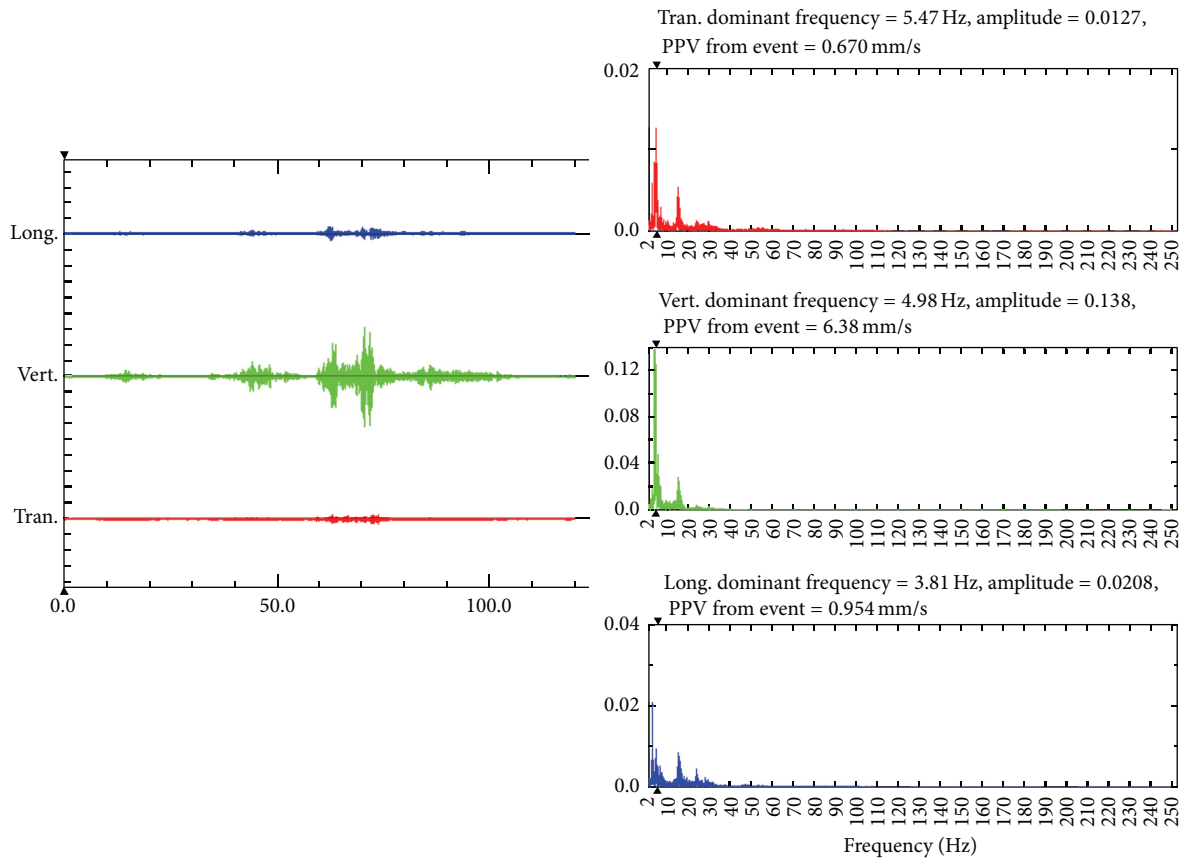


Figure 5: Measurement on base station 6R (middle of the span).

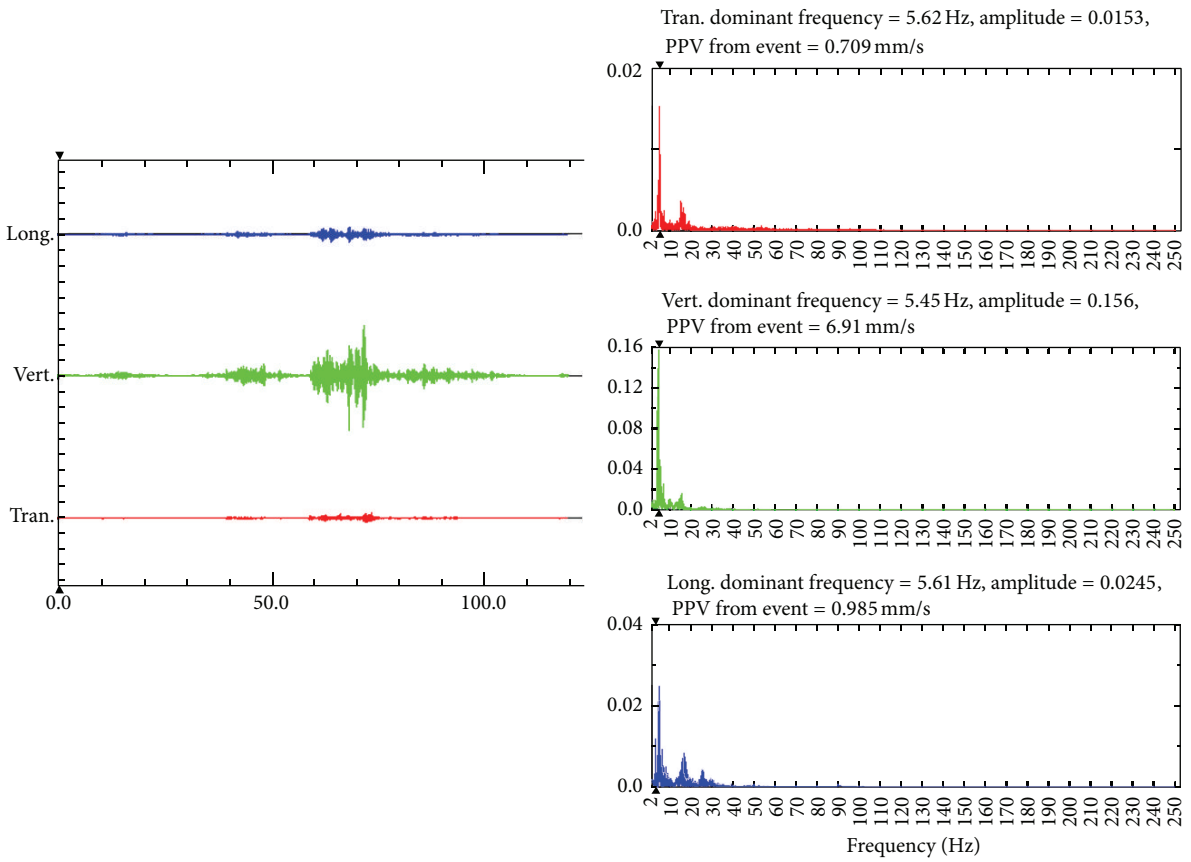


Figure 6: Measurement on moveable station 4R (middle of the span).

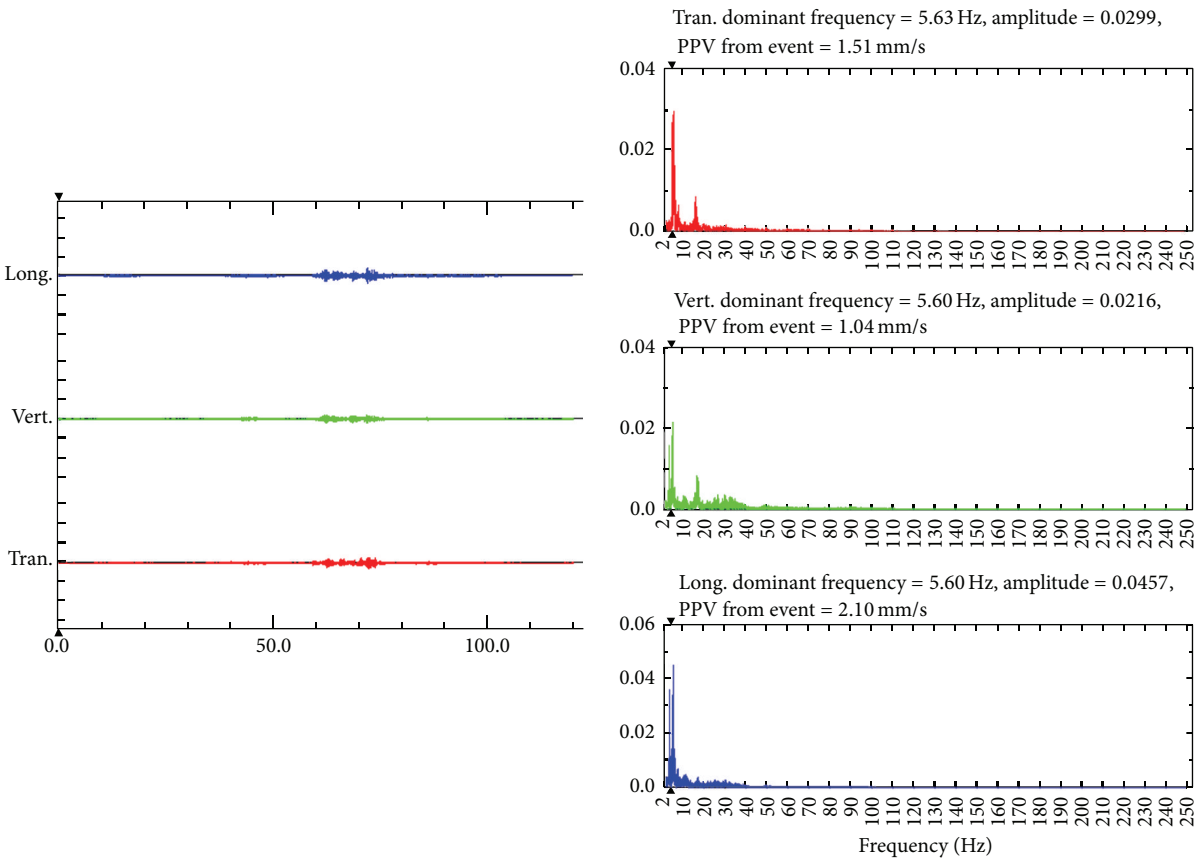


Figure 7: Measurement on moveable station 5R (over the support).

