Damir Zenunovic[1]

SYSTEM IDENTIFICATION OF BRIDGES USING AMBIENT VIBRATION MEASUREMENTS AND NUMERICAL SIMULATIONS

Abstract: In the last ten years research with the aim of defining the real response of the bridge structures to environmental impacts was performed at the Department of Civil Engineering, University of Tuzla. The research included measurements of ambient vibration of bridge structures and the surrounding soil, data analysis, modeling of real behavior of bridge structures. The parts of the research related to identification of bridges real behavior under environmental impact were published in the papers [1] - [6], given in the reference list. Here is presented one case study of typical multi-span girder bridge in Bosnia and Herzegovina.

Key words: bridge structures, ambient vibration measurement, numerical model, real behavior

[1] Full Professor,University of Tuzla, Univerzitetska 2, 75000 Tuzla, Bosnia and Herzegovina, e-mail: damir.zenunovic@untz.ba, damir.zenunovic@gmail.com

1. INTRODUCTION

The backbone of any country's economy consists of its assets of constructed facilities, such as highways and bridges. Bridges are some of the most critical components of transportation infrastructure systems. For these structures, failure is defined as any interruption of pedestrian or vehicular traffic across or under them due to structural distress. Direct consequences of failure can range from injury to loss of life and property in the case of collapse, and indirect consequences such as disruptions to economic activities and reduced access to emergency facilities in the event of collapse or closure.

Bridges are large and expensive structures that are of great importance to our economy and society. They are often exposed to adverse environmental and weather conditions and because of that a maintaining of bridges is very important. Health monitoring and identification of structural modes of bridges is a major component of system maintenance. Monitoring of bridges is the recording of the actual behaviour of a complex structure, often modelled in a rather simple way thereby neglecting behaviour of the structure in three-dimensional space.

Major threats to bridges primarily consist of the aging of the structural elements, earthquake-induced shaking, and standing waves generated by windstorms. In order to ensure their reliability, and especially their stability and serviceability, it is important to analyze the bridge structure loaded by dynamic excitation. 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.

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. Therefore, the trend is the application of continuous monitoring of bridges by measuring of bridge structure kinematic parameters under ambient action. 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.

In ambient vibration testing, it does not require a external excitation of the structure. The structures response to ambient excitation records in large number of points. The system identification technologies were applied to determine and analyse the frequency response functions from measured signal data. The loading could be from environmental, vehicular or pedestrian traffic or any other service loading.

Full scale vibration testing of bridge structure (ambient vibration test) is very usefull to assess the overall integrity of the structure and understand the actual behavior of the structure. From the vibration data collected during the test are utilized to extract the modal parameters. These modal parameters are used to control and update numerical calculation methods and to validate theoretical models of structures. Mathematical models of real structures usually involve significant assumptions especially with regard to boundary conditions. Therefore, as the structural system becomes more complex and sophisticated, it becomes more difficult to understand its mechanisms. Therefore, to develop an appropriate model, this will give a good prediction of its dynamic responses. Comparison and updating of theoretical predictions with measured response will lead to a better understanding of the structure. A vibration measurement on full scale bridges serves to increase the data base in a form of case studies on different type bridges. This database can be utilizes for performance information on the complete similar structures and also gives useful data for future designs.

Here is presented one case study of multi-span girder bridges, which is typical structure in Bosnia and Herzegovina and region. On selected bridge measurements of ambient vibration have been made. Using software ARTEMIS real modes of bridge structure oscillation were determinated. Then numerical models, updated with the measured oscillation parameters, were made.

2. CASE STUDY – THE BRIDGE OVER RIVER BOSNIA IN SARAJEVO, BOSNIA AND HERZEGOVINA

2.1. Description of the bridge and measurements setup

The bridge over the river Bosnia, shown in Fig. 1 and 2, is a 45 years old bridge located on the route M05, section Lasva-Stup. Its overall length is 117m and comprises of two spans of 21m and three spans of 25m. Width of the bridge is 10.40m. Each span is built with four precast post-tensioned I beams of 1.30m height with recessed deck. The five spans of the deck are interconnected through a 16cm 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).

Fig.1. The bridge over river Bosnia, close to Sarajevo

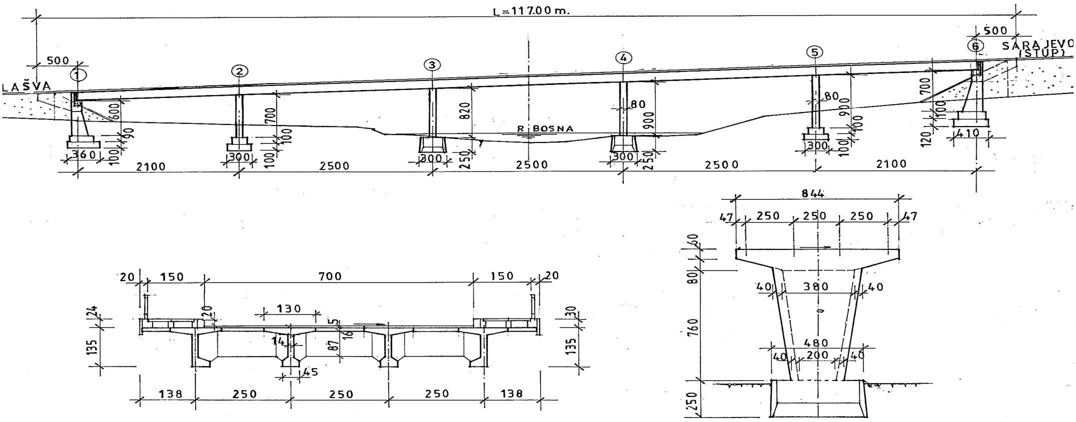


Fig.2. Elevation and cross-sections of the bridge

The bridge is instrumented for ambient vibration test. Arrangement of measuring points is shown in Fig. 3. Ambient vibration test was made with five three-axial geophone sensors (Fig. 4). One sensor was used for the reference point (stationary point) 6R, while the remaining 4 sensors moved to individual measuring points 1 – 17 according to the arrangement. Ambient vibration measurements were performed using Instantel Blastmate Pro4 vibration monitor with Sample Rate 1024 to 16384 S/s per channel. The three-axial geophone sensors used had 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. The recording system has start/common trigger capabilities to enable synchronized data acquisition. 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.

