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Experimental Dynamic Analysis of the Railway Bridge Near Žatec

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Abstract. The paper describes the experimental dynamic analysis of the railway bridge near Žatec in the Czech Republic. The main aim of this analysis was to verify the compliance between measured quantities and calculated ones determined on the finite element model (FEM) for improvement of the FE model. The analysis was divided into two parts, modal analysis and dynamic load test. The used measurement system with dynamic range 160 dB enabled to measure the response of the bridge to train passages and also ambient vibration of the bridge between the passages without changing the setting of the measurement system.

Introduction

Steel bridges are an important part of bridge structures in the Czech Republic, especially on the railway network. Many of them are older than one hundred years and they are still in operation. Despite a certain degree of their degradation, the quick replacement of large number of bridges is unrealistic. Thus it is necessary to prove their building state and bearing capacity. For that assessment many factors must be considered, e.g. the stiffness of joints, spatial behaviour of the structure, the influence of corrosion [1] and fatigue life.

The current computational methods are highly advanced and allow to perform intensive computational analysis. Nevertheless, the input parameters for these models and their aptness can be obtained in many cases only by properly and efficiently designed experiments [2], [3] and [4]. The use of the test results for validation of the computational model can help to refine it enough that unsatisfactory bridge structure becomes at least limited satisfactory.

Description of the bridge

The studied railway bridge consists of two independent structures K01 and K02 (Fig. 1) and its total length is 116 m. Both bridge independent structures are identical, they are 58 meters long and consist of a riveted steel structure with truss main girders with an intermediate bridge deck (Fig. 2). The height of the main girders is 3.53 m The pillars and abutments are made of stone masonry (Fig. 1 and Fig. 2).

The railway track is transferred across the bridge in a straight and partially in a transition curve. The width of the clearance profile 2.5 m is kept only at niches on the pillar. On the rest of the bridge, the width of the clearance profile does not even reach 2.2 m.

Fig. 1: The view on the whole bridge

Fig. 2: The view on the first investigated half of the bridge

Measurement system

The sensors and measurement stations were prepared to positions in the pauses between regular train passages, which were approximately two hours.

The vibrations of the bridge were measured by uniaxial piezoelectric acceleration transducers 8344 Brüel&Kjær, which were placed in five cross sections, one position on the left upper chord and one position on the right upper chord in each cross section (Fig. 3 – red dots). The measurement of vibration was realized in vertical direction (Fig. 3 – from Az 041 to Az162) and in horizontal direction perpendicular to the longitudinal axes of the bridge (Fig. 3 – from Ay041 to Ay162). The transducers were mounted to the steel structure using magnets. They were put to all points on the upper chords at once, thus the response was measured in these points simultaneously.

The modal analysis was supplemented by measurement at next 10 points at contacts of the cross members of the deck with the main girders (Fig. 3). For the modal analysis, the Ambient Vibration Testing (AVT) system was used. It means, the measurements were realized between train passages by measuring only responses to ambient forces (wind, microseismicity, etc.).

The response of the bridge to regular train passages were also measured. The passages of trains with known characteristics were used for the dynamic load test.

Two interconnected measurement stations Front-end 3560-B-120 and 3050-B-040 Brüel&Kjær were used for data acquisition and the data analysis was done using the software PULSE and MS Excel. The evaluation of the natural frequencies and modes of vibration of the footbridge were done in the software MEScopeVES of the company Vibrant Technology.