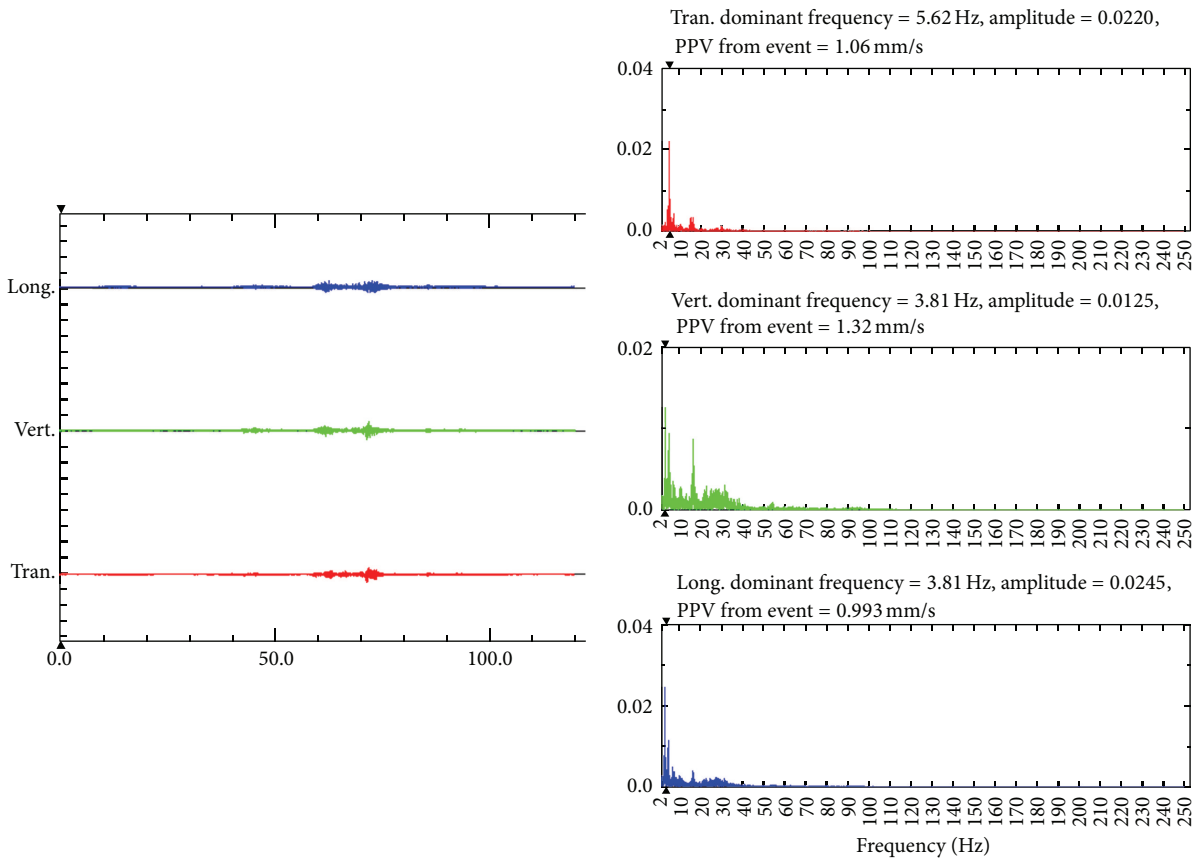


Figure 8: Measurement on moveable station 7R (over the support).

Table 2: Measured dominant frequency.

Measuring points	Trans. freq. (Hz)	Vert. freq. (Hz)	Long. freq. (Hz)
M1: 4R, 5R, 6R, 7R, 8R	5.62, 5.63, 5.47, 5.62, 5.63	5.45, 5.60, 4.98, 3.81, 5.61	5.61, 5.60, 3.81, 3.81, 3.81
M2: 2R, 3R, 6R, 9R, 10R	5.77, 5.97, 16.4, 5.97, 6.00	7.69, 5.50, 4.97, 5.43, 7.58	19.6, 5.43, 3.73, 5.43, 6.16
M3: 1R, 2R, 6R, 10R, 11R	5.76, 5.75, 5.57, 5.75, 5.75	7.85, 7.86, 4.95, 5.73, 5.73	21.7, 5.56, 3.85, 5.56, 5.56
M4: 6R, 4L, 5L, 6L, 7L	5.76, 5.76, 5.76, 5.49, 5.76	5.38, 5.74, 5.76, 5.36, 17.2	5.48, 5.48, 5.48, 5.48, 3.88
M5: 6R, 3L, 4L, 8L, 9L	5.54, 6.48, 5.75, 7.74, 5.02	5.02, 5.54, 5.78, 5.02, 5.54	3.86, 5.54, 5.54, 17.2, 5.54
M6: 6R, 1L, 2L, 10L, 11L	16.3, 6.70, 19.3, 5.67, 8.24	6.85, 20.3, 6.70, 8.17, 20.9	3.92, 7.95, 19.50, 6.29, 6.29
M7: 6R, 12, 13, 16, 17	4.95, 14.1, 4.99, 2.01, 2.00	4.99, 2.27, 4.99, 2.00, 2.01	16.1, 5.50, 5.50, 2.01, 2.02
M8: 6R, 12, 13, 16, 17	6.84, 16.6, 5.84, 2.00, 2.00	6.84, 2.39, 17.1, 2.00, 2.00	3.91, 6.28, 6.37, 2.00, 2.00

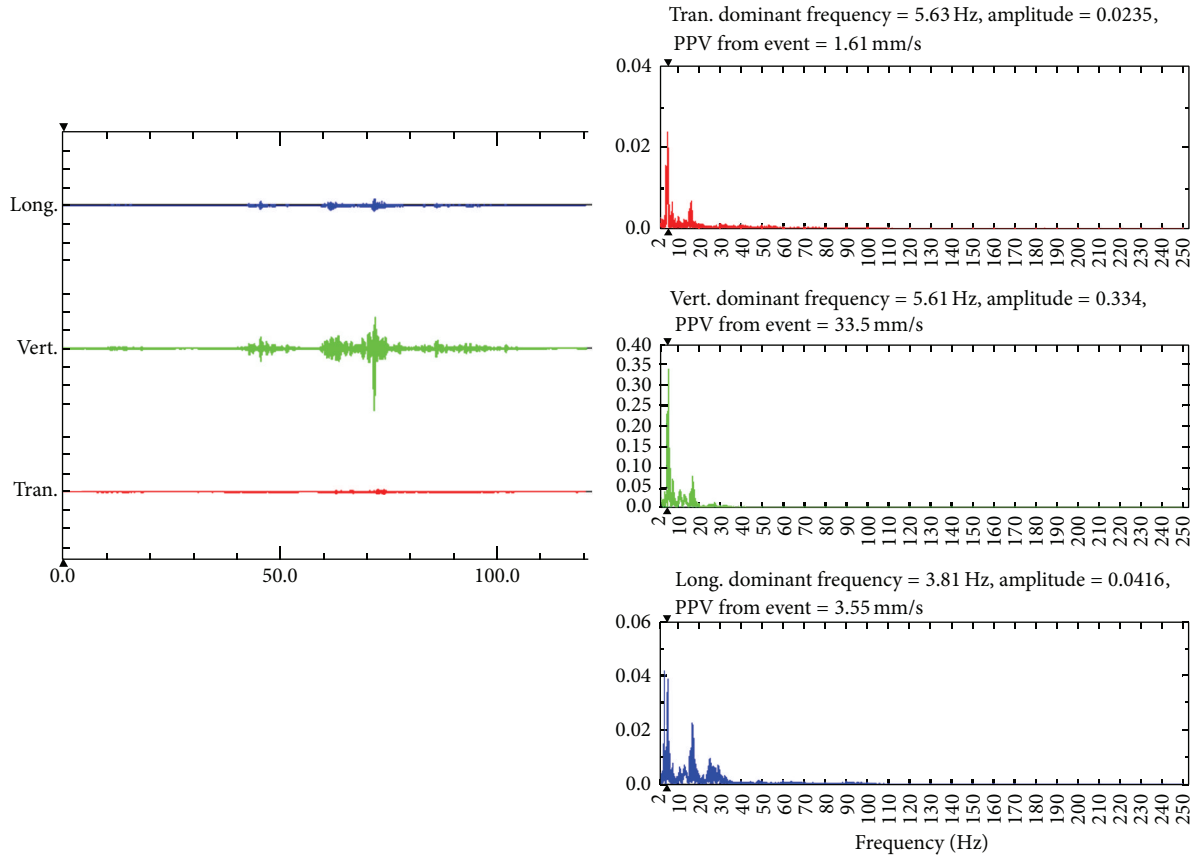


Figure 9: Measurement on moveable station 8R (middle of the span).

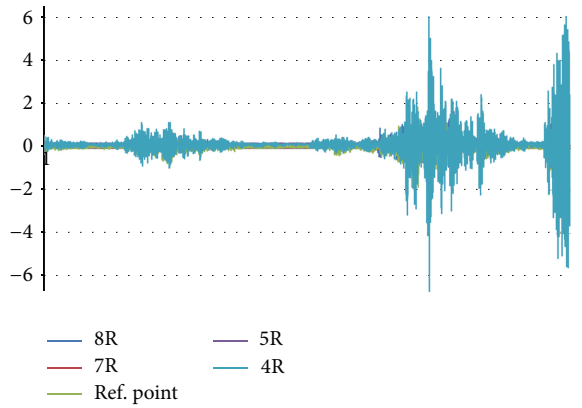


Figure 10: Comparison of vertical channels.

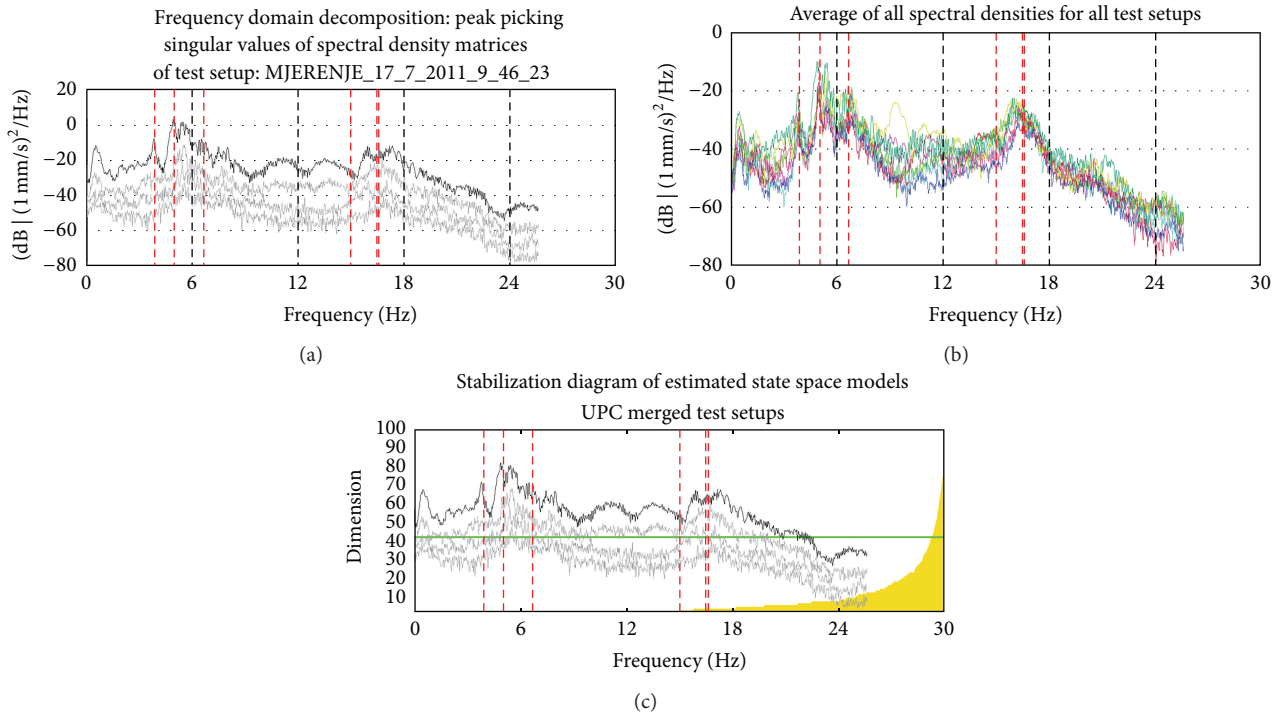


Figure 11: Dynamic characteristics attained from the test using EFDD and SSI methods: (a) singular values of spectral density matrices, (b) average of auto spectral densities, and (c) stabilization diagram of estimated state space model.