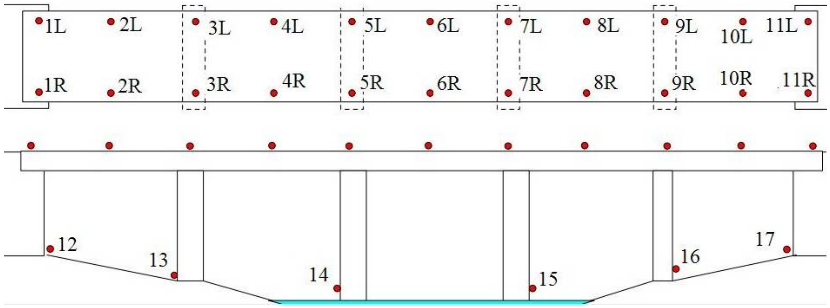


Fig.3. Measurements setup

Fig.4. Test equipment

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.

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.

2.2. Ambient vibration measurements

The sensors installed along the deck and at the bottom of the abutments and the piers recorded the bridge's response and the velocity time histories with 120 second recording time. Then frequency spectrums were obtained by using the FFT. In Fig. 5-10 results of M1 measurement is presented.

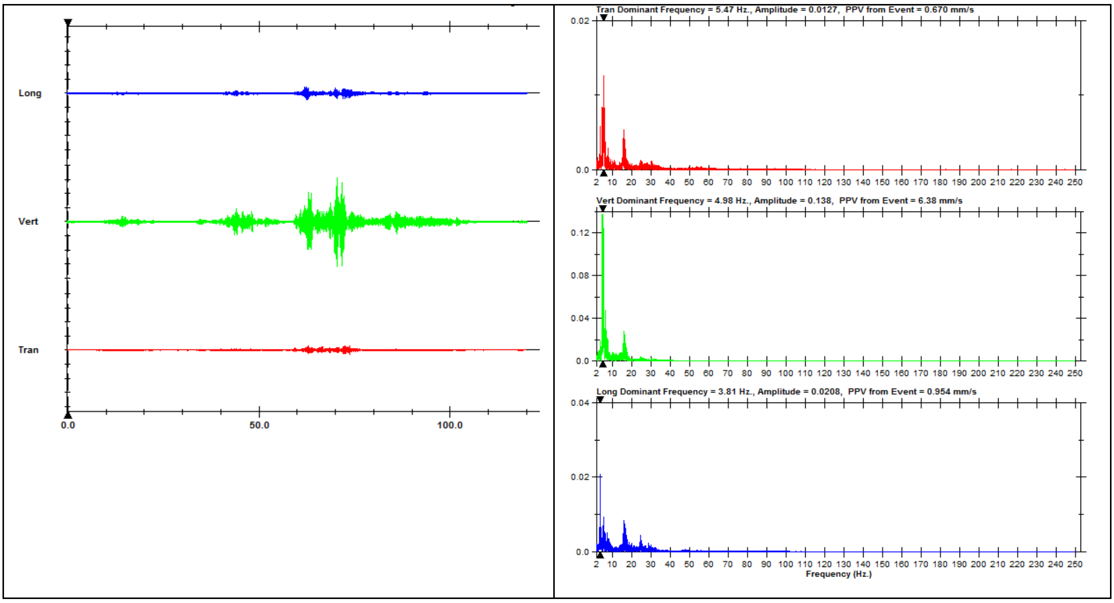


Fig.5. Measurement on base station 6R (middle of the span)

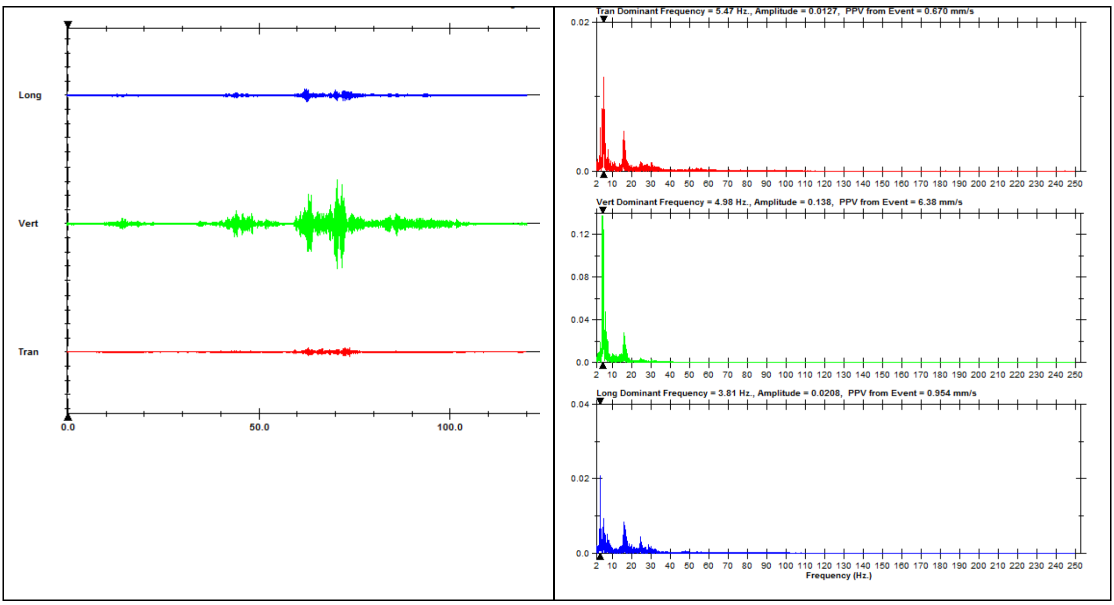


Fig.6. Measurement on moveable station 4R (middle of the span)

