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Assessment of structural condition of steel bridge in Brandýs nad Orlicí

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Abstract. This paper deals with structural condition assessment of historical steel through arch bridge in Brandýs nad Orlicí. Structure was dynamically tested to obtain mode shapes and eigenfrequencies on non-loaded structure and to estimate logarithmic damping decrement after traffic loading. Material properties were verified. Calculation model was created to validate its statical and dynamical behaviour. Article focuses on comparison of calculated results and properties of real structure upon excitation.

1 Description of a bridge

Bridge was built (according to bridge control protocol from 2014 [1]) between 1905 and 1929. Superstructure consists of single span through arch bridge made out of riveted steel sections. Compressed upper part of the arch is strutted using diagonal struts in outer profile of the cross section. Base of an arch is connected by steel tie. Original deck was rebuilt in 2014 using modern rolled "I" beams with steel ribbed plate. Structure is stiffened by longitudinal and transversal diaphragms. Main arches with ties are simply supported using cast iron bearings. Longitudinal deck beams are resting on abutments, which are made out of stone blocks.

Fig. 1. Tested bridge over Orlice river.

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1.1 Description of bridge surroundings

Bridge is located in active industrial area with nearby railroad and a weir. These surroundings are affecting the bridge itself and the dynamic response of the structure in idle non-loaded state is fairly measurable, while train passage had a significant effect on a structure.

2 Dynamic load testing

Structure was tested in two states [2]. Dynamic response was recorded in idle non-loaded state, where the dynamic response of the structure is caused by natural and technical seismicity without the need of using vibration exciter. For this state the operational modal analysis was performed. In second state the bridge was opened for traffic and dynamic response due vehicular and pedestrian loading was recorded to estimate logarithmic damping decrement.

Fig. 2. Example of sensor assessment during modal analysis.

Dynamic behaviour was measured using 8 piezoelectric acceleration sensors in vertical and horizontal direction. Sensor’s placement is shown in Fig. 3.

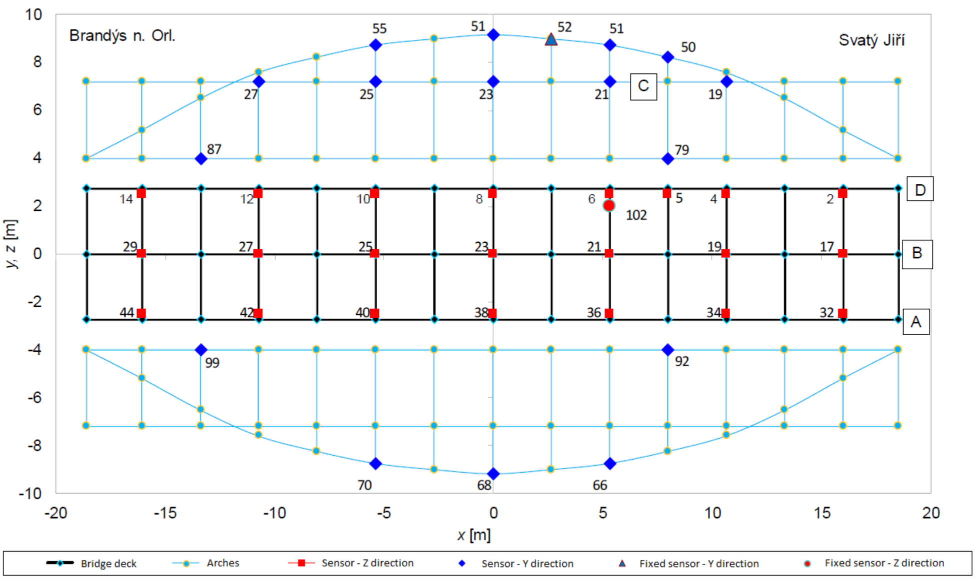

Fig. 3. Sensor's placement along the structure.

2.1 Results

In Table 1, the results of operational modal analysis are shown. Certain doubts exist for the values in brackets. These might be caused by chosen method of measurement or the nonstationarity of external conditions, or overall condition of a bridge and hidden failures. These doubts were aroused by anomalies in corresponding mode shapes.