Dimensions of neoprene bearings are 300/400/100 mm. For model AM1 neoprene bearings were modelled with vertical and horizontal springs. The stiffness of the vertical springs was calculated using [23]

$$k = \frac{EA}{t}, \tag{1}$$

where E is Young's modulus ($E = 630\ \mathrm{N/mm^2}$), A is the cross-sectional area, and t is the bearing thickness. The lateral shear capacity of bearings is controlled by the dynamic coefficient of friction between concrete and neoprene of 0.40. For model AM2 neoprene bearings were modelled as absolute stiff. For both models, modulus of elasticity for concrete C 30/37 was modelled using equations from (3.1) to (3.5) in [24]. Based on the mentioned equations the predicted modulus of elasticity E_{cm} after 45 years' service life was obtained. Modulus of elasticity $E_{c0} = 38{,}5\ \mathrm{GPa}$ was obtained according to Section 3.1.4 (2) in [24].

In the model AM1 soil was modelled with stiff linear springs, and in the model AM2 with solid elements. Modulus E, determined by preliminary geophysics soil measurements, was used for the solid elements. Values are presented in Table 3. Very stiff clay is at the depth of 10.0 m. The analysis of mathematical models showed that the aforementioned clay can be approximated by rigid springs for the purposes of determining the tone of oscillation. In Figure 13 mathematically identified first transverse and first vertical mode shapes of models AM1 and AM2 are presented. Modal analysis of models AM1 and AM2 was carried out with experimentally obtained damping ratios (see Figure 13).

Table 3: Modulus E for layered soil model.

Depth (mm)	E (MPa)
4.0	1797
6.0	3193
8.0	7179
10.0	∞ (very stiff clay)

Table 4: Experimentally and mathematically identified modal frequencies (Hz).

	First transverse mode	First vertical mode
Ambient vibration test	3.855	4.971
Mathematical model AM1	3.641	3.798
Mathematical model AM2	2.513	4.639

Since the scope of the research is the comparison of the modal parameters identified by ambient vibration tests and mathematical models, two FEMs (AM1 and AM2) were developed. The comparison is made for the first transverse and the first vertical modes. Experimentally and mathematically identified modal frequencies are presented in Table 4.

When the mathematically (AM1) and experimentally identified first transverse mode are compared with each other, it is seen that there is a good agreement between natural frequencies. There was an approximate 5% difference. Also, it is good agreement between natural frequencies of first vertical mode identified by AM2 and AVT with approximate 6% difference. AM1 fails to predict well the first vertical mode

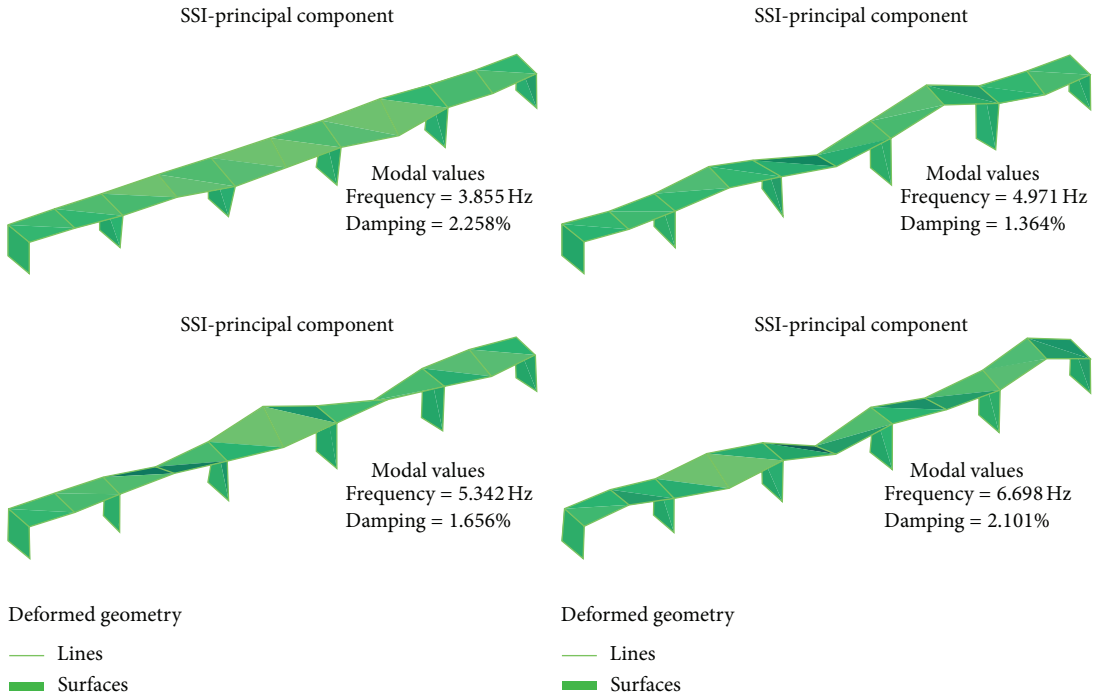


Figure 12: Experimental modal parameters attained from the test setup.

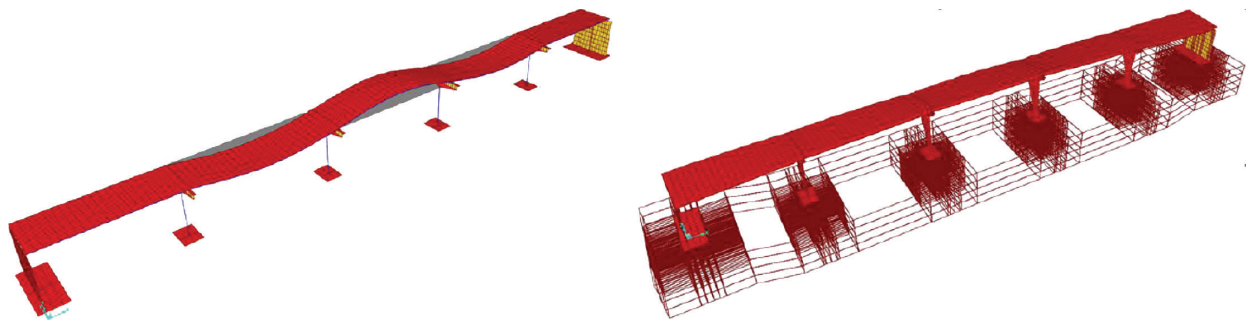


Figure 13: Mathematically identified mode shapes: (a) model AM1, 1st vertical mode; (b) model AM2, 1st transverse mode.

as they exhibit large deviations from the identified modal frequencies that exceed 23% and for the first transverse mode identified by AM2 and AVT difference was 35%.

There is a good agreement between vertical mode shapes obtained by AVM and AMs (Figure 14). The deviations between the experimentally identified and mathematically predicted first transverse mode shapes (Figure 15) can be attributed partially to the uncertainty in the material properties of structure elements and neoprene bearings after 45-year service. But also deviations could be partially due to unexpected channel errors, which will be studied and presented in next paper.

Based to the study it can be seen that AVM can be used to assess the dynamic properties of older existing bridges, but more detailed investigation is necessary due to identify influential parameters and reasons for identified deviation.

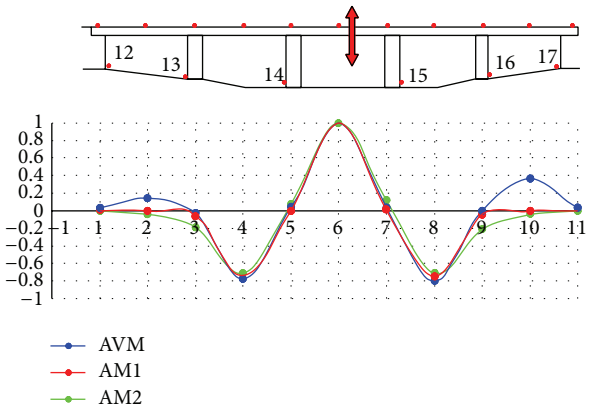


Figure 14: Experimentally and mathematically identified first vertical mode shapes.

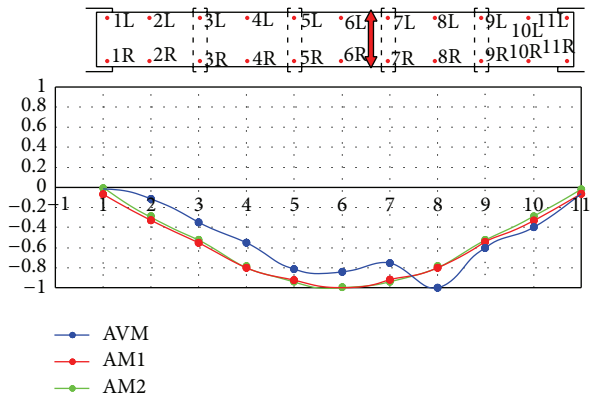


Figure 15: Experimentally and mathematically identified first transverse mode shapes.