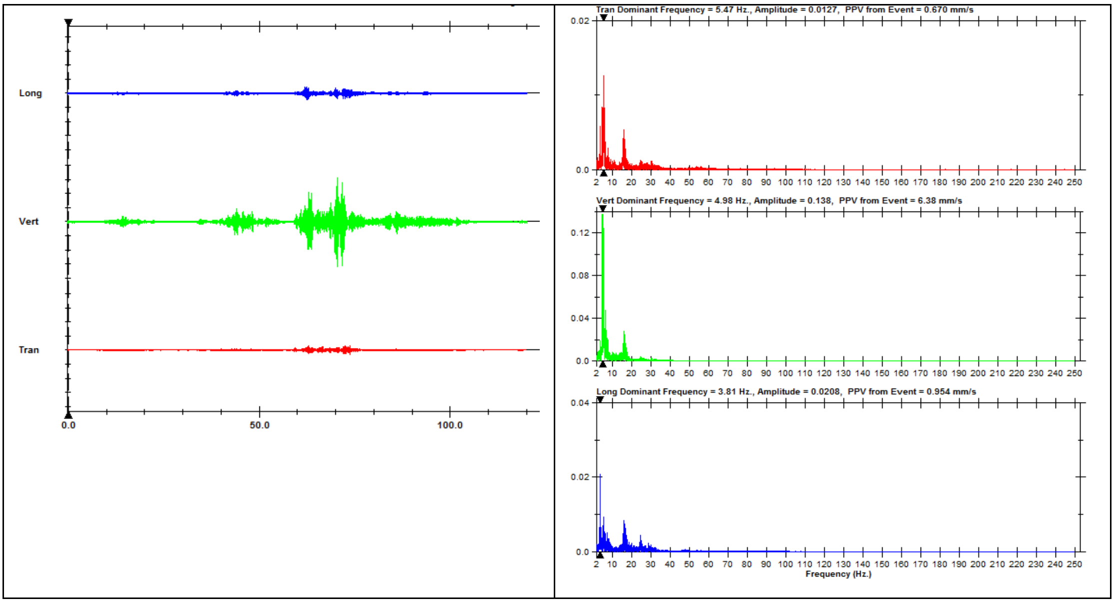


Fig.7. Measurement on moveable station 5R (over the support)

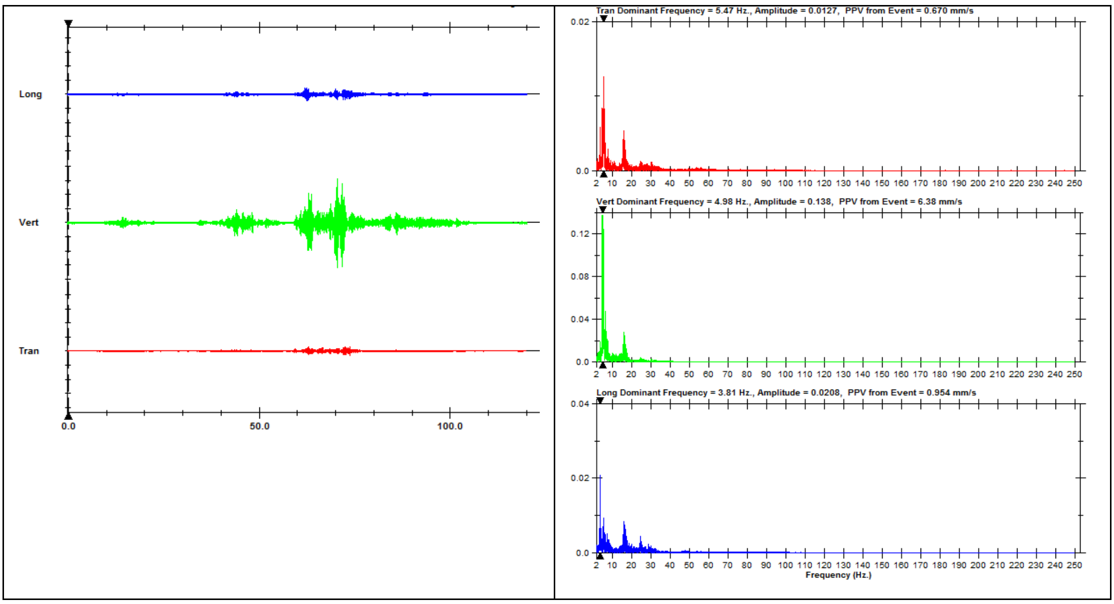


Fig.8. Measurement on moveable station 7R (over the support)