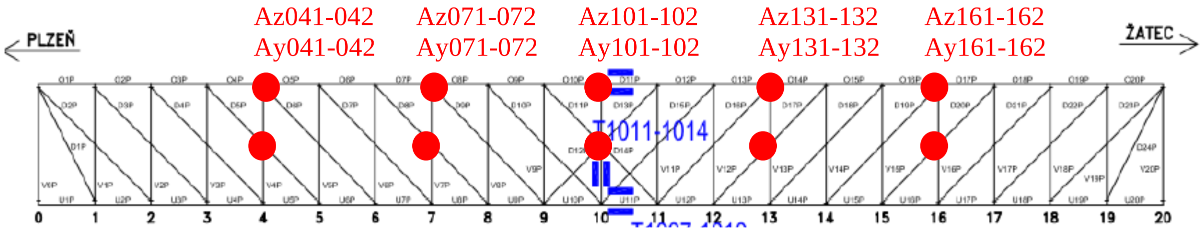

Fig. 3: The measured points marked in the longitudinal section of the railway bridge

Results of the dynamic analysis

The nine natural frequencies were evaluated in the frequency range 1.0 to 11.0 Hz during the experimental modal analysis of the investigated bridge [5]. The natural frequencies, which were compared with the theoretical results, are mentioned in Table 1.

Table 1: Comparison of the measured and calculated natural frequencies

Natural frequencies calculated		Natural frequencies measured		Frequency difference	Limit deviation	Character of the mode shape
No.: (j)	$f_{(j)}$ [Hz]	No.: (j)	$f_{(j)}$ [Hz]	$\Delta_{(j)}$ [%]	$\Delta_{(j)}$ [%]	
(1)	1.58	(1)	1.67	-5.7	+10 ; -15	1st horizontal bending mode shape
(2)	3.75	(2)	3.84	-2.4	+10 ; -15	2nd horizontal bending mode shape
(3)	3.87	(3)	3.91	-1.0	+10 ; -15	1st vertical bending mode shape
(4)	4.46	(4)	4.45	0.2	+/-15.2	1st torsional mode shape
(5)	6.10		x		+/-15.6	1st longitudinal mode shape
(6)	6.31	(5)	6.26	0.8	+/-15.6	3rd horizontal bending mode shape
(7)	8.66	(6)	8.58	0.9	+/-16.2	4th horizontal bending mode shape
(8)	9.73	(7)	9.80	-0.7	+/-16.5	2nd torsional mode shape
(9)	10.28	(8)	10.09	1.8	+/-16.7	2nd vertical bending mode shape
(10)	10.66	(9)	10.66	0.0	+/-16.8	mode shape of the upper flanges of the main girders

Notice: x – According to the theoretical calculation, the bridge vibrates in its longitudinal direction (X axes of the bridge) in the 5th natural mode shape. In this direction, the vibration was not measured.

The mode shapes corresponding to the natural frequencies were evaluated too and they were visually compared with the theoretical ones calculated on the FE model. The bridge was modelled in 3D with the use of beam elements. The real section properties were used. The truss joints and bearings were modelled as semi-rigid, according to the real stiffness. The material data for the assessment were used from the coupon tests. The FE model is described in more detail in [6]. The comparison is shown in Figures 4 - 12. In the figures, there are plan views (upper one) and elevations (lower one) of the theoretical mode shapes (on the left side) and experimental mode shapes (on the right side) of natural vibration.

The frequency differences meet all limits prescribed in the standard ČSN 736209 (see Table 1 – Limit deviations), the corresponding theoretical and experimental mode shapes have the same number of node lines at the same positions.

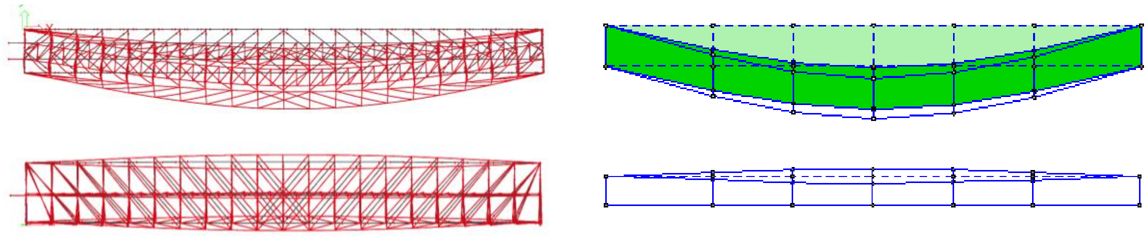

Fig. 4: Comparison of the 1st theoretical mode shape (left) of natural vibration with the 1st experimental one (right).