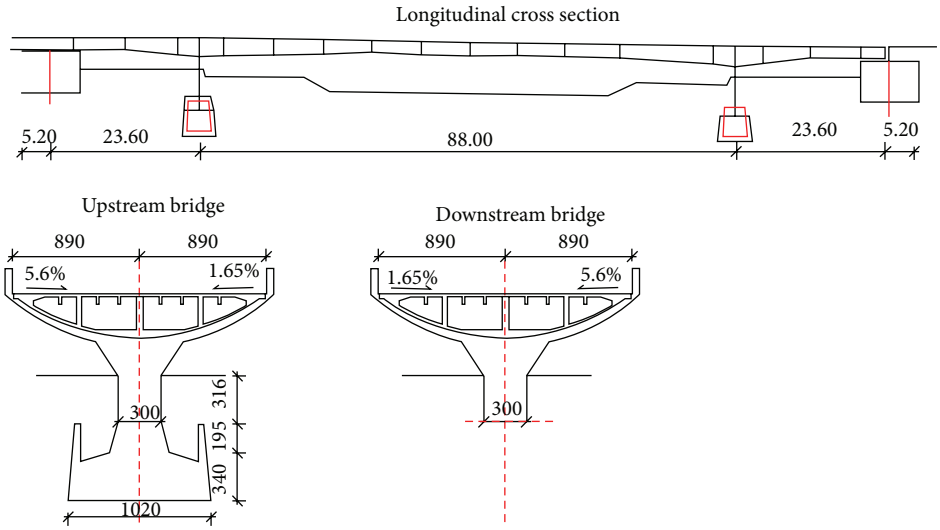


Figure 16: The twin bridge "Goce Delcev," across rivers Vardar, Skopje, and Macedonia, elevation, and cross section of the bridge.

4. Case Study 2: The Twin Bridge "Goce Delcev" in Skopje, Macedonia

4.1. Description of the Bridge and Measurements Setup. The twin bridge "Goce Delcev" is prestressed girder bridge with multibox cross section (Figures 16 and 17). Total length of the bridge is 136.00 m with three spans 24.00 + 88.00 + 24.00. The main supports are middle pears and counter weight at the ends of the bridge.

The bridge is a very complex structure. During the serviceability period of about 40 years, several revitalization works have been done in order to improve the structural conditions of the bridge. Based on structural state diagnosis, an appropriate bridge revitalization concept has been selected and realized. The generated experimental results from the ambient vibration tests are very important in verification of the formulated mathematical models and reanalysis of the bridge seismic safety. Arrangement of measuring points is shown in Figure 18.

Arrangement of measuring points is the same for both bridges. One point (point R; point 16) is selected for stationary station. Two measuring points are selected outside both ends of both bridges (points numbers 37, 38, 39, and 40). Two measuring points are selected at the bottom of the two middle pears of each bridge (points numbers 41 and 42). Bridge measuring points from number 1 up to number 36 are distributed in two lines at a longitudinal distance of 8.0 m. Table 5 shows the different stations distribution of each setup.

4.2. Ambient Vibration Measurements. The traffic was held up during the testing procedure. Each measurement setup yielded 3 base channels (16R) and 12 moveable channels. The ambient vibration was simultaneously recorded for 120 s resulting in 491,520 data points per data set. In Figures 19 and 20 results of M3 measurement, velocity in real time, and frequency spectrums are presented. Comparison of transversal channels is presented in Figure 21.

The dominant frequencies measured at measuring points of the upstream bridge are given in Table 6.

From the results presented in the Table 6 it can be seen that the dominant frequencies of vibration of the superstructure measuring points are in range from 1.28 to 2.92 Hz.

Figure 17: The twin bridge "Goce Delcev," across rivers Vardar, Skopje, and Macedonia.

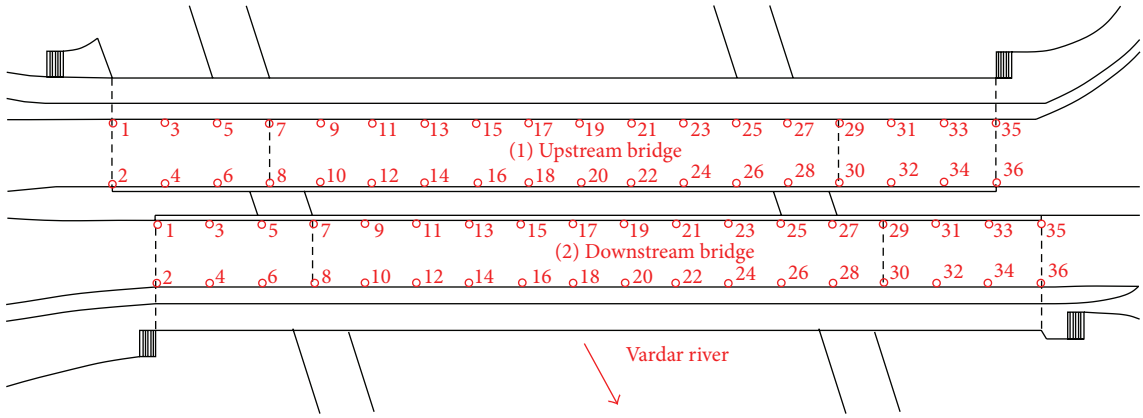


Figure 18: Measurements setup.

Table 5: Setup of stations during the ambient vibration testing.

Setup	Moveable stations	Base stations
M1	37, 38	16R
M2	1, 2, 3, 4	16R
M3	1, 2, 3, 4	16R
M4	5, 6, 7, 8	16R
M5	9, 10, 11, 12	16R
M6	13, 14, 15	16R
M7	17, 18, 19, 20	16R
M8	21, 22, 23, 24	16R
M9	25, 26, 27, 28	16R
M10	29, 30, 31, 32	16R
M11	33, 34, 35, 36	16R
M12	33, 34, 35, 36	16R
M13	39, 40	16R
M14	41, 42	16R

In this study SSI method is used for modal parameter extraction. Singular values of spectral density matrices, average of auto spectral densities, and stabilization diagram of estimated state space model of the first test setup attained from vibration signals using the SSI are shown in Figure 22. Modes obtained using the SSI are 1.273 Hz, 2 Hz, 3.235 Hz, and 4.65 Hz (Figure 23).

4.3. Numerical Models and Comparative Analysis of Experimental and Numerical Modal Parameters. The mathematical model AM3 was created using the finite elements (SAP2000N). As the complete bridge structure was executed as monolithic structure, the superstructure and substructure were modelled as integrated structure. The superstructure was modelled as a shell element and piers as elastic beam elements, as this approach provides effective stiffness and mass distribution characteristics of the bridge. Modelling of concrete properties was briefly described in task in Section 3.3 (E_{c0} = 38, 5 GPa). The piers-soil contact was modelled as stiff support. In Figure 24 mathematically identified first vertical and second transverse mode shapes are presented.

Modal analysis of models AM1 and AM2 was carried out with experimentally obtained damping ratios (see Figure 24).

The scope of the research was the comparison of the modal parameters identified by AVM and AM3. The comparison was made for first vertical, second vertical, first transverse, and second transverse modes. Experimentally and mathematically identified modal frequencies are presented in Table 7.

When the mathematically (AM3) and experimentally identified (AVM) modes are compared with each other, it is seen that there is a very good agreement between natural frequencies of first modes. Also, there is a good agreement between first vertical, first transverse, and second vertical mode shapes obtained by AVM and AM3 (Figures 25–28).

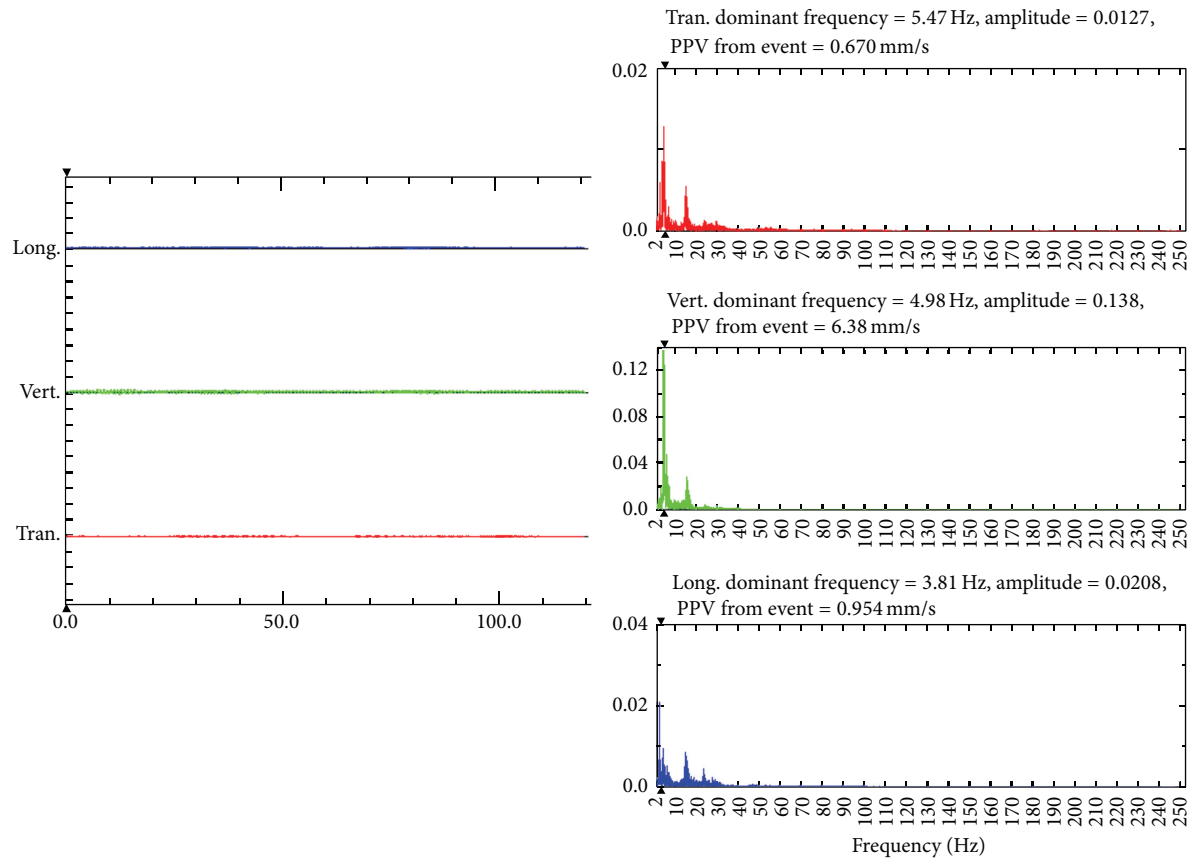


Figure 19: Measurement on base station 16R: velocity measurement in real time (120 s recording) and frequency spectrums.

Table 6: Measured dominant frequency of upstream bridge.

