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MONITORING BASED STRUCTURAL IDENTIFICATION OF TWO RAILWAY BRIDGES

Data-driven tools applied on measured structural responses enable extraction of valuable information on the behavior of in-service structures and identification of the actual system in its "natural habitat".

An output-only testing campaign on two railway bridges is in the focus of this study. The specifics of the dynamic behavior of the structures are studied under three different scenarios. More precisely, the measured accelerations at relevant points of the structures, for unloaded and loaded conditions, are herein employed with the following objectives:

I) Structural dynamics identification via estimation of the first natural bending frequency of the structures during ambient unloaded conditions using the AutoRegressive Moving Average (ARMA) method and the time-domain Subspace System Identification (SSI) method

II) Assessment of traffic safety aspects based on the maximal amplitudes of vibrations during loaded operating conditions of the structure, for two separate train induced vibrations (train velocity of 40 km/h and 60 km/h)

III) Evaluation of the dynamic effects through a dynamic amplification factor estimated from the recorded responses during dynamically loaded structure (train velocity of 40 km/h and 60 km/h).

By utilization of the response-based data analysis methods the first natural frequencies of the steel structures are successfully identified and compared with FEM numerical results. The acceleration data is further assessed by preprocessing filtering techniques and extracted features are assessed according to EN 1990:2002 or UIC (International Union of Railways) code 776-2 (2009).

Keywords: Data-driven, railway bridge, structural identification, structural health monitoring, ambient vibration.

1. INTRODUCTION

Testing programs can deliver a significant input in the management of important civil infrastructures and reduce the multiple sources of uncertainty and variability typical for assessment of real engineering systems. Outlined by Brownjohn in [1], the cases where civil infrastructure monitoring may be required are: (i) modifications to an existing structure, (ii) monitoring of structures affected by external works, (iii) monitoring during demolition, (iv) structures subject to long-term movement or degradation of materials, (v) feedback loop to improve future design based on experience, (vi) fatigue assessment, (vii) novel systems of construction, (viii) assessment of post-earthquake structural integrity, (ix) decline in construction and growth in maintenance needs, and (x) the move towards performance-based design philosophy.

Since the earliest documented systematic bridge testing in the 1940s, monitoring programs for bridge infrastructure have historically been utilized for the purpose of understanding the load–structure–response chain, which eventually brings improvements and calibration of numerical models [1]. Nowadays, remote control and data retrieving hardware advancements, as well as software solutions for data-driven analysis drive the evolution of traditional bridge monitoring programs towards more holistic strategies based on output-only structural health monitoring (SHM) systems [2].

The focus of this study is to explore data records attained from a short-term output-only testing strategy utilized on two short-span bridge structures. The purpose is to obtain an initial structural identification of the systems' dynamics, data-based estimation of dynamic amplification factors, as well as vibration amplitudes of the bridge decks. [3].

2. TESTING AND DATA ANALYSIS

A short-term acceleration measuring system was installed on 2 railway bridges located in Kosovo. A schematic overview of the testing strategy is presented in Fig. 1. Summary on relevant bridge data, overview of the sensor placements and available monitored responses are provided in Figures 2 and 3.

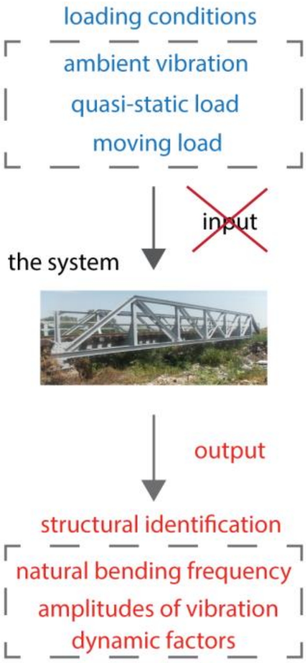


Figure 1. Schematic overview of the short-term SHM strategy.

Acceleration time histories are recorded by three tri-axial accelerometers for ambient vibration (Digitexx D110-T sensor) placed at three measurement points along the span of the structures. The acceleration responses were recorded for the following three scenarios: i) ambient vibration; ii) quasi-static load: a train passing with crawl speed of 5km/h; iii) moving load: a train passing with 40 km/h and 60 km/h.

Figure 2. Photographs of the structures under study.

