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IDENTIFICATION OF BEARINGS STATE ON THE BRIDGE CHECKED BY DYNAMIC TESTS

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Abstract: The paper deals with identifying the condition of bearings on a truss bridge with a reinforced concrete bridge deck by means of repeated dynamic tests. Interferometric radar was used to record the dynamic response of the bridge. This device measures the dynamic displacements of several points of the structure simultaneously. In addition to displacements, the values of eigenfrequencies in the vertical direction were also compared. The frequency values were also determined from acceleration records for purposes of comparison.

KEYWORDS: Structural health monitoring, bridge, radar interferometry, bearing, friction.

1 Introduction

During the structural health monitoring (SHM) of bridges, it is imperative to know how the structure works. Experience has shown that it is very difficult to correctly evaluate all measured data without a thorough and detailed numerical analysis [8, 9]. In some cases, it would be impossible to verify the proper operation of structures without calibrated numerical models. Some structural failures that may not be visible can be detected using a comparison of numerical calculations and test data. The influence of some non-bearing elements of the structure on its behaviour can also be verified in this way.

In SHM, there are four levels of damage description: damage detection; damage localisation, assessment of the extent of the damage; and prediction of the remaining service life of the structure [2, 11, 12]. This paper presents an attempt at answering the questions of whether the structure operates correctly and if not, where the damage is located. An interferometric radar was used to measure the dynamic response of the bridge. It is a technology for measuring distances and displacements of several points of a structure simultaneously. Measurements of displacements by an interferometric radar are currently widespread globally [4, 6, 7, 9, 10] and many scientific teams consider this method to be very promising. Due to the relatively high sampling frequency of observed points' position records, as well as the relatively high resolution and accuracy, it can be considered an alternative method to commonly used technologies in testing the response of structures. When testing with an interferometric radar, a comparison of the phase shift of the electromagnetic wave reflected from the measured object between two time points is used [3]. The radar works remotely and provides efficient recording of dynamic displacements of the structure. Depending on the intensity of the reflected signal, a measurement accuracy of 0.1 mm can be easily achieved. Due to the high sampling frequency of up to 200 Hz, it can detect fast movements of a structure excited by an external load, such as wind or traffic. In addition, the device is portable and can be quickly installed and commissioned, which makes it suitable for quick monitoring, even in emergency situations.

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2 Description of the tested bridge

This paper deals with testing, evaluation and identification of possible damage on the Bratislava Harbour Bridge (Fig. 1). It is a steel four-span combined highway-railway bridge with a length of 461 m. The main bridge structure is two-storey – there is a railway line in the lower part of the bridge and a motorway in the upper part. Additionally, the lower part also contains footbridges for cyclists and pedestrians on both sides.



Fig. 1 Bratislava Harbour Bridge

A detailed numerical model of the bridge was created. The model was calibrated and verified to provide a good conformity of the results of stresses, displacements, eigenfrequencies and eigenmodes compared to experimental tests.

3 Comparison of displacements

IBIS-S interferometric radar was used to measure displacements of the bridge. The speed and accuracy of measurements using the interferometric radar largely depends on the skills and experience of the person operating the device. It is important to find a suitable position for the radar. This should allow enough stabilization of the radar and at the same time should provide enough space for its handling (Fig. 2).



Fig. 2 Location of the radar under the bridge

One of the disadvantages of this technology is a very high sensitivity of the device to environmental influences. However, this can often be eliminated by removing interfering objects from the radar measuring range before starting the measurement itself. Fig. 2 shows the location of the radar under the bridge. The tripod is anchored to a solid base of concrete and rocks. This bank of the Danube is accessible and cleared of shrubs and trees. This place was suitable for targeting the edge of the structure, which significantly helped to identify specific

points of the structure. These were secondary elements – parts of a highway bridge deck. Because of that, evaluation of the displacements of the tested structure must be carried out carefully. The displacement records include also local effects, such as bridge deck deflections between two load-bearing elements of a steel structure, which can not be neglected. The device was aimed at the transverse girders of the upper bridge deck and, in the individual range resolution areas, there were almost no other significant points of the structure. Mainly for this reason, such a placement can be considered appropriate.