Table 1. Measured eigenfrequencies.

Mode shape	Frequency [Hz]	Damping [%]
1	5.31	0.90
2	5.99	0.52
3	6.43	0.61
4	(7.57)	0.49
5	7.69	0.38
6	(8.11)	0.10
7	(8.25)	0.50
8	8.43	0.79
9	8.80	0.67
10	11.32	0.16
11	11.77	0.29

Maximal peaks of acceleration in vertical direction recorded during vehicular loading were ± 0.4 m/s^2 for cars and ± 1.0 m/s^2 for bus. Acceleration in horizontal direction was recorded as ± 0.6 m/s^2 for car and ± 1.5 m/s^2 for bus. Logarithmic damping decrement was calculated as $\delta = 0.05$.

2.1.1 Idle non-loaded state

Bridge permanently vibrates with low acceleration amplitudes. Cause of this behaviour is technical and natural seismicity of the surroundings. Eigenfrequnecies can be found in acceleration spectra (Fig. 4) as sharp peaks. These frequencies are systematically lower than frequencies calculated in FEA model. Both results are various around 8.0 Hz area, where significant peaks are missing in measurement. Vertical lines in upper part are representing calculated eigenfrequencies.

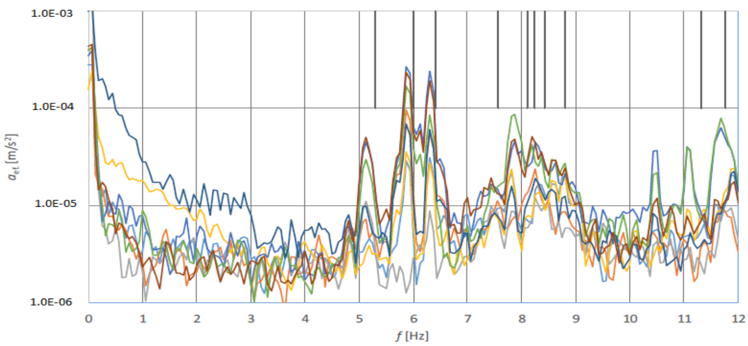

Fig. 4. Acceleration spectra in idle state of the structure.

3 Result comparison

In Table 2, measured mode shapes are assigned to calculated mode shapes in FEA model [3]. Assignment was based on characteristic common signs of each mode shape.

As can be seen in Table 2 the lowest mode shape wasn't found during modal analysis. It is possible that the lowest calculated self-frequency exists but could not be measured because the excitation in this part of the spectrum was small and the response measured by the acceleration sensor was lost in the noise of the used electronics. Most common mode shape occurred at 8.80 Hz, but commonalities may only be found for vertical direction. Horizontal mode shape is different for this frequency. First calculated eigenmode is a simple oscillation of the structure in vertical direction. Also mode shapes No. 5 and 8 could not be matched to any measured mode shapes.

Table 2. Mode shape assignment.

Mode shape	Dynamic calculation [Hz]	Modal analysis [Hz]
1	2.84	-
2	4.63	5.31
3	5.29	5.99
4	6.77	6.43
5	7.04	-
6	7.64	7.68
7	7.67	8.43
8	8.64	-
9	-	(7.57)
10	-	(8.11)
11	-	(8.25)
12	-	11.32
13	-	11.77

4 Conclusion

Operational modal analysis performed on the bridge has shown, that 5 of 11 mode shapes are comparable to calculated mode shapes using FEA model of the bridge. Three of the measured mode shapes are incomparable. While the number of irregularities is fairly high it is convenient to find the cause of such behaviour. Two mode shapes are indicating on non-specified anomalies around one of its abutment. Causes of these anomalies might be hidden failures of the structure. While no information explaining these irregularities can be found the bridge should stay closely monitored and should be further analysed to found the answer on deviations in dynamic behaviour. Standardized tolerance [4] are slightly exceeded if we take 2.84 Hz as the lowest eigenfrequency.

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