2.1 VERTICAL BENDING FREQUENCIES

An initial step is to validate finite element results for the modal properties of the structures. To this end, first vertical bending frequencies of the systems were identified from the collected acceleration data.

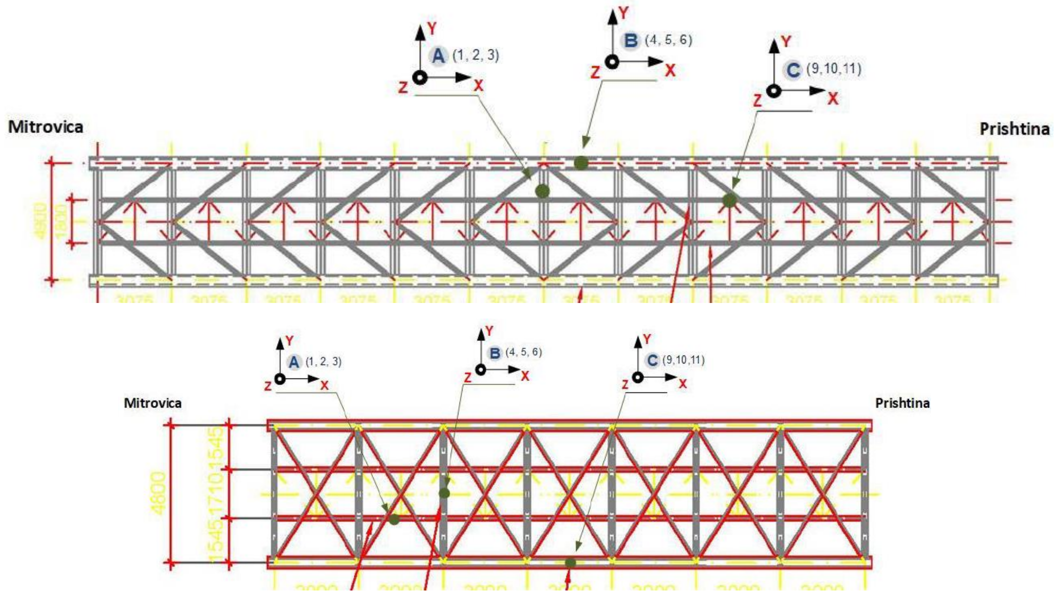
More precisely, the recorded data set lengths, approximately 10 min long and originally sampled with 200 Hz, were primarily preprocessed: detrended, downsampled (from 200 Hz to 50 Hz) and filtered with a lowpass filter (cutoff frequency of 25 Hz). The first vertical bending frequency of the bridges is then identified by employment of the time-domain parametric methods: ARMA method, an approach less sensitive to coupled modes, noise and short data set lengths, and the time-domain SSI method. For further details on the theory behind the methods and their limitations the interested reader is referred to [4].

The stabilization plots and estimated first natural frequencies for the vertical direction are presented in Figures 4 and 5, for the two bridge structures. As an additional criterion for eliminating computational and/or non-structural modes are the estimated modal damping values (accepted values less than 10%). Marked with red arrow at the same plots is an identified harmonic.

2.2 VIBRATION AMPLITUDES

Excessive vibrations of a given bridge deck correspond to measured levels of accelerations of the bridge deck. A traffic safety requirement for the prevention of track instability is based on the verification of the maximum acceleration of the bridge for the two loaded cases.

According to EN 1990:2002 or UIC code 776-2 (2009) as far as operating safety is concerned, the deck acceleration should be considered a serviceability limit state. In the case of bridges with slab tracks, the acceleration limit value is set at 0,5g for filtered low pass frequencies below 30 Hz. The measured acceleration responses and maximal amplitudes for the passing train with velocity of 40 km/h and 60 km/h are presented in Figures 6 to 9, for each velocity and bridge structure separately.



Bridge	Span [m]	1st FEM bending Frequency [Hz]	Sensors (unloaded case)	Sensors (loaded case)
B15	36.9	8.0	A,B,C	A,B,C
B05	21.0	9.8	A,B,C	A,B,C

Figure 3. Summary on relevant data and overview of the sensor placements.