The tests were performed without traffic restrictions on the bridge. Traffic was recorded during the testing using a video camera with a sampling frequency of 30 frames per second. The position and speed of all vehicles were documented. In addition to traffic loads, the structure is loaded by wind at almost all times. In cases where a strong gusting wind blows during the measurement, its effects must be considered. Simultaneously with recording the dynamic response of the bridge, it is thus advisable to also record the speed and direction of the wind, e.g. using an anemometer. The wind speed was therefore recorded in the middle of the main span. The sampling frequency of the used anemometer was 1 Hz, which is sufficient in context of the global effects of the wind. Using these records, it was possible to perform numerical simulations. The load from real traffic and wind was applied to the numerical model. The calculated records were then compared with test data.

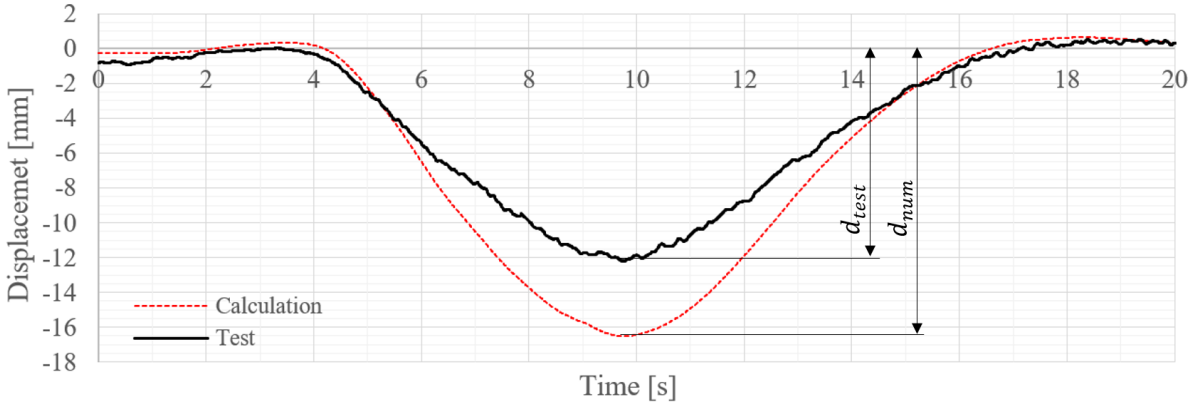


Fig. 3 Comparison of displacement records

A performance indicator (PI_d) was defined to compare the numerical results with the test data [8]

$$PI_d = \frac{d_{num}}{d_{test}}, \tag{1}$$

where d_{num} is the maximum quasi-static value of the calculated displacement and d_{test} is the maximum quasi-static value of the measured displacement (Fig. 3). This indicator shows how the result of the numerical model corresponds to the test data. The closer the value PI_d approaches 1, the more the test results correspond to the results of the calculations.

Thanks to the experience with measuring the displacement of different bridges, where the difference between the results of calculations and the tested data did not exceed the value of ±5%, the research team noticed larger differences between the measured and calculated values in measurements of this bridge. The reason of these differences had to be further investigated. The question was whether the reason for this was a measurement error or a change in the behaviour of the structure. Due to this, several comparative calculations were carried out. However, the difference between the measured and the calculated values was quite large (more than 30 % on average – Tab. 1), so that the only explanation was that a significant part of the cross-section was not included in the stiffness of the cross-section of the bridge. It was concluded that it was, with the greatest likelihood, the reinforced concrete bridge deck, which

was designed so as not to interact with steel truss beams. This deck extends continuously over the entire surface of the bridge, coupled in the transverse direction by steel crossbeams. This entire deck lies on 5 rows of bearings (a total of 1260 bearings) (Fig. 4 and 6).