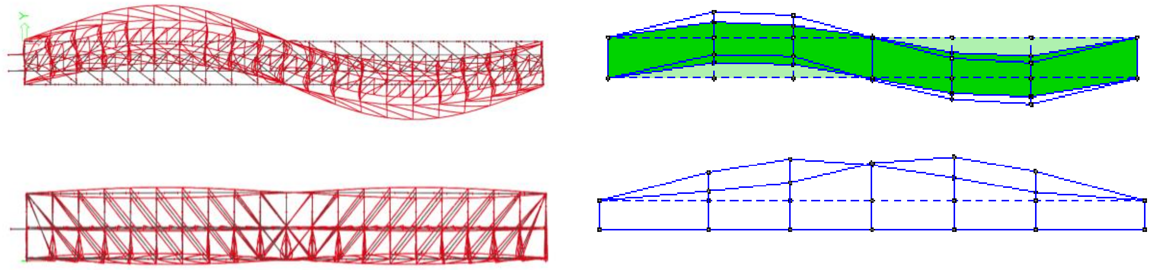

Fig. 5: Comparison of the 2nd theoretical mode shape (left) of natural vibration with the 2nd experimental one (right)

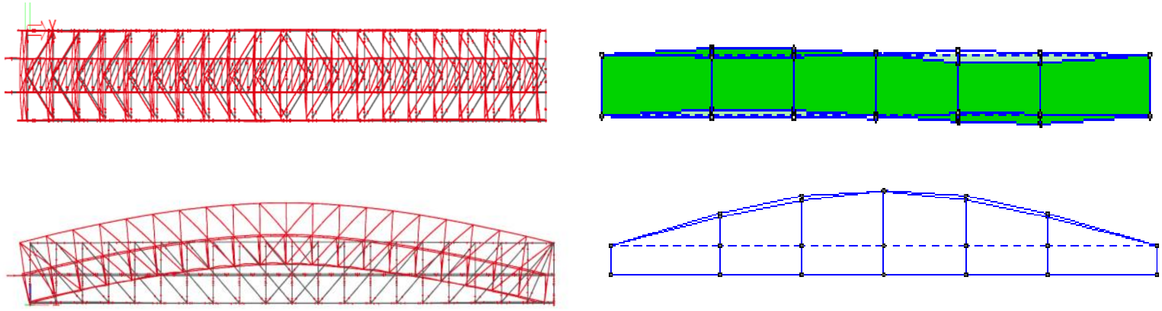

Fig. 6: Comparison of the 3rd theoretical mode shape (left) of natural vibration with the 3rd experimental one (right)

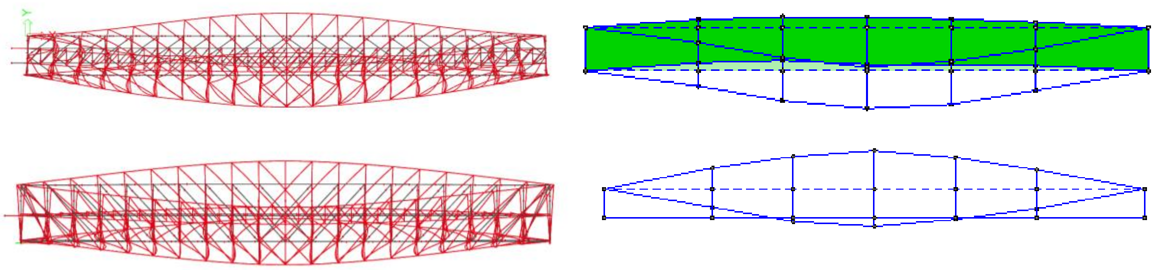

Fig. 7: Comparison of the 4th theoretical mode shape (left) of natural vibration with the 4th experimental one (right)