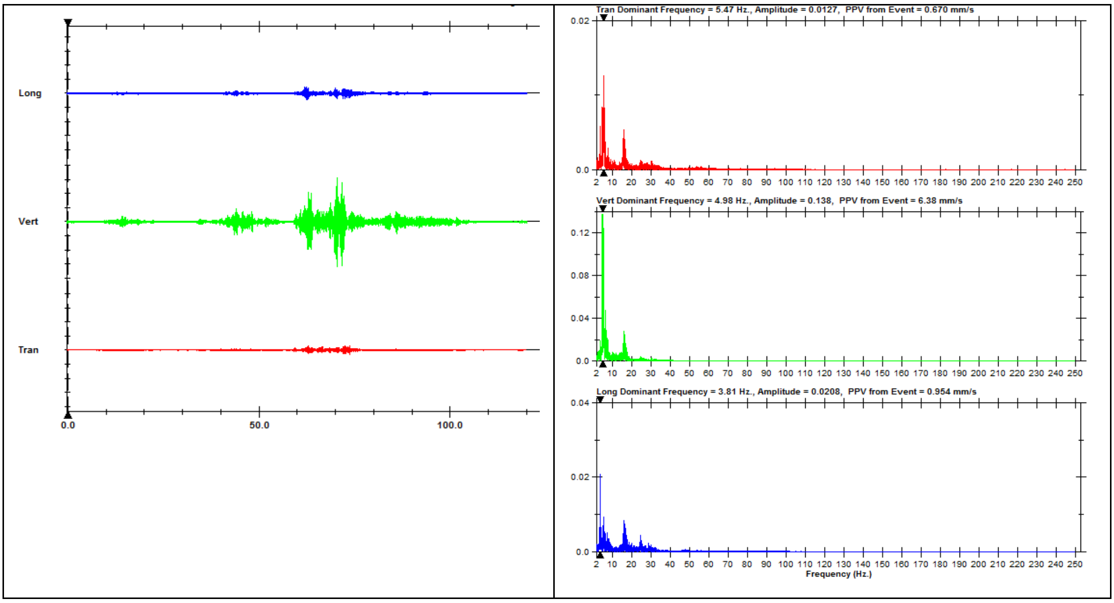


Fig.9. Measurement on moveable station 8R (middle of the span)

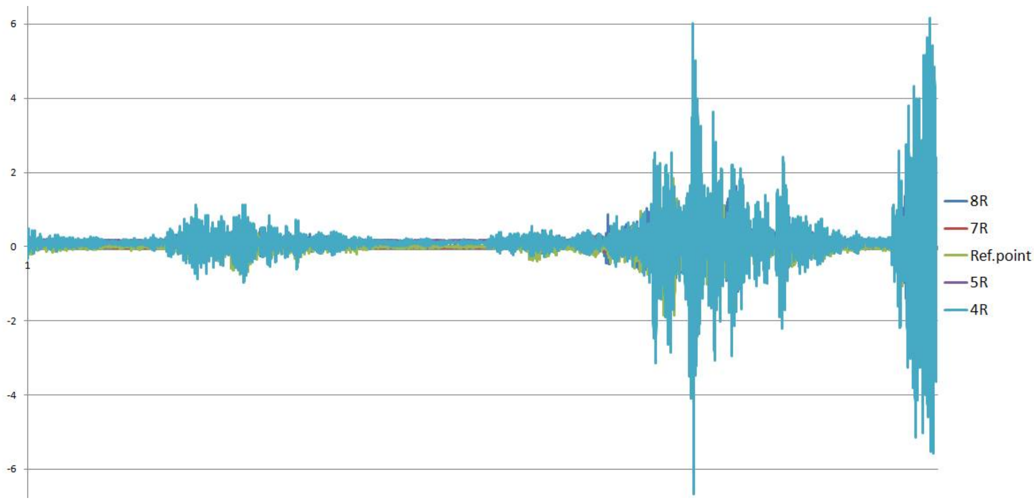


Fig.10. Comparison of vertical channels

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.

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

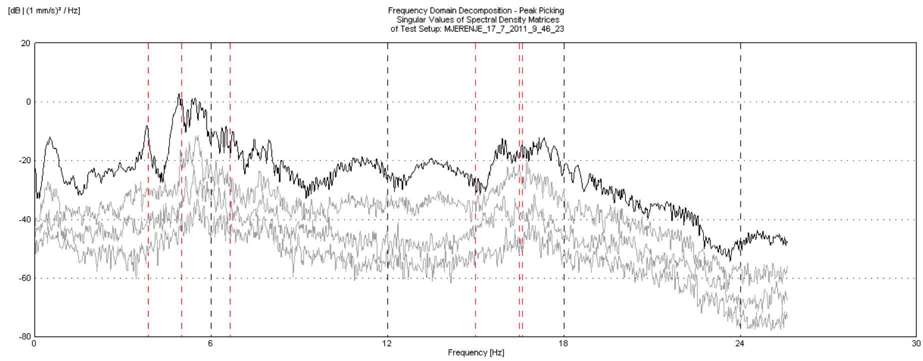
The computer program ARTeMIS Extractor was used to perform the modal identification of the structure. Two techniques were used to perform the modal identification: the Frequency Domain Decomposition (FDD) and the Stochastic Subspace Identification (SSI). The FDD technique consists on performing an approximate decomposition of the system response into a set of independent single degree of freedom (SDOF) systems for each mode. The singular values are estimates of the spectral density of the SDOF systems, and the singular vectors are estimates of the mode shapes.

The Enhanced Frequency Domain Decomposition (EFDD) feature offered by ARTeMIS adds a modal estimation layer to the FDD editor. When the FDD analysis is completed and the mode shapes are identified, the EFDD identifies the SDOF Bell functions and from these SDOF Spectral Bells, all modal parameters are estimated. The damping ratios can be obtained from the EFDD but not the FDD.