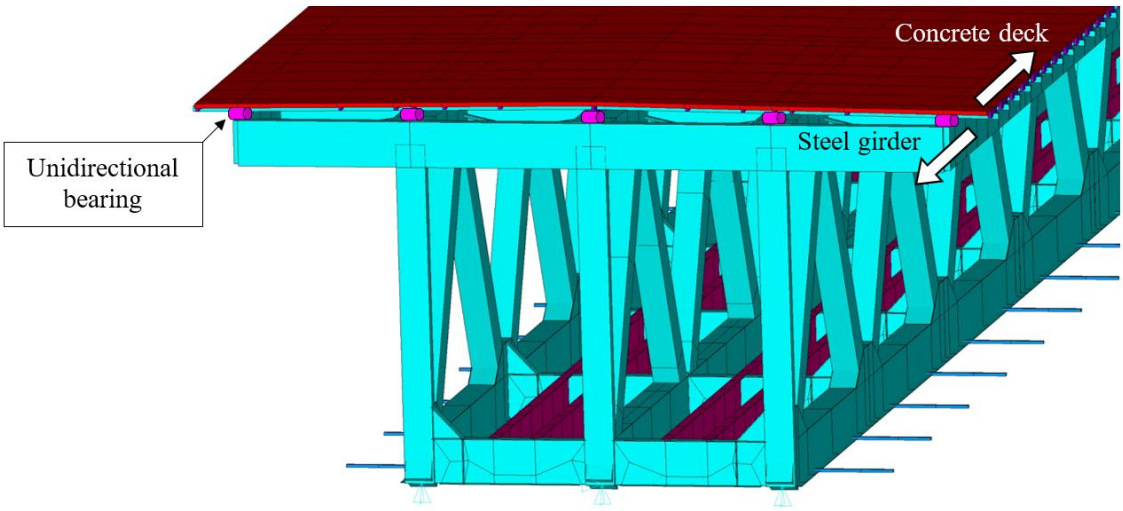


Fig. 4 Bearings

Tests showed that the system of hundreds of plain bearings between the bridge deck and the steel structure may not function as plain bearings as intended. It is possible that there is no sliding and the concrete bridge deck carries part of the load in the cross section. It is a bold statement, but it was also confirmed by the independent measurement of eigenmodes and frequencies in the vertical direction using accelerometers. Since the state of the bearings connecting the steel structure with the concrete bridge deck is unknown, it was necessary to numerically investigate several alternatives (Fig. 5). The first alternative was that the bearings worked perfectly, so the coefficient of friction in the bearings was 0.00. Another alternative was the opposite condition and the coefficient of friction in the bearings was 0.95 – 1.00. The last alternative was a situation where the coefficient of friction in the bearings was only 0.10.