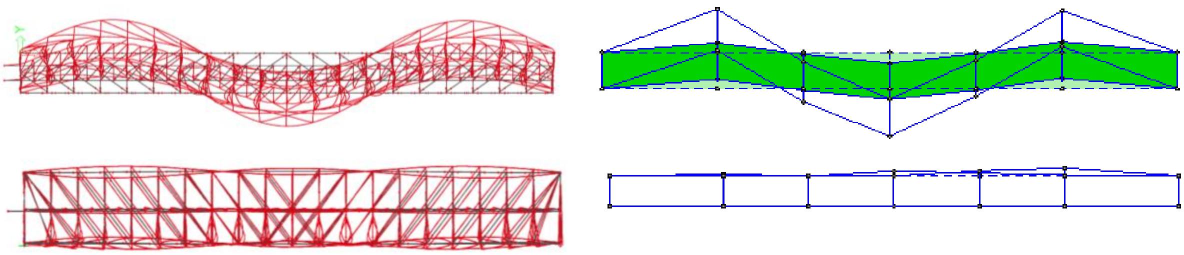

Fig. 8: Comparison of the 6th theoretical mode shape (left) of natural vibration with the 5th experimental one (right)

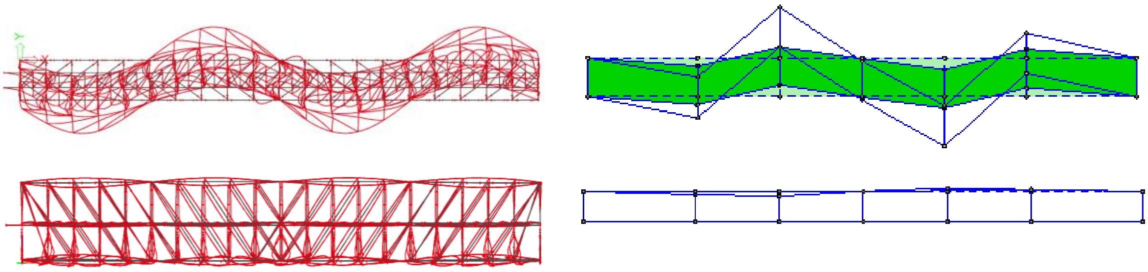

Fig. 9: Comparison of the 7th theoretical mode shape (left) of natural vibration with the 6th experimental one (right)

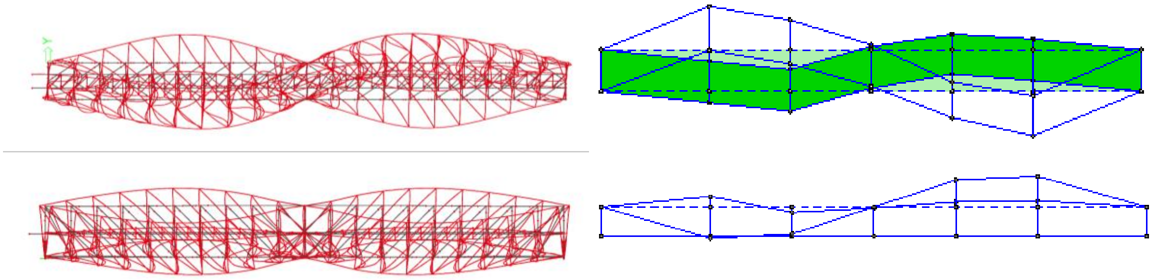

Fig. 10: Comparison of the 8th theoretical mode shape (left) of natural vibration with the 7th experimental one (right)