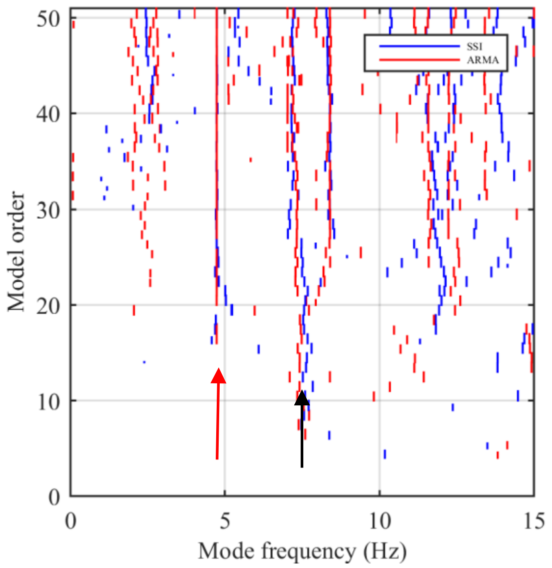


Figure 4. Stabilization plot for ARMA and SSI method for Bridge B15
(red arrow is identified harmonic,
black arrow is identified structural frequency)

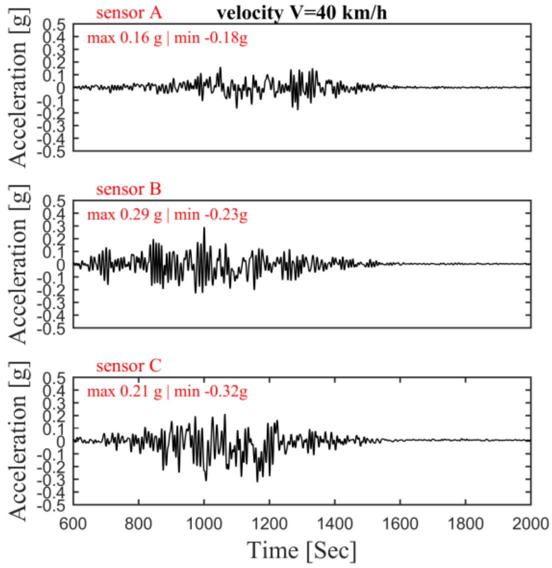


Figure 6. Time history plot of vertical accelerations and maximal amplitudes for velocity 40km/h for Bridge B15

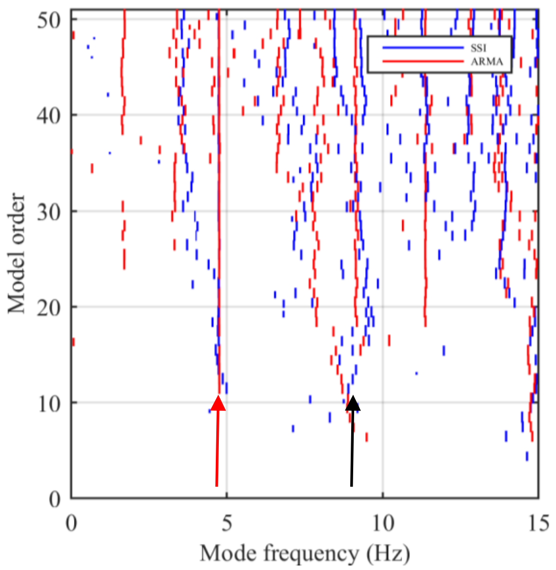


Figure 5. Stabilization plot for ARMA and SSI method for Bridge B05
(red arrow is identified harmonic,
black arrow is identified structural frequency)

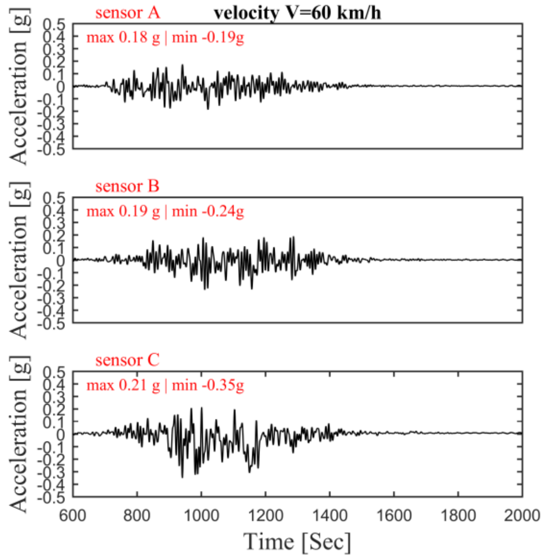


Figure 7. Time history plot of vertical accelerations and maximal amplitudes for velocity 60km/h for Bridge B15