a) situation on the bridge in 9.8 s



b) comparison of displacements

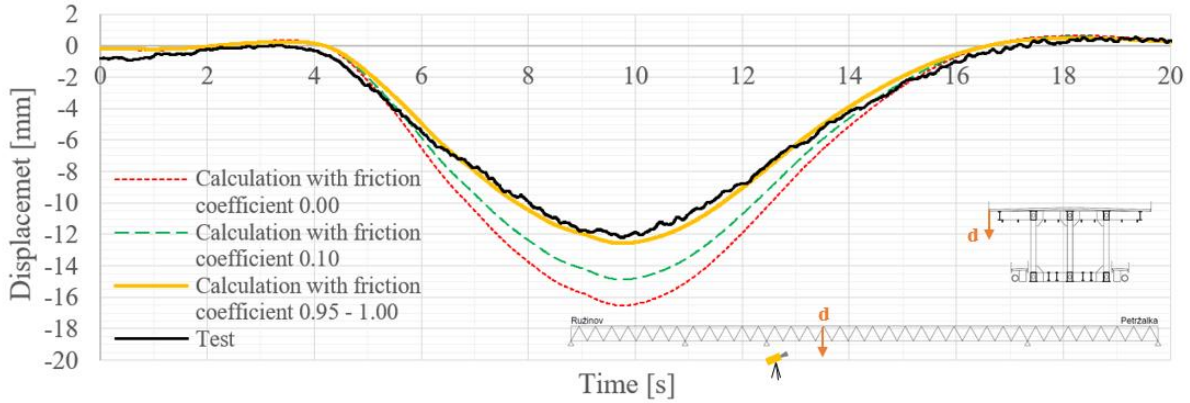


Fig. 5 Influence of friction in bearings on the size of displacement

In Fig. 5 b), there is one of the displacement records of the bridge, when the local (personal) train passes in the direction from Ružinov (Tab. 1 – Tab. 3 – test number 6). At the same time, there are several cars and trucks on the main span of the bridge (situation in Fig. 5 a), the effects of which are also considered in the calculation. Multiple crossings of different trains were also evaluated (Tab. 1 – Tab. 3). Since these were crossings of different types of trains, the individual values of PI_d were assigned a weight w based on the size of the displacement. The size of the total displacement of the structure greatly affects the size of the load and the direction of the load movement. The radar was aimed at the edge of the bridge closer to the track in the direction from Ružinov to Petržalka, and thus the size of the displacement of the structure is larger when the train passes in this direction. Tab. 1 – Tab. 3 contain the average values of the performance indicator $\overline{PI}_{d,w}$, considering the different values of the coefficient of friction.

Tab. 1 Average values of the performance indicator - friction coefficient 0.00

Test number	Type of locomotive	d_{test} [mm]	d_{num} [mm]	w [-]	PI_d [-]	$\overline{PI}_{d,w}$ [-]	σ [-]	*COV*
1	HKV 380	-3.84	-5.97	0.32	1.55	1.39	0.36	26%
2	HKV 671	-11.85	-15.88	0.98	1.34			
3	HKV 383	-6.27	-8.69	0.52	1.39			
4	HKV 380	-5.34	-7.60	0.44	1.42			
5	HKV 350	-10.67	-14.70	0.88	1.38			
6	HKV 380	-12.09	-16.52	1.00	1.37			

Tab. 2 Average values of the performance indicator - friction coefficient 0.10

Test number	Type of locomotive	d_{test} [mm]	d_{num} [mm]	w [-]	PI_d [-]	$\overline{PI}_{d,w}$ [-]	σ [-]	*COV*
1	HKV 380	-3.84	-5.37	0.32	1.40	1.25	0.22	18%
2	HKV 671	-11.85	-14.29	0.98	1.21			
3	HKV 383	-6.27	-7.82	0.52	1.25			
4	HKV 380	-5.34	-6.84	0.44	1.28			
5	HKV 350	-10.67	-13.23	0.88	1.24			
6	HKV 380	-12.09	-14.87	1.00	1.23			

Tab. 3 Average values of the performance indicator - friction coefficient 0.95 – 1.00

Test number	Type of locomotive	d_{test} [mm]	d_{num} [mm]	w [-]	PI_d [-]	$\overline{PI}_{d,w}$ [-]	σ [-]	*COV*
1	HKV 380	-3.84	-4.54	0.32	1.18	**1.05**	0.06	5%
2	HKV 671	-11.85	-12.07	0.98	1.02			
3	HKV 383	-6.27	-6.61	0.52	1.05			
4	HKV 380	-5.34	-5.78	0.44	1.08			
5	HKV 350	-10.67	-11.17	0.88	1.05			
6	HKV 380	-12.09	-12.55	1.00	1.04			

Train direction:

Ružinov → Petržalka

Petržalka → Ružinov

The values of displacements calculated with the coefficient of friction considered to be 0.95 – 1.00 are in quite good agreement with the test data (Tab. 3). From Tab. 1 – Tab. 3 it is clear that the friction in the bearings has a significant effect on the size of the measured

displacements. At the average value of the coefficient of friction of 0.95, the difference between the calculated and measured displacement is approximately 5%.

Repeated dynamic tests and their statistical evaluation then provide more reliable parameter values for SHM. Such a statistical evaluation makes it possible to perform repeated tests in other periods and still evaluate the same parameter, although the load is entirely different. For example, data from Tab. 3 may be input for further repeated tests, which are likely to be carried out annually.