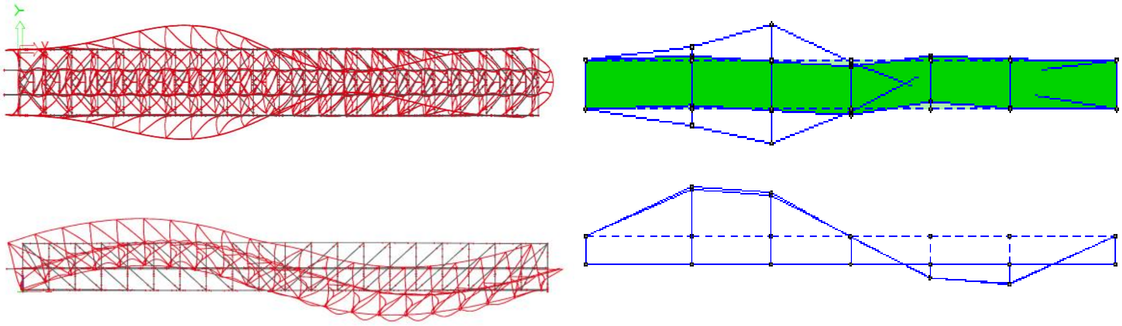

Fig. 11: Comparison of the 9th theoretical mode shape (left) of natural vibration with the 8th experimental one (right)

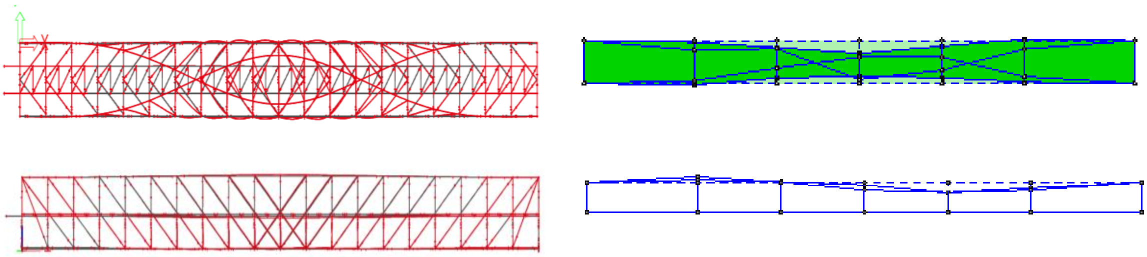

Fig. 12: Comparison of the 10th theoretical mode shape (left) of natural vibration with the 9th experimental one (right)

From all the recorded vibrations of the monitored railway bridge caused by the train passages, the extreme amplitudes of the measured accelerations were evaluated, which are clearly summarized in the Table 2.

All acquired records of acceleration were additionally adjusted by filtration. A low-pass Butterworth filter with cut-off frequency set to 30 Hz was used.

Table 2: Extreme amplitudes of vibration in the middle of the monitored span of the bridge

Train crossing the bridge	Extreme amplitudes of vibration					
	Az - 101		Az - 102		Ay - 102	
	Min.	Max.	Min.	Max.	Min.	Max.
	[m/s^2]	[m/s^2]	[m/s^2]	[m/s^2]	[m/s^2]	[m/s^2]
Passenger train	-0.57	0.59	-0.55	0.55	-0.82	0.88
Passenger train	-0.43	0.41	-0.36	0.51	-0.52	0.67
Passenger train	-0.49	0.38	-0.38	0.50	-0.71	0.73
Passenger train	-0.60	0.51	-0.49	0.45	-0.69	0.70
Goods train	-1.01	0.78	-0.76	0.82	-1.05	1.42
Passenger train	-0.46	0.46	-0.63	0.44	-0.66	0.71
Passenger train	-0.70	0.68	-0.46	0.50	-0.68	0.84

Conclusions

The experimental dynamic analysis of the railway bridge near Žatec in the Czech Republic is presented in the paper. The simplified modal analysis using AVT system which was carried out on this bridge only in five cross-sections was good enough to measure and reliably identify nine natural frequencies and natural modes of vibration of the bridge. Based on comparison between calculated and measured characteristics the FE model was modified. The agreement between finally calculated and measured modal characteristics was good and met all limits prescribed in the standard ČSN 736209 (Table 1). The validated model of the bridge can be used for apt static and dynamic calculation of the bridge in its real immediate state.

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