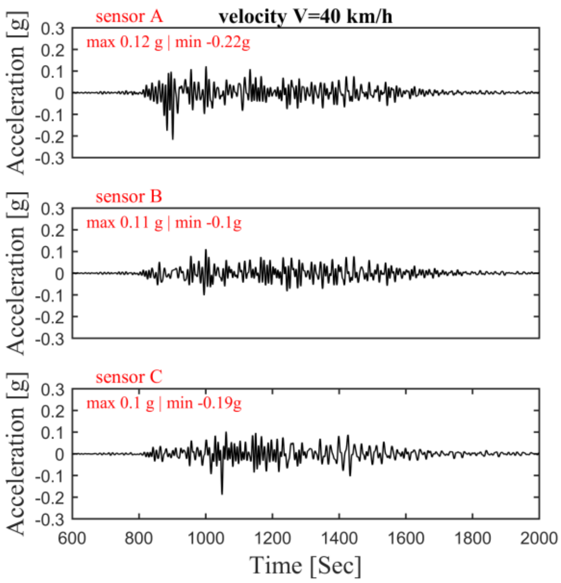


Figure 8. Time history plot of vertical accelerations and maximal amplitudes for velocity 40km/h for Bridge B05

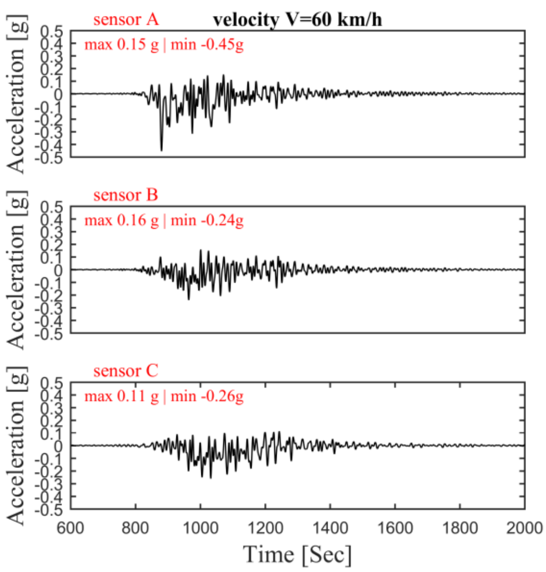


Figure 9. Time history plot of vertical accelerations and maximal amplitudes for velocity 60km/h for Bridge B05

2.3 DYNAMIC AMPLIFICATION FACTOR

In order to estimate the Dynamic Amplification Factor (DAF), the recorded vertical acceleration time histories corresponding to the three setups, namely the responses from the testing with crawl speed, train velocity of 40km/h and 60 km/h were preliminary filtered with a low-pass digital filter (cutoff frequency of 25 Hz). Then the DAF is calculated as the ratio of the maximum dynamic response for the corresponding velocity (40km/h and 60 km/h) and the maximum response for the crawl speed testing scenario.

The assessment results for the records of sensors placed at the most affected area of the bridge, namely sensor A for bridge B15, and sensor C for bridge B05 are herein presented for DAF estimation corresponding to velocity 60 km/h, Figures 10 and 11.