Measuring points	1st freq. (Hz)	2nd freq. (Hz)	3rd freq. (Hz)
16R, 1, 2, 3, 4	1.30V, 2.00VTL, 1.98L, 1.30V, 1.98	1.98VT, 2.88L, —, 1.99VTL, —	—, 3.31L, —, 2.86L, —
16R, 5, 6, 7, 8	1.28V, 2.03VT, 2.03VT, 2.03VT, 2.03VT	2.03VT, 2.92V, 2.92L, 2.92L, 2.92L	4,27V, —, —, —, —
16R, 9, 10, 11, 12	1.29V, 1.29V, 1.29V, 1.30V, 1.30V	2.05VT, 2.06VT, 2.06VT, 2.05VT, 2.06VT	—, —, —, 2.95L, —
16R, 13, 14, 15	1.28V, 1.28V, 1.28V, 1.28V	2.02VT, 2.02VT, 2.02VT, 2.02VT	—, 2.91L, —, 2.91L
16R, 17, 18, 19, 20	1.29V, 1.29V, 1.29V, 1.29V, 1.28V	2.03VT, 2.03T, 2.03VT, 2.03T, 2.03T	4,23V, 3.34L, —, 3.35L, 3.35L
16R, 21, 22, 23, 24	1.28V, 1.28V, 1.28V, 1.28V, 1.28V	2.02VT, 2.02VT, 2.02VT, 2.02VT, 2.01VT	—, 2.90L, —, 2.91L, 2.91L
16R, 25, 26, 27, 28	1.28V, 1.28V, 1.28V, 1.28V, 1.28V	2.00VT, 2.00VTL, 2.00VT, 2.00VT, 2.00VTL	—, 2.88L, 2.88L, 2.88L, 2.88L
16R, 29, 30, 31, 32	1.27V, 1.80VTL, 1.80VT, 1, 80V, —	2.10VT, 2.10VTL, 2.10VT, 2.10VT, 2.10VT	—, 2.88L, 2.88L, 2.91L, 2.91L
16R, 33, 34, 35, 36	1.28V, 1.28V, 1.28V, 1.28L, 2.00L	2.02VT, 2.02VTL, 2.02VTL, 2.02TL, 2.84L	—, 2.84L, —, 2.84L, —
16R, 37, 38	1.27V, 4.20VTL, 2.00T	2.01VT, —, 2.14V	4.23V, —, 8.88L
16R, 39, 40	1.29V, 2.90L, 2.07V	1.97VT, 7.83T, 4.63T	—, 13.7V, 11.8L
16R, 41, 42	1.27V, 2.00T, 3.24L	1.95VT, 2.52L, 4.2T	4.21V, 35.8V, 26.4V

V: vertical; T: transverse; L: longitudinal.

The significant difference, 20.50%, was identified in natural frequencies of second transverse modes. The larger deviation was identified between the experimentally identified and mathematically predicted second transverse mode shapes. For the exact definition of obtained deviations detailed investigation is necessary. The reasons could be boundary conditions, geometrical and material properties assumptions, and eventually unexpected channel errors.

5. Case Study 3: The Cable-Stayed Bridge in Tuzla, Bosnia, and Herzegovina

5.1. Description of the Bridge and Measurements Setup. This case study is interesting because short tower is designed due to urban conditions (Figure 29), which resulted in a small angle of the first and second cable. Consequently during the construction, increased deflections appeared in the part

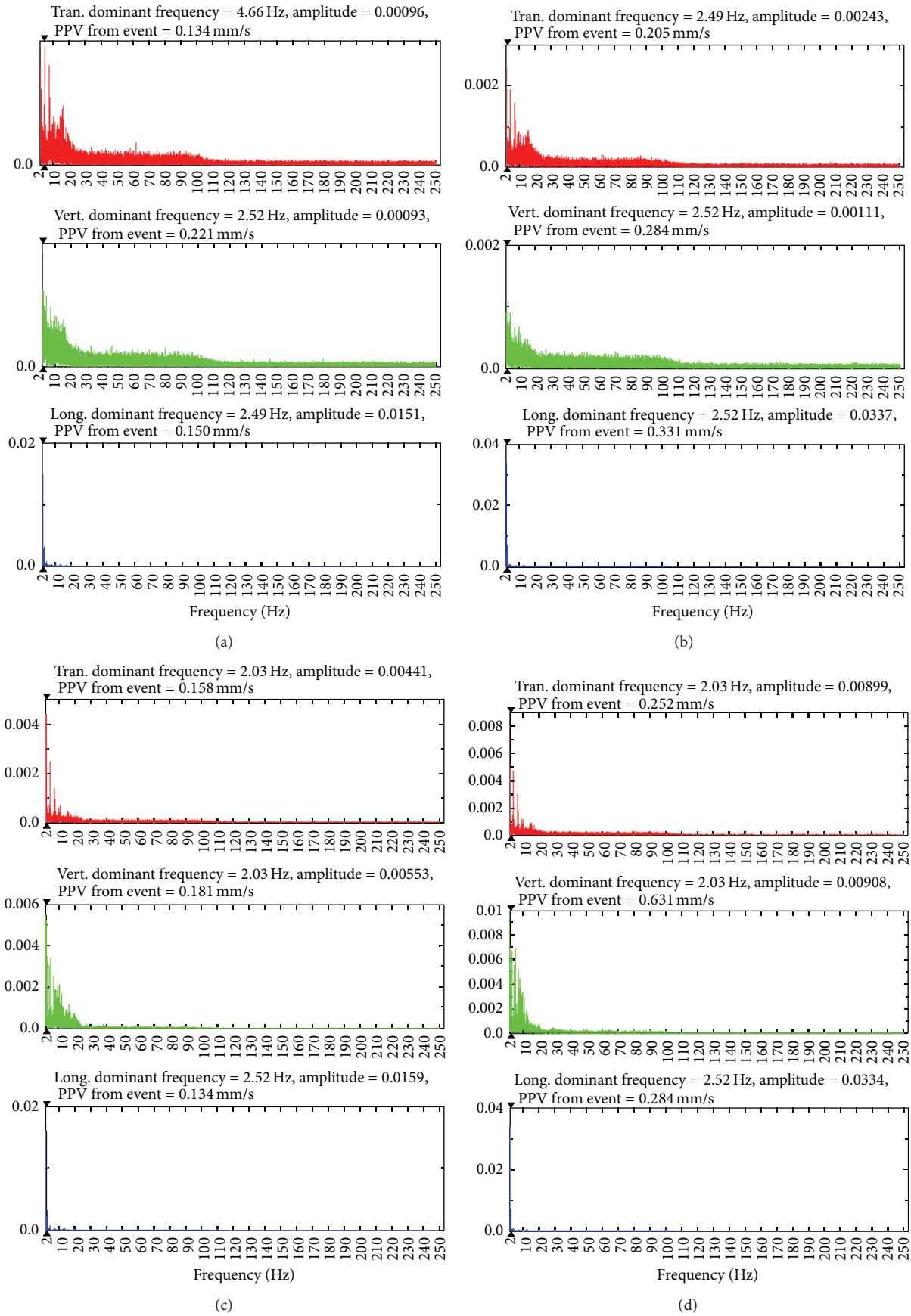


Figure 20: Frequency spectrums: (a) moveable station 1; (b) moveable station 2; (c) moveable station 3; (d) moveable station 4.

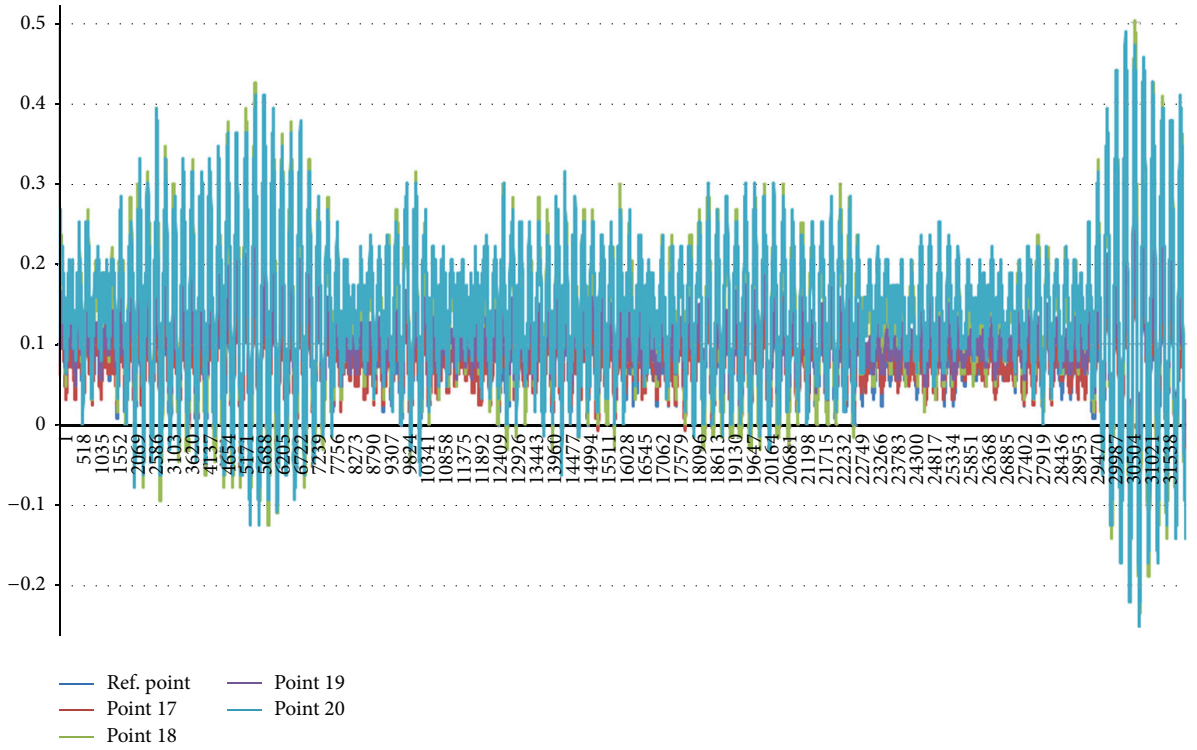


Figure 21: Comparison of transversal channels.