The SSI technique consists of fitting a parametric model to the raw times series data collected by the sensors. Using a specific representation of the transfer function, all the modal parameters are exposed. Therefore, the natural frequencies, damping ratios, and mode shapes can be extracted. The Unweighted Principal Component (UPC) algorithm was used to analyse the data.

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

a)



b)

c)

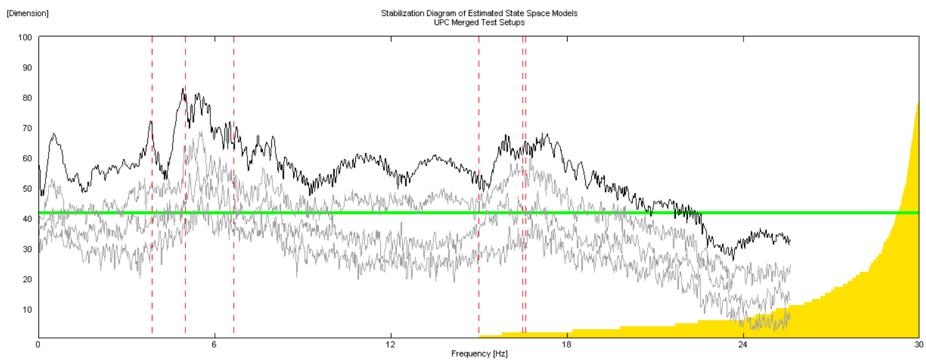


Fig.11. Dynamic characteristics attained from the test using EFDD and SSI methods: (a) singular values of spectral density matrices, (b) average of auto spectral densities, and (c) stabilization diagram of estimated state space model.

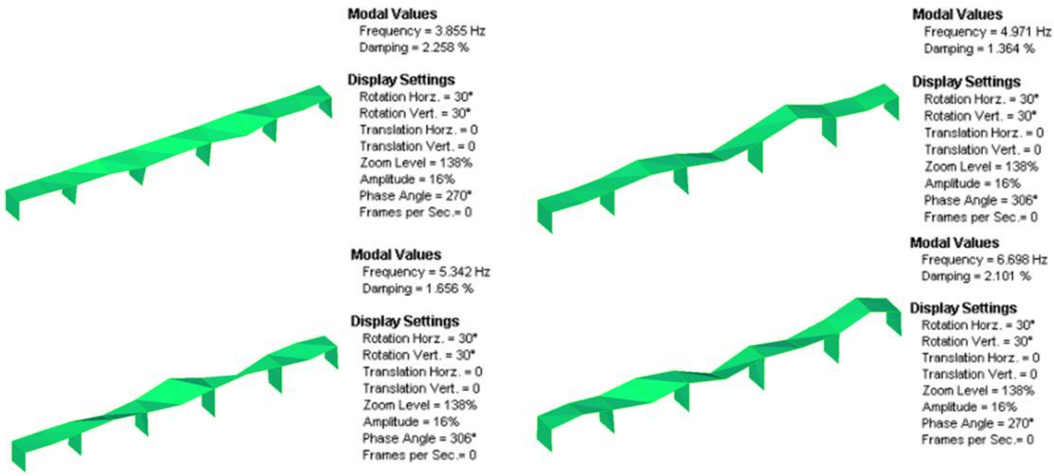


Fig.12. Experimental modal parameters attained from the test setup

2.3. Finite Element Models (FEMs) and comparative analysis of experimental and mathematical modal parameters

The FEMs were created using SAP2000N, based on the geometrical and material properties that were used for design, prediction of material properties after 45 years' service life and on the preliminary soil investigation. The first set of models (hereinafter called "S(1-6)-NB1-M1") is FEMs with different soil stiffness, designed neoprene bearings stiffness and designed strength of concrete, the second ("S(1-6)-NB1-M2") is FEMs with designed neoprene bearings stiffness and theoretically predicted strengthening of concrete during service life, and the third and the fourth ("S(1-6)-NB2-M2" and "S(1-6)-NB3-M2") are FEMs with increased neoprene bearings stiffness and theoretically predicted strengthening of concrete during service life.

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.

Dimensions of neoprene bearings are 300/400/100 mm. Neoprene bearings are modelled with vertical and horizontal springs. The stiffness of the vertical springs was calculated using Eq. (1):

$$k = \frac{EA}{h} \tag{1}$$

where E is the Young's modulus (E=630 N/mm^2), A is the cross sectional area, and h is the bearing height. The lateral shear capacity of bearings is controlled by the dynamic coefficient of friction between concrete and neoprene of 0,40.

Soil was modelled in two ways:

- Soil model with linear springs, used for the structure-soil interaction (S1-S4). Spring stiffness k = 15000 kN/m^2 has been selected based on experiences with similar class - C soils.
- Layered soil model with solid elements (S5 and S6). Modulus E, determined by preliminary geophysics soil measurements (PGSM), was used for the solid elements (details about soil measurements in papers [1] and [6])

The foundation structure was also modelled using solid elements so that the soil-foundation contact has been realized by identically arranged nodes of the solid soil elements and the foundation structure. Review of the models is presented in Tab. 3. The FEMs are shown in Fig. 13.