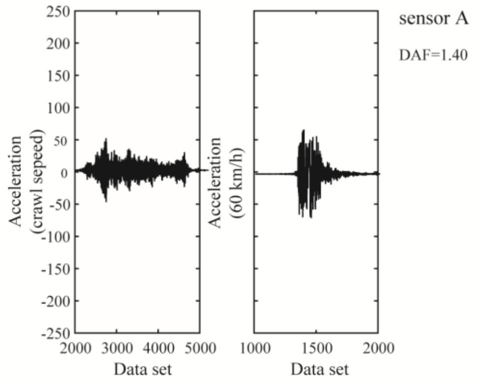


Figure 10. Signals for DAF estimation, velocity 60km/h, Bridge B15

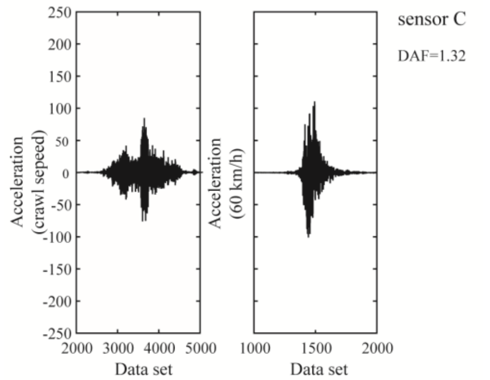


Figure 11. Signals for DAF estimation, velocity 60km/h, Bridge B05

3. CONCLUSIONS

The extracted features from recorded acceleration response data for unloaded and loaded operating conditions of the 2 bridges are summarized in Fig. 12 and Tab.2. With the purpose of verification, the FEM calculated natural frequencies (provided by Obermeyer Hellas Ltd.) are presented as well. The experimentally estimated first natural frequencies have a good agreement with the numerical results. The maximum acceleration amplitudes are within the serviceability limit case threshold value (EN 1990:2002) of 0.5g.

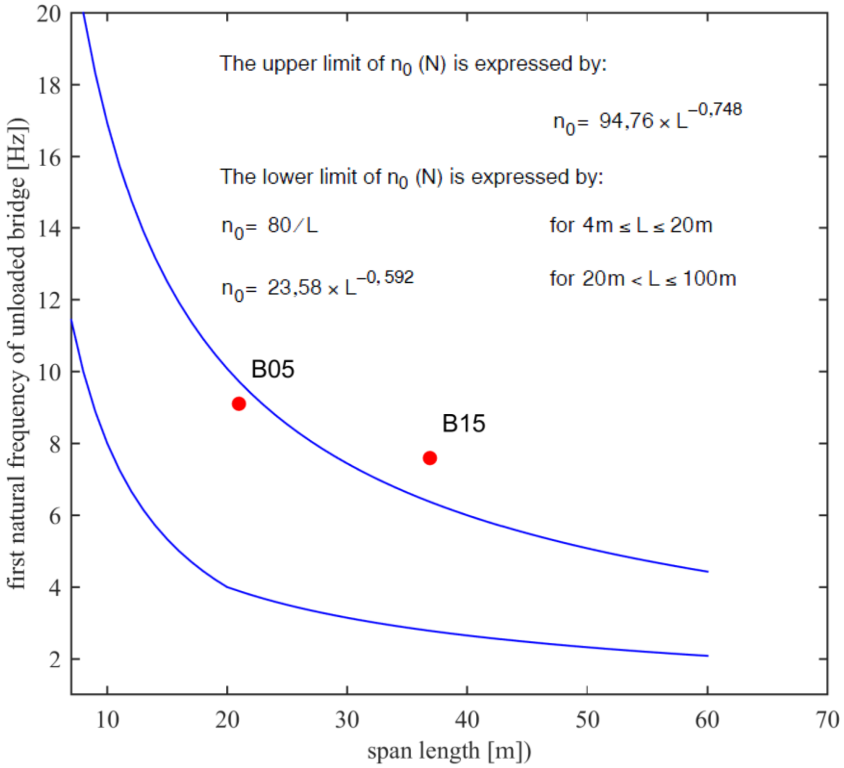


Figure 12. Limits of natural frequencies in relation to span length (UIC code 776-2, 2009)

Table 2. Summary of estimated parameters from the measured structural responses

Bridge	Experimental modes [Hz]	Numerical modes [Hz]	DAF 40 km/h	DAF 60 km/h	Max vertical acceleration 40 km/h	Max vertical acceleration 60 km/h
B15	7.6	8.0	1.16 ; 1.07	1.40 ; 1.37	0.32g	0.35g
B05	9.11	9.8	1.00	1.32	0.22g	0.45g

The necessity for a subsequent dynamic analysis can be further assessed by estimating the domain N (UIC code 776-2, 2009). Dynamic analysis is necessary for speed of lines below 200km/h and first natural frequency above the upper limit of the predefined range N. In Fig. 12 the limits of domain N for the experimentally estimated natural frequencies in [Hz] as a function of the span length [m] are presented. Bridge B15 is above the upper limit of N.

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