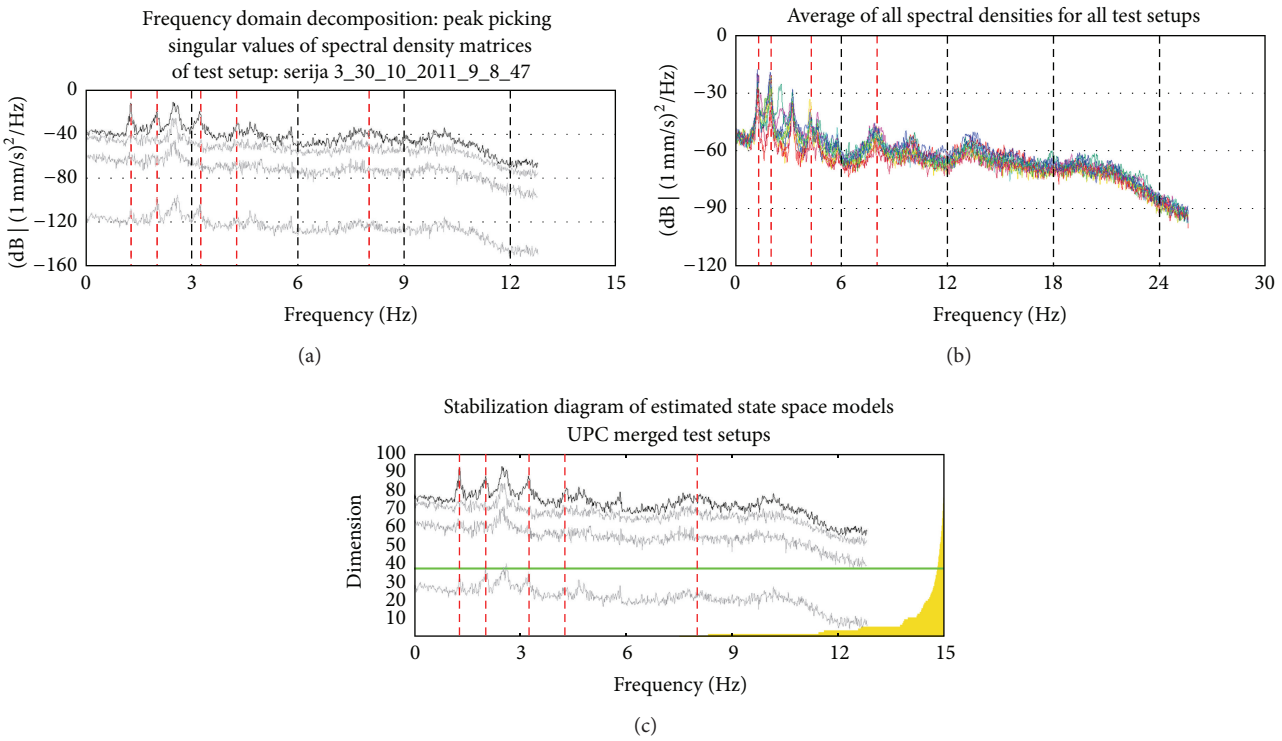


Figure 22: Dynamic characteristics attained from the test using PP and SSI method: (a) singular values of spectral density matrices, (b) average of auto spectral densities, and (c) stabilization diagram of estimated state space model.

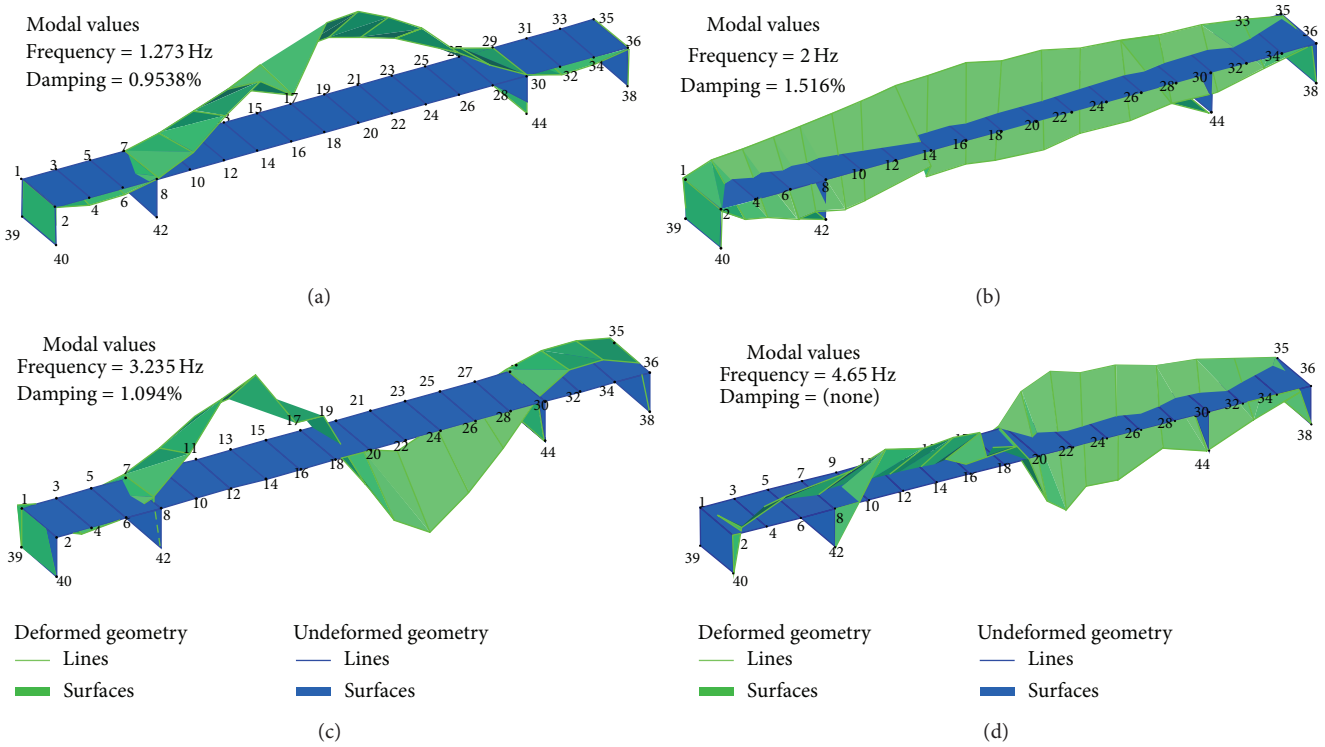


Figure 23: Vibration modes obtained by SSI (Artemis): (a) 1st vertical mode; (b) 1st transverse mode; (c) 2nd vertical mode; (d) 2nd transverse mode.

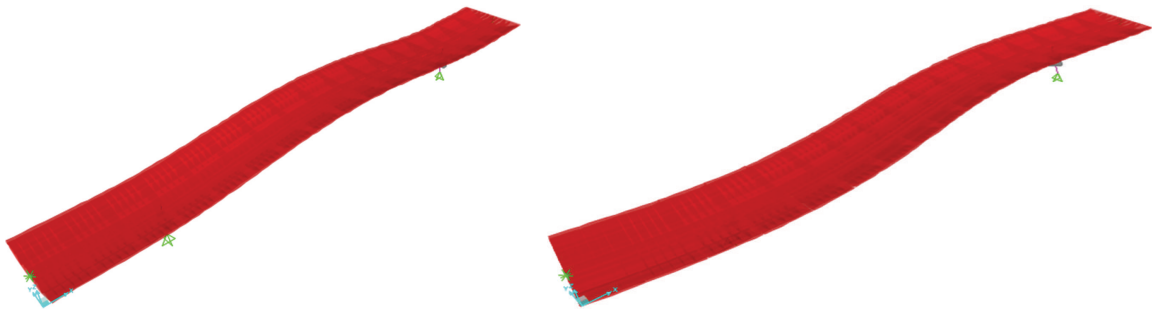


Figure 24: Mathematically identified mode shapes: (a) 1st vertical mode; (b) 2nd transverse mode.

Table 7: Experimentally and mathematically identified modal frequencies (Hz).

Modes	AVM	AM3
First vertical	1.273	1.273
First transverse	2.000	1.921
Second vertical	3.235	3.542
Second transverse	4.65	5.852

of the structure supported by two mentioned cables. Bridge testing and ambient testing were performed due to determine the behaviour of cable-stayed overpass for service conditions. The results of AVM and comparative mathematical analysis are the following.

The overpass is used for crossing pedestrians and vehicles over the city streets in Tuzla. It was designed and constructed as the cable-stayed bridge (Figure 29). The cross section of the bridge consists of two traffic lanes 6.0 m width and pedestrian lane. The total width of the bridge is 14.6 m. The bridge span is 32.0 m. Superstructure consists of two main prestressed beams and composite bridge deck. Composite bridge deck consists of concrete slab with variable thickness 16–36 cm, which is coupled with steel I beams with 3.25 m grid. Steel beams are profiles HEB700, steel quality S355. Complete superstructure is designed with concrete C30/37, reinforcement B500, and prestressed cables 1570/1770 N/mm^2. Main prestressed beams are hanged on four cables. The primary load-bearing structure consists of a reinforced concrete tower, cables, and steel ties. Reinforced concrete tower is 9.5 m

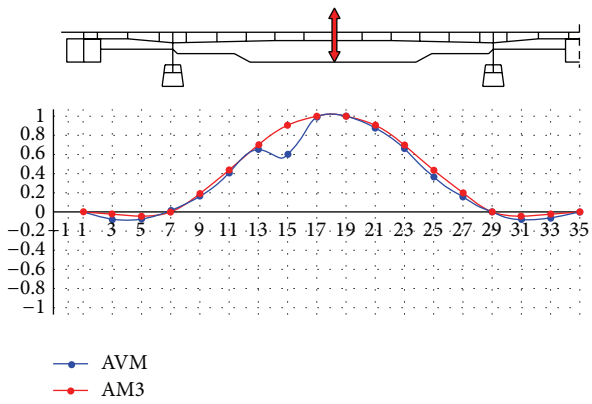


Figure 25: Experimentally and mathematically identified first vertical mode shapes.

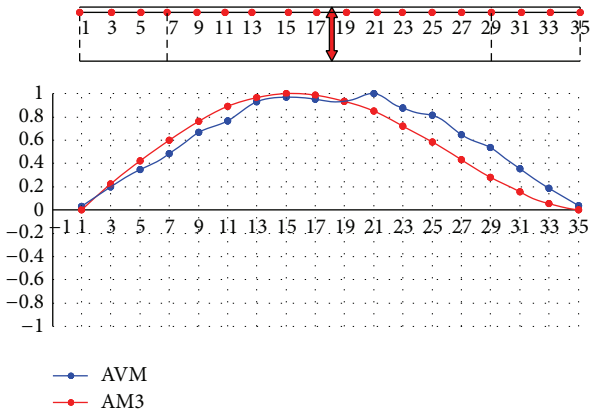


Figure 26: Experimentally and mathematically identified first transverse mode shapes.

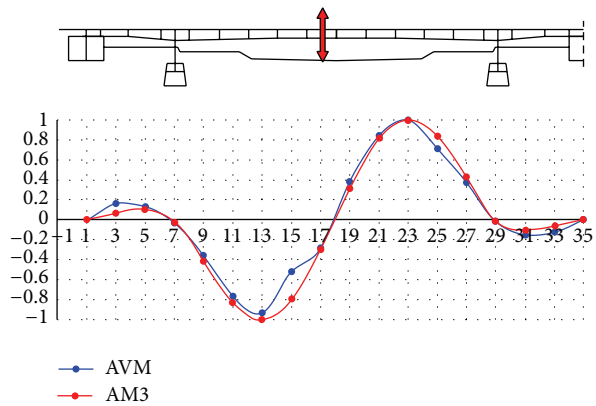


Figure 27: Experimentally and mathematically identified second vertical mode shapes.