Table 3 - Review of the FEMs

S1-NB1-M1 S2-NB1-M1 S3-NB1-M1 S4-NB1-M1 S5-NB1-M1 S6-NB1-M1	S1-NB1-M2 S2-NB1-M2 S3-NB1-M2 S4-NB1-M2 S5-NB1-M2 S6-NB1-M2	S1-NB2-M2 S2-NB2-M2 S3-NB2-M2 S4-NB2-M2 S5-NB2-M2 S6-NB2-M2	S1-NB3-M2 S2-NB3-M2 S3-NB3-M2 S4-NB3-M2 S5-NB3-M2 S6-NB3-M2

SOIL	NEOPREN ELASTOMERIC BEARING	CONCRETE STRENGTH
S1- absolute stiff S2 – 10 x k (S4) S3 – 5 x k (S4) S4 – empirical stiffness of soil (k = 15000kN/m^2) has been selected based on the experience with similar soils S5 – layered soil (LS) with modulus E determined by (PGSM) S6 – LS with 10% of modulus E	NB1 – designed stiffness (DS) NB2 – 1.5 x DS NB3 - stiff	M1 – designed strength (fck = 30MPa, Ecm = 31GPa) M2 – theoretical strength after 45 years' service (Ec0 = 38,5GPa)

(a) *(b)*

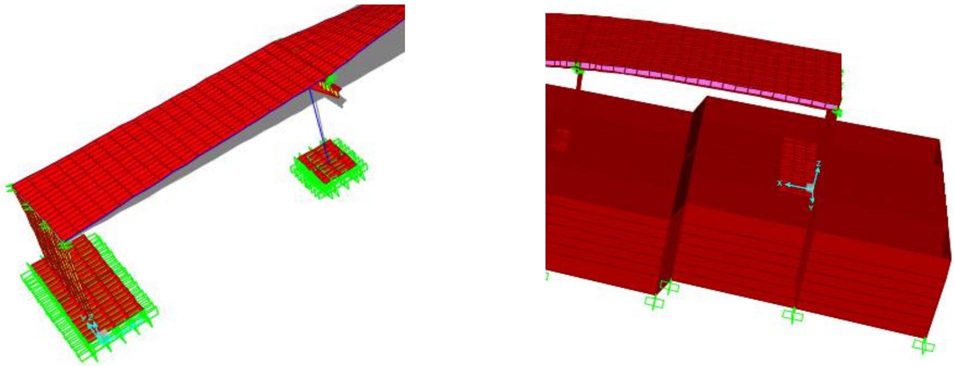


Fig.13. FEMs: (a) S1-S4; (b) S5 and S6

Since the scope of the research was to validate the modelling assumptions made and to identify their relative impact on the numerically predicted structural response, the developed FEMs were assessed comparatively. Through this procedure, longitudinal, transverse, bending and torsional modes were identified, whose modal frequencies are summarized in Table 4.

Table 4 - Modal frequencies

	LONGITUDINAL MODE	TRANSVERSE MODE	BENDING MODE	TORSIONAL MODE
Ambient Vibration Measurements	-	3.855	4.971	5.342
FEMs				
S1-NB1-M1	2.654 (2)	2.803 (3)	3.673 (4)	3.896 (6)
S2-NB1-M1	2.442 (1)	2.454 (2)	3.654 (3)	3.810 (6)
S3-NB1-M1	2.267 (2)	2.148 (1)	3.651 (4)	3.616 (3)
S4-NB1-M1	1.567 (2)	1.444 (1)	3.630 (4)	2.967 (3)
S5-NB1-M1	2.523 (2)	2.312 (1)	3.907 (6)	3.711 (5)
S6-NB1-M1	2.459 (2)	1.969 (1)	3.884 (6)	3.405 (4)
S1-NB1-M2	2.751 (1)	**3.641 (2)**	3.798 (5)	4.377 (6)
S2-NB1-M2	2.512 (1)	2.522 (2)	3.828 (3)	3.949 (6)
S3-NB1-M2	2.323 (2)	2.198 (1)	3.824 (4)	3.737 (3)
S4-NB1-M2	1.586 (2)	1.448 (1)	3.794 (6)	3.030 (3)
S5-NB1-M2	2.614 (2)	2.369 (1)	4.081 (6)	3.806 (5)
S6-NB1-M2	2.542 (2)	2.011 (1)	4.054 (6)	3.483 (3)
S1-NB2-M2	2.971 (1)	**3.731 (2)**	3.825 (3)	4.443 (6)
S2-NB2-M2	2.666 (2)	2.556 (1)	3.860 (3)	4.065 (6)

S3-NB2-M2	2.442 (2)	2.224 (1)	3.857 (3)	3.857 (4)
S4-NB2-M2	1.627 (2)	1.467 (1)	3.827 (6)	3.124 (3)
S5-NB2-M2	2.804 (2)	2.411 (1)	4.184 (6)	4.126 (5)
S6-NB2-M2	2.715 (2)	2.046 (1)	4.156 (6)	3.755 (4)
S1-NB3-M2	3.574 (1)	**4.246 (3) (fig.14a)**	4.136 (2)	**5.026 (5) (fig.14b)**
S2-NB3-M2	3.207 (2)	2.689 (1)	4.261 (3)	4.383 (4)
S3-NB3-M2	2.915 (2)	2.317 (1)	4.253 (4)	4.143 (3)
S4-NB3-M2	1.965 (2)	1.528 (1)	4.198 (4)	3.320 (3)
S5-NB3-M2	3.407 (2)	2.513 (1)	**4.639 (3)(fig.14c)**	4.847 (3)
S6-NB3-M2	3.239 (2)	2.146 (1)	4.609 (4)	4.318 (3)

(n) marks in parenthesis denote modes of certain models

The first comparison was made between modal frequencies of the model S2-NB1-M1 and ambient vibration modal frequencies. From the presented results, it is evident that S2-NB1-M1 model fails to predict well the measured responses as they exhibit large deviations from the identified modal frequencies that exceed 57% in the transverse direction, 36% for the bending mode and 40% for the torsional mode. In general, it is observed that the modes measured via ambient vibrations are on average 44% higher than those predicted by the model S2-NB1-M1.The real structure is identified as significantly stiffer than predicted using the S2-NB1-M1model.