4 Nonlinear Modelling of Bearing Sliding

The phenomenon of bearing sliding between a concrete deck and steel girder was also investigated numerically. In total, there are 1260 bearings between concrete deck and steel structure (Fig. 6 a), specifically, there are five bearings per section, all sections are 1.83 m in distance, spanning the entire length of the bridge. The Ansys nonlinear sliding element Combin40 (Fig. 6 b) [1] was used for connections between deck and steel truss girder in longitudinal direction. In such a model, different values of the coefficient of friction are investigated. In perpendicular direction, a fix connection was assumed, which satisfied the condition that both concrete deck and steel truss work together and secure horizontal rigidity.

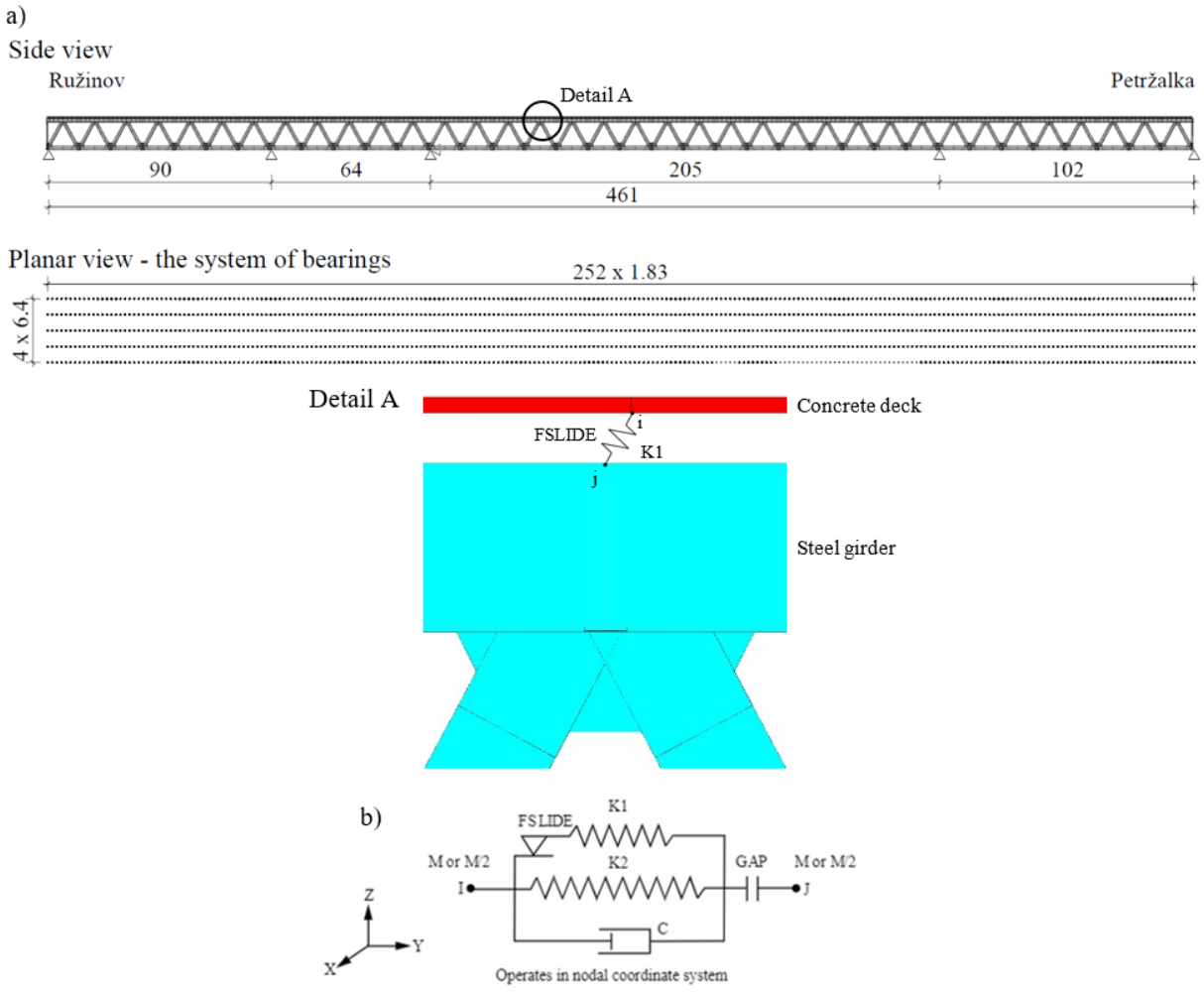


Fig. 6 a) Scheme of the bridge with modeling of nonlinear connections between deck (point i) and steel girder (point j). b) Combin40 element with slide option [1]

Numerical studies with various friction coefficients were performed. The axle loads of train and corresponding traffic (Tab. 1 – Tab. 3 – test number 6) passing over the bridge (Fig. 5a) was investigated in the next numerical study.

Fig. 7 contains the result of the relatively high used value of friction coefficient 0.95. It is easy to recognize that the friction in bearings is almost negligible and maximum value of vertical displacement reaches about 12 mm, which correspond with the test results (Fig. 5b).