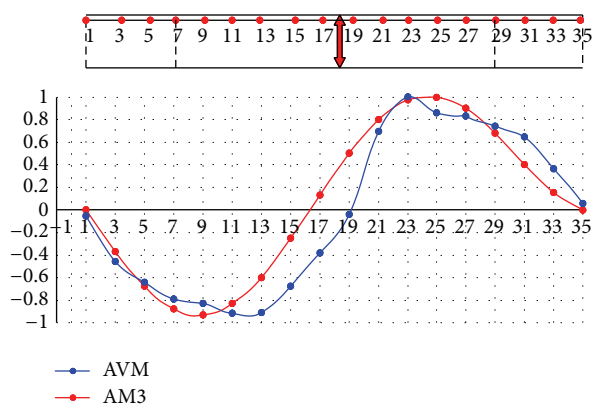


Figure 28: Experimentally and mathematically identified second transverse mode shapes.

height. Cables for supporting main prestressed beams are PV360 Lock Coil St 1570/1760 with diameter 60 mm (the first and the second cable) and PV640 Lock Coil St 1570/1760 with diameter 80 mm (the third and the fourth cable). Steel ties with counter weight concrete block consist of 20 rebar with diameter 32 mm on one tower. The quality of rebar steel is B500.

Arrangement of measuring points is shown in Figure 30. Ambient vibration test was made using 0.5 g Accelerometer Sensors. Operating frequency range of used accelerometer is 0.5 to 500 Hz, with acceleration range 0.5 g and resolution 0.00025 g. One accelerometer was used for the reference point (stationary point) 1, while the remaining 2 accelerometers moved to individual measuring points 2–22, according to the arrangement.

5.2. Ambient Vibration Measurements. Normal traffic flow was permitted during the testing procedure. The ambient vibration was simultaneously recorded for 120 s resulting in 491,520 data points per data set. In Figure 31 the results of the measurements, frequency spectrums are presented. The dominant frequencies of vibration of the superstructure measuring points are in range from 2.50 to 6.50 Hz.

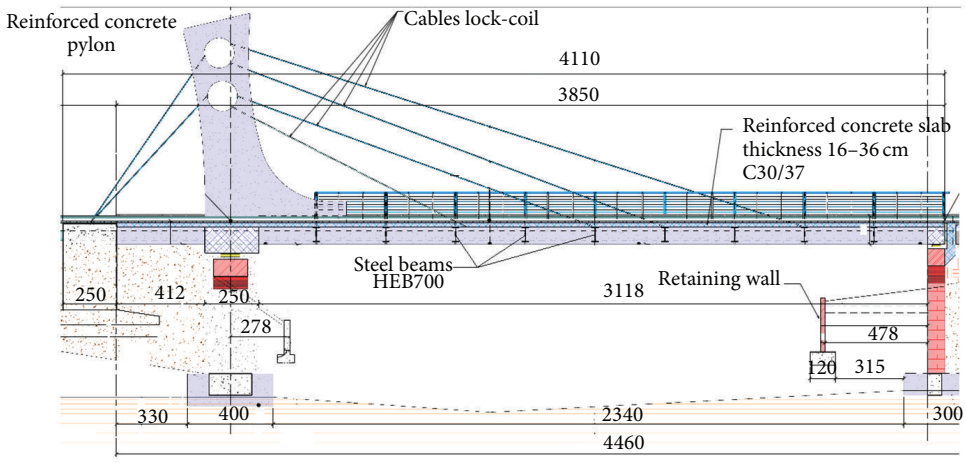
The EFDD technique is used for modal parameter extraction. Singular values of spectral density matrices and average of auto spectral densities attained from vibration signals using the EFDD are shown in Figure 32. The first and the second modes obtained using the EFDD are 2.719 Hz and 6.79 Hz (Figure 33).

5.3. Mathematical Models and Comparative Analysis of Experimental and Numerical Modal Parameters. The mathematical model AM4 was created using the finite elements (SAP2000N). The deck was modelled as a shell element. Main prestressed beams and steel I beams were modelled as an elastic beam elements and cables as a ties. Material property was modelled based on the design parameter, described in Section 5.1. The piers-soil contact was modelled as stiff support. In Figure 34 mathematically identified first and second mode shapes are presented.

The comparison was made for the first and the second mode. Experimentally and mathematically identified modal frequencies are presented in Table 8.

The AVM was performed a year after construction of the bridge. When the mathematically (AM4) and experimentally identified (AVM) modes are compared, it is seen that there is

(a)



(b)

Figure 29: Cable-stayed overpass in Tuzla: (a) view, (b) longitudinal cross section.

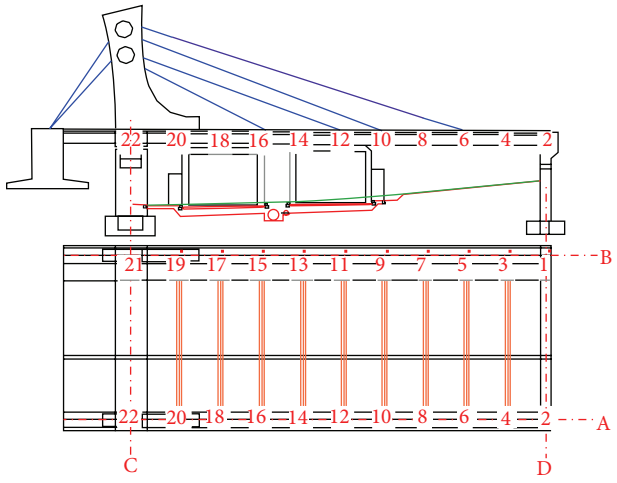


Figure 30: Measurements setup.

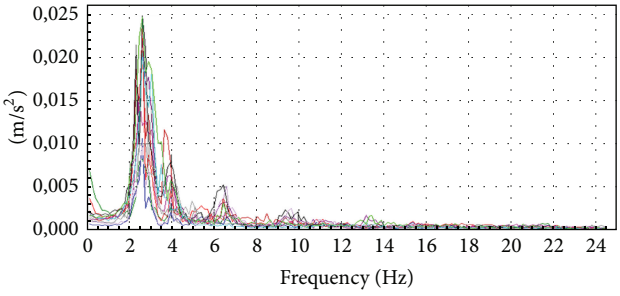


Figure 31: Frequency spectrum, ambient vibration.

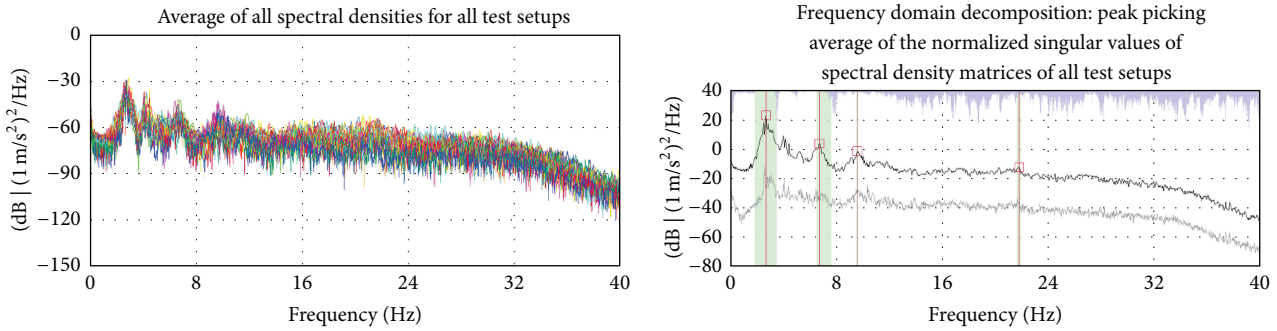
Table 8: Experimentally and mathematically identified modal frequencies (Hz).

Modes	AVM	AM4
First mode	2.719	2.727
Second mode	6.790	6.512

a very good agreement between modes (Figures 35 and 36). The difference of natural frequencies obtained by AVM and AM4 is 4%. Since the bridge is new, the mentioned value can be attributed to the geometry simplifications in modelling of the tower and the uncertainty in the designed material properties of structure elements (the uncertainty in stiffness of structure elements).

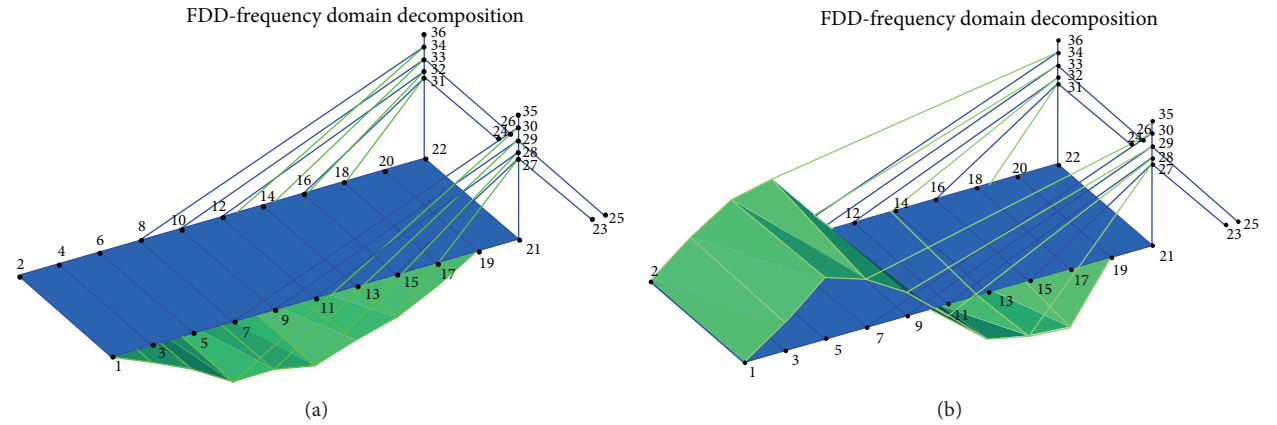
6. Conclusions

This paper deals with the measurements and interpretation of the results of ambient vibration tests done on the two older existing bridges in Sarajevo and Skopje and the newly erected cable stayed bridge in Tuzla. A series of ambient vibration tests were carried out on the bridges with the aim of demonstrating the usefulness of ambient vibration tests for



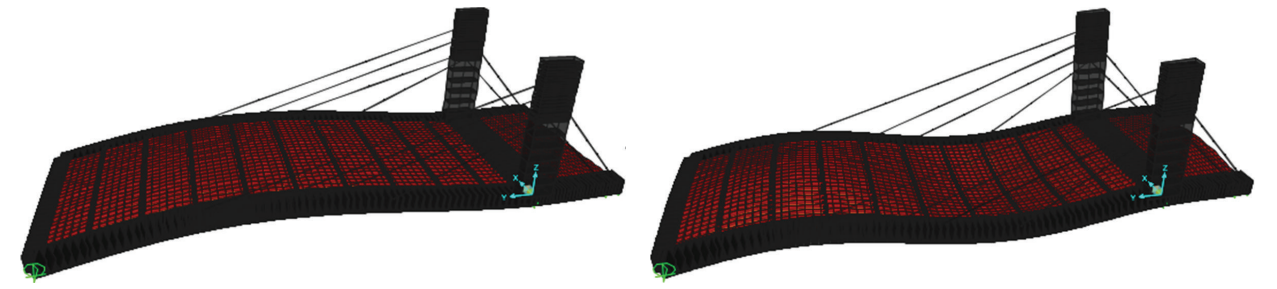
(a) (b)

FIGURE 32: Dynamic characteristics attained from the test using PP and SSI method: (a) average of auto spectral densities and (b) singular values of spectral density matrices.



(a) (b)

FIGURE 33: Vibration modes obtained by EFDD (Artemis): (a) 1st mode; (b) 2nd mode.



(a) (b)

FIGURE 34: Mathematically identified mode shapes: (a) 1st mode; (b) 2nd mode.

identification of the modal parameters of the tested structure. The processing of the recorded data was carried out using specific software Artemis in order to extract the dynamic characteristics of the bridge. The natural frequencies, mode shapes, and modal damping ratios were extracted from the structural responses. Mathematical models are proposed. The comparison of the natural frequencies and modes shapes provided by the AVM with their counterparts derived from the AMs was done. Generally, a good agreement was obtained for the computed and measured natural frequencies and mode shapes. The mathematical responses of the new bridge (Case Study 3) are quite close to those obtained experimentally. Some deviations are registered in Case Study 1 and Case Study 2, where the old bridges were analyzed.

Two proposals of mathematical models are analyzed in Case Study 1 with the aim of finding a model that has a response similar to the response structure, determined by measurement. Certain deviations of measurements were

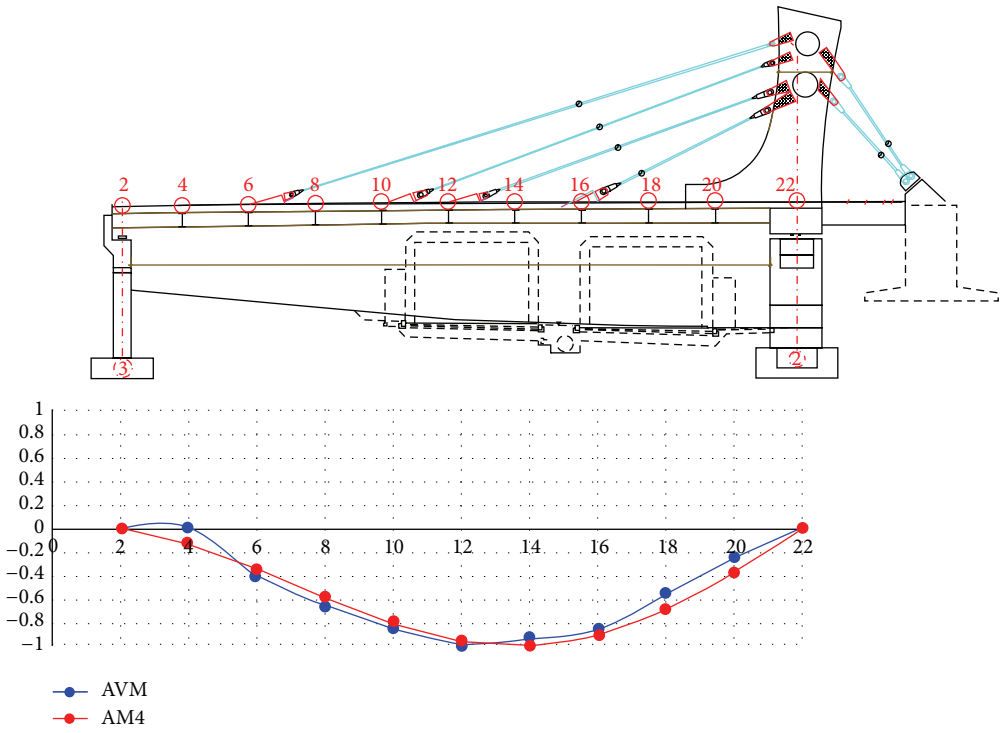


Figure 35: Experimentally and mathematically identified first mode shapes.

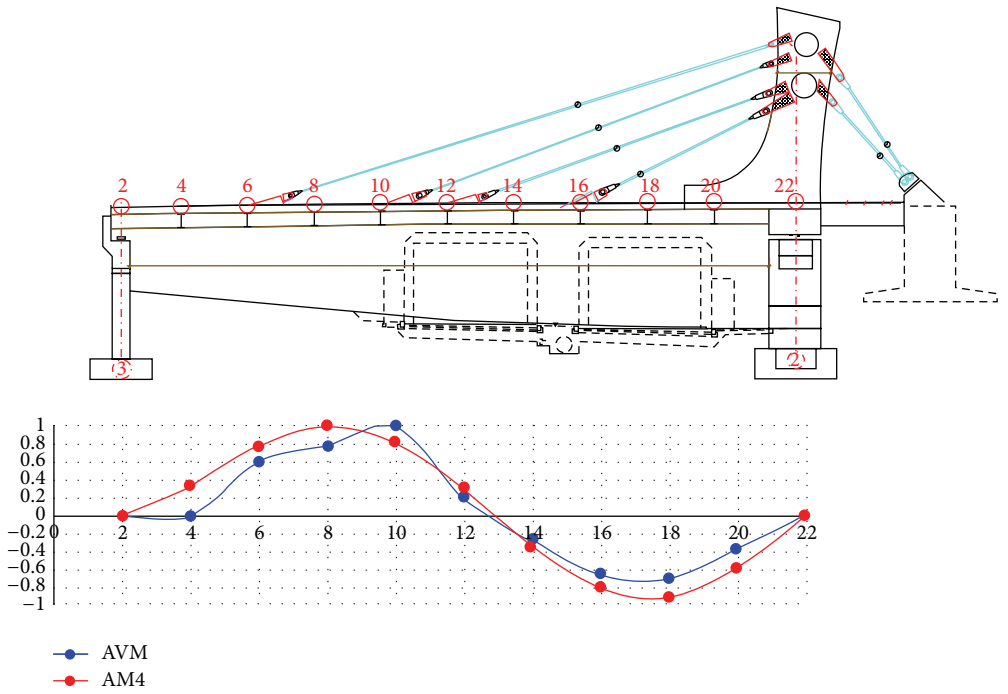


Figure 36: Experimentally and mathematically identified second mode shapes.

registered in both models. The reason can be attributed to the real nature of the boundary conditions and the uncertainty in the material properties of structure elements after 45-year service. Also, the differences between mathematical and experimental results can be attributed to the mathematical models assumptions, the low deformation (strain) levels, and the definition of the modulus of elasticity according to the code used, which is calculated at strains higher than the ones imposed by ambient vibrations, strengthening of concrete due to aging, and friction mechanisms as well as to

construction practice during concrete casting. The registered scattered values, between the computed and measured frequencies, could be partially due to unexpected channel errors.

Deviations in the second transverse tone are registered in Case Study 2. The reasons could be similar to the ones in Case Study 1.

The presented studies show that the signal analysis of ambient vibration records allows the determination of the dynamic characteristics of the bridge. In addition, the frequency and associated modes of vibration can be assessed with adequate mathematical model. The presented results clearly indicate the great potential that ambient vibration measurements hold for monitoring bridge structures. The data collected during the ambient vibration test, which only took some hours and very few resources, processed with adequate algorithms provided very useful information. The comparisons presented in case studies constitute a validation of the developed mathematical models and at the same time permit some fine tuning, especially concerning the boundary conditions and unexpected channel errors.

In particular, this paper clearly shows that it was possible to extract a lot of useful information from data collected during the ambient vibration test. For the exact definition of obtained deviations detailed investigation is necessary. In future, further research should be directed towards a new set of measurements, upgrading the mathematical models and assessment of unexpected channel errors.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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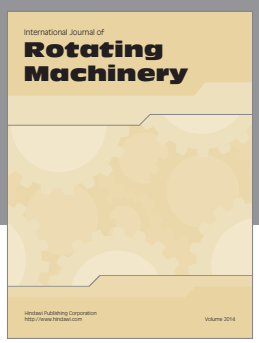

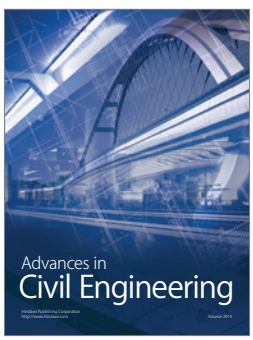

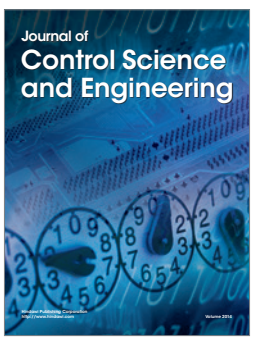

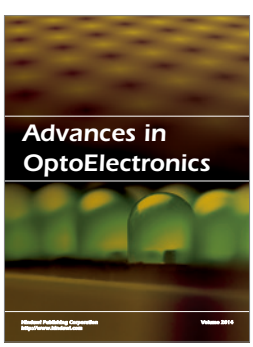

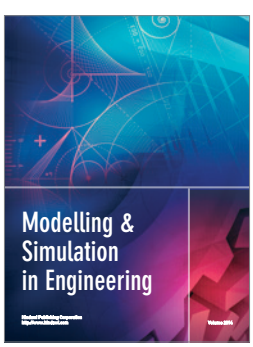

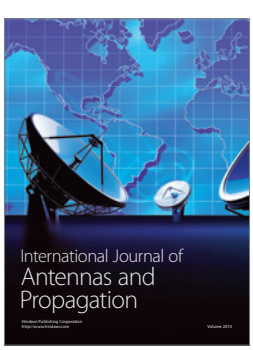

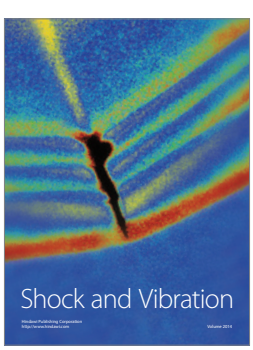

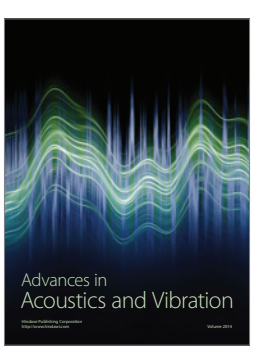

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