A second comparison was made between the models with different soil stiffness (S1-S6) in order to quantify the importance of soil compliance on the predicted dynamic characteristics of the structure. The refined consideration of soil flexibility leads to significant lower values of longitudinal, transversal and torsional modal frequencies while the reduction of bending modal frequencies is not significant. Through comparison of the S2-NB1-M1 model and the S3-NB1-M1 model, the longitudinal mode is found 8% more flexible, the transverse mode 14% more flexible and the torsional mode 5%. The reduction of bending modal frequencies is not significant.

Since the FEMs used were refined as much as possible, the model induced uncertainty can be deemed as relatively low. As a result, the deviations between the identified and numerically predicted modal frequencies can be attributed primarily to the uncertainty in the material properties, which seem to be a key parameter for the reliable estimate of the dynamic characteristics of the structure. In order to improve the convergence, sequential parametric analysis was conducted. The idea was to gradually modify specific structural parameters through a step-by-step parametric analysis scheme, until a nearly optimal fit was achieved.

The results of this parametric analysis resulted in the combination of updated structural parameters, with different neoprene bearings stiffness and concrete strength of structure elements, summarized in Table 4.

The results presented in Table 4 show the improvement of the modal frequencies predicted by the models S1-NB2-M2, S1-NB3-M2 and S5-NB3-M2 compared to the modal frequencies predicted by the S2-NB1-M1 model. By comparing the modal frequencies predicted by the mentioned models, it is clear that the Young Modulus of Elasticity for the bearings, the deck and the piers had to be significantly increased compared to the values assumed in the initial design. This can be clearly attributed to the low deformation (strain) levels that are developed under ambient vibrations at which the bearing stiffness is significantly higher than that assumed during design. Also, the effect of concrete strengthening during service life has to be considered.

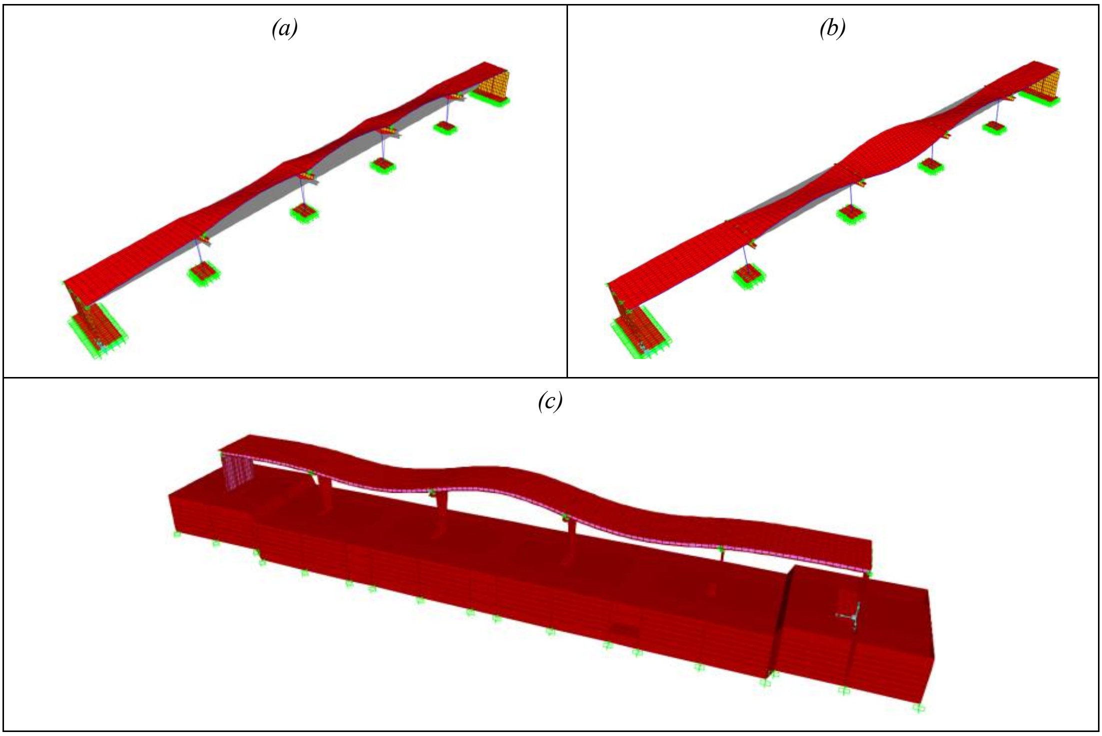


Fig.14. Numerically predicted modes: (a) Transverse mode of S1-NB3-M2 model; (b) Torsional mode of S1-NB3-M2 model; (c) Bending mode of S5-NB3-M2 model

For comparison of the modal parameters identified by ambient vibration tests and mathematical models, two FEMs (named AM1 and AM2) were selected. AM1 is selected model S1-NB1-M2 and AM2 is S5-NB3-M2. 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.

Table 5 - Experimentally and mathematically identified modal frequencies (Hz)

	FIRST TRANSVERSE MODE	FIRST VERTICAL MODE
AMBIENT VIBRATION TEST	3.855	4.971
ANALYTICAL MODEL AM1	3.641	3.798
ANALYTICAL MODEL AM2	2.513	4.639

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 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 15). The deviations between the experimentally identified and mathematically predicted first transverse mode shapes (Figure 16) 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 in further research. 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.