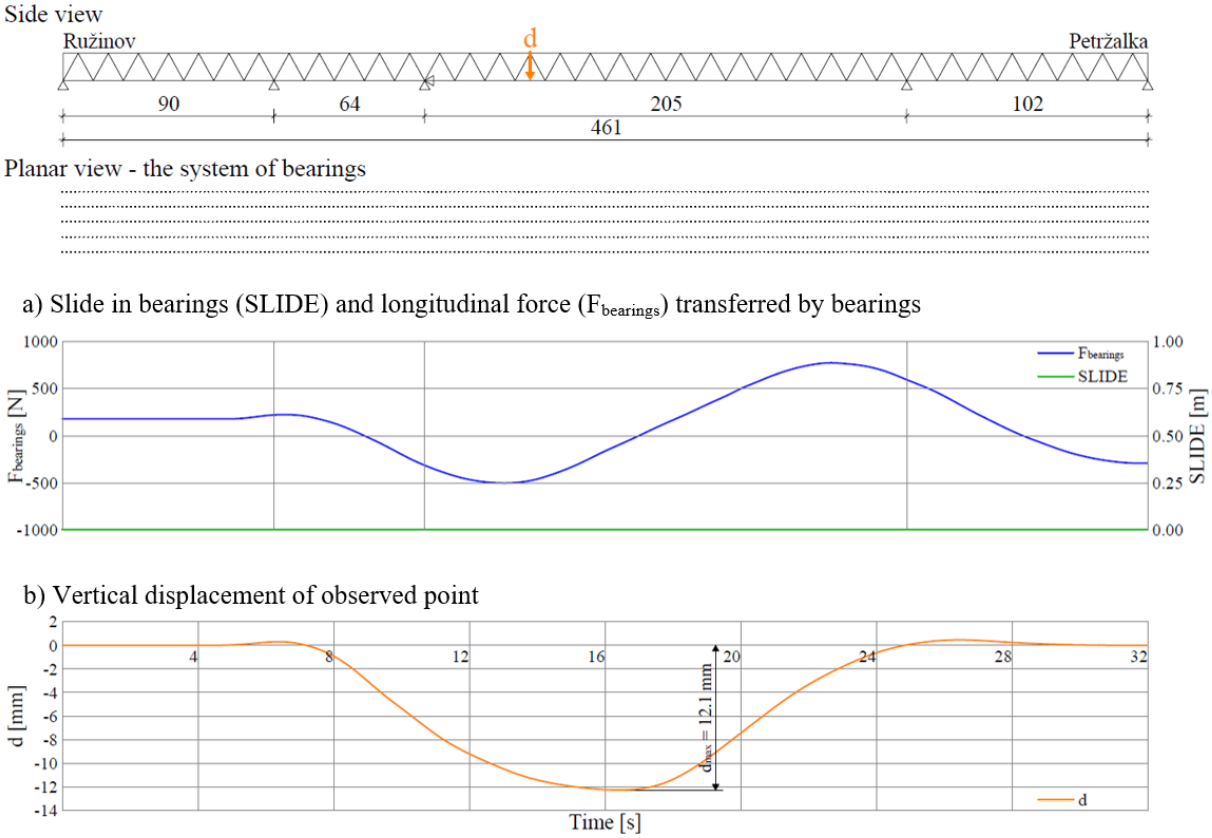


Fig. 7 Case considering the friction coefficient 0.95

Fig. 8 shows the result when friction coefficient is only 0.08. It corresponds with high quality of bearings working as investigated during design period. In Fig. 8 a), the minimum (MN) and maximum (MX) values of sliding along the bridge are shown, while in b), the values of SLIDE can be seen, and in part c), one can see the maximum value of displacement d. One can notice that the value of vertical displacement (16 mm) does not correspond to the value obtained from the test (12.1 mm).

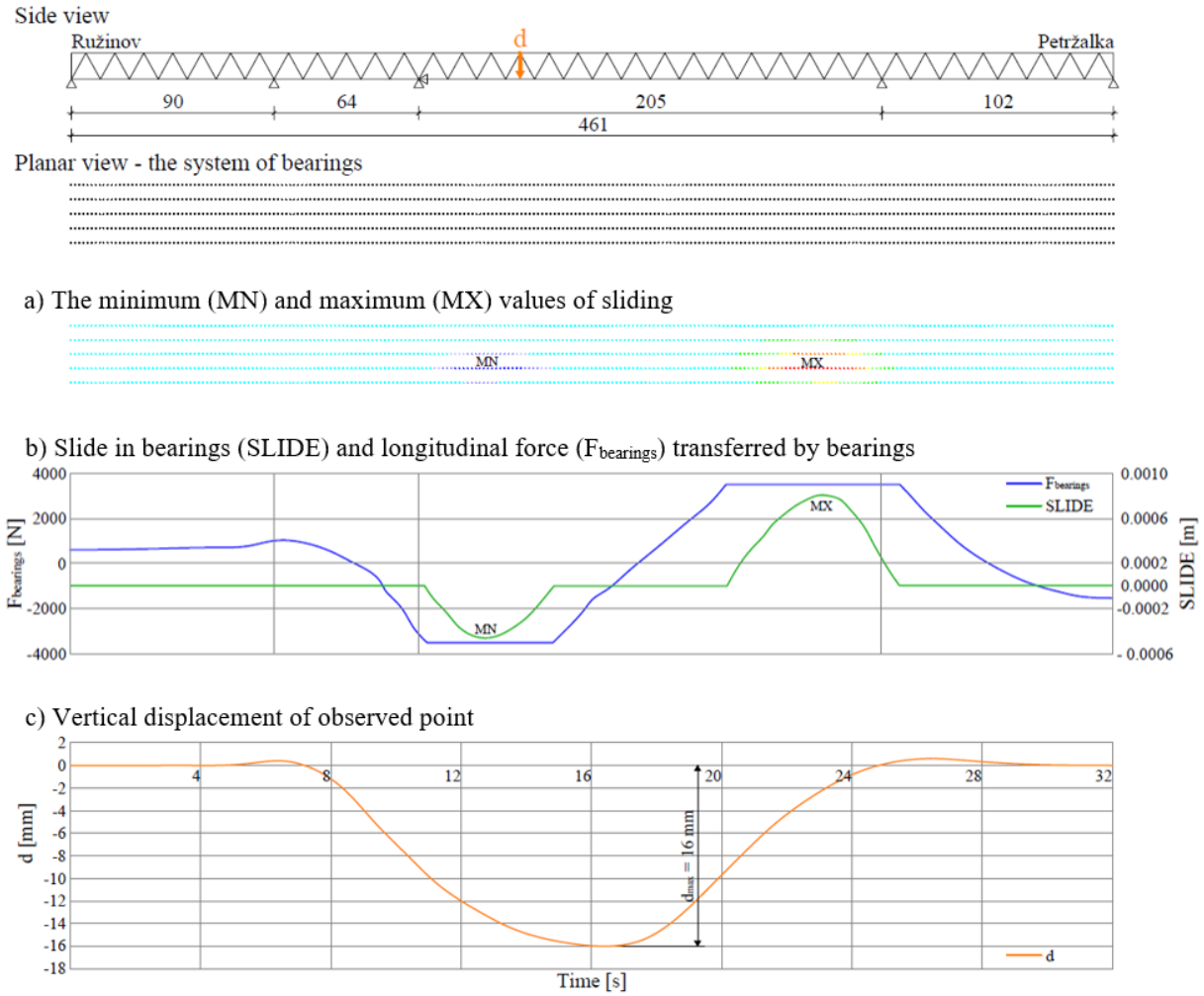


Fig. 8 Case considering the friction coefficient 0.08

From this analytical study follows a conclusion that the bearings between deck and steel truss are not working the way they had been designed to. This can be caused by problems with maintenance or by degradation of materials.

Due to this state, the forces transferred to the deck are bigger than those calculated during the design and, conversely, the stresses in steel truss are a little lower due to redistribution.

The positive result is that it was possible to check the state of bearings on the bridge connecting the upper concrete deck and bottom steel truss using radar interferometry.

5 Eigenfrequencies

In addition to displacements, the values of the first eigenfrequencies in the vertical direction were also compared. The eigenfrequencies were determined from the displacement records using a discrete Fourier transform [5]. The eigenfrequencies were determined from the amplitude spectrum of displacements, which were compared with the frequencies calculated by modal analysis of the numerical model. Fig. 9 shows the amplitude spectrum of displacements from radar record, where the first frequency of 0.89 Hz is well identified. This value was also verified by the measurement using accelerometers placed in the vertical direction along the main span. Other modes and frequencies were also obtained using accelerometers (Tab. 4) [11].

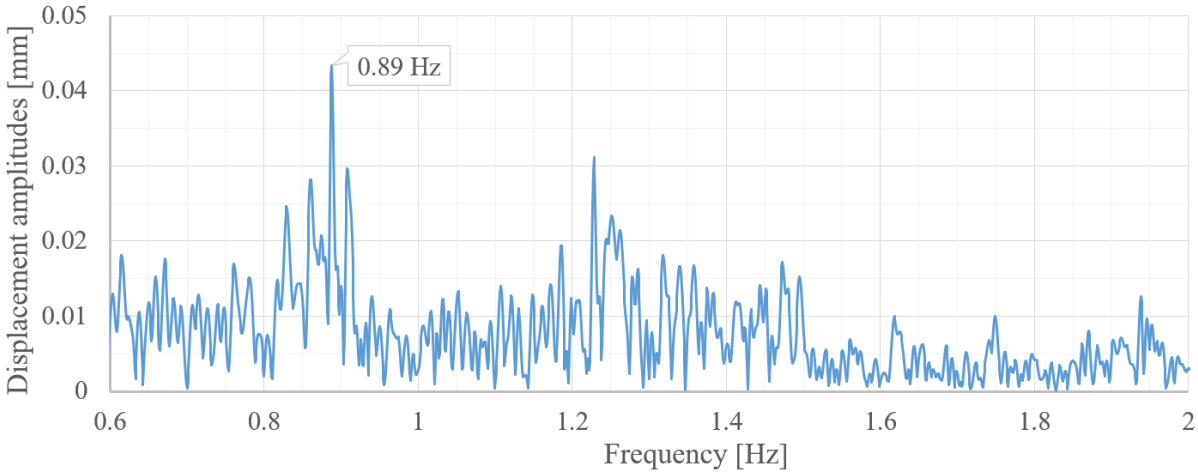
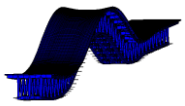
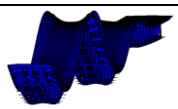
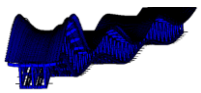


Fig. 9 Amplitude spectrum of displacements

The effect of friction in the bearings was also proved by comparing these results, as shown in Tab. 4. Again, the calculated eigenfrequencies