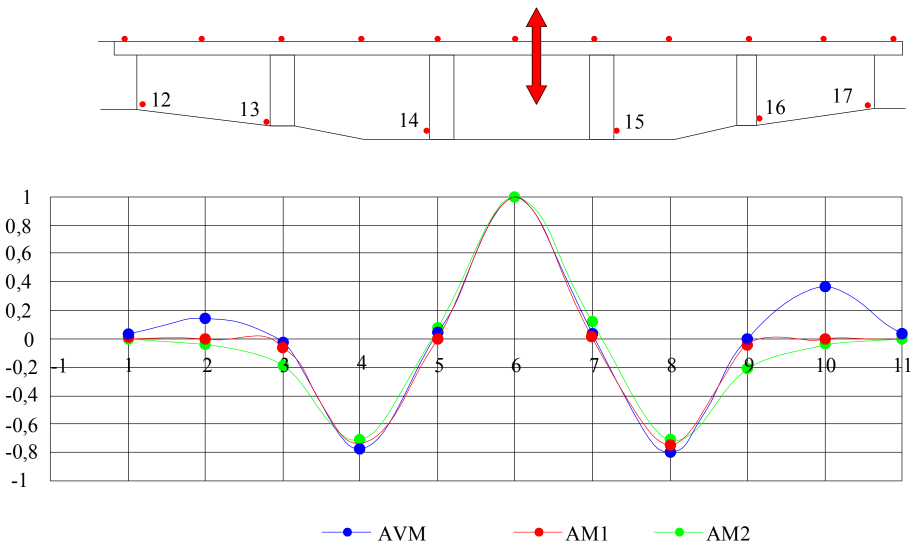


Fig.15. Experimentally and mathematically identified first vertical mode shapes

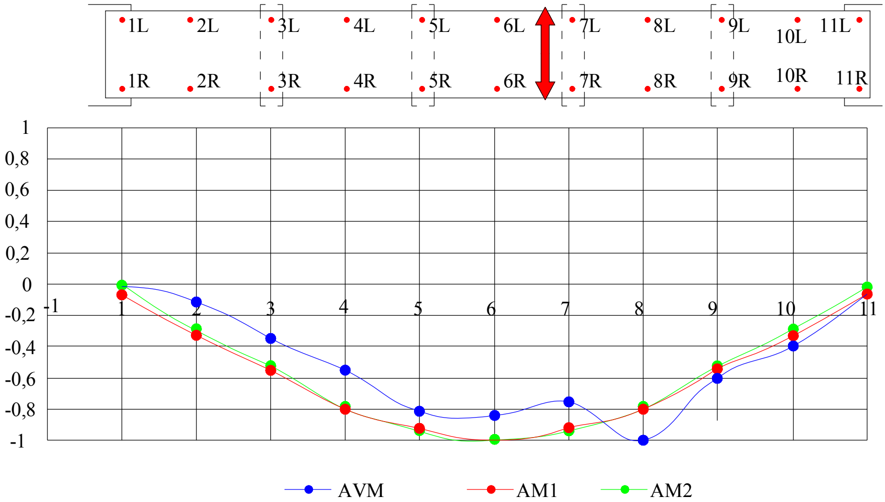


Fig.16. Experimentally and mathematically identified first transverse mode shapes

3. CONCLUSIONS

In the lecture was presented the measurements and interpretation of the results of ambient vibration tests done on the existing bridge in Sarajevo, Bosnia and Herzegovina. A series of ambient vibration tests were carried out on the bridge with the aim of demonstrating the usefulness of ambient vibration tests for 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. Using FEMs with various levels of complexity and modelling refinement in terms of consideration of the soil parameters and structure material parameters, the modal frequencies of the bridge are computed and compared with the ones identified using ambient vibrations. 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. Some deviations are registered. Two proposals of mathematical models are analyzed with the aim of finding a model that has a response similar to the response structure, determined by measurement. Certain deviations of measurements were 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.

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 study 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 lecture 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.

4. REFERENCES

[1] Zenunović D., Topalović M., Folić R. (2015): System identification of RC girder bridges based on field measurements and numerical simulations, *Technical Gazette*, Vol.22, No.3, doi:10.17559/TV-20140603143241, Elsevier Ltd., pp.667-675.

[2] Zenunović D., Topalović M., Folić R. (2015): Identification of Modal Parameters of Bridges Using Ambient Vibration Measurements, *Shock and Vibration*, Volume 2015, Article ID 957841, 21 pages, http://dx.doi.org/10.1155/2015/957841

[3] Zenunović D., Husejnagić E., Folić R., Topalović M. (2012): Analysis of influential parameter for modelling of real behavior of bridge, *Proceedings of the 12th International Scientific Meeting INDIS*, Novi Sad, pp. 344-352.

[4] Zenunović D., Topalović M., Folić R. (2013): Assessment of Cable-Stayed Pedestrian and Motorway Overpass, *The Eight International Conference „Bridge in Danube Basin“*, Springer Vieweg, Wiesbaden, pp. 117-127.

[5] Topalović M., Zenunović D., Folić R. (2014): Seismic response analysis of multi-span concrete girder bridges, *Proceedings of the Fourth International Conference „Earthquake Engineering and Engineering Seismology“,* Bor, Serbia, pp. 203-210.

[6] Zenunović D., Folić R., Topalović M., Husejnagić E. (2015): Analysis of site effects on bridge structures by ambient vibration measurements, *Proceedings of the 13th International Scientific Conference INDIS*, Novi Sad, Serbia, Paper No. 52.

REVIEW QUESTIONS

1. Which parameters of structure can be determined by ambient vibration measurements (AVM)?
2. What are the constraints (limits) of AVM?
3. How many measurement points do you need to measure at the same time?
4. Which measured parameters can be input data for software Artemis?
5. How can the realistic behavior of a structure be assessed on the basis of results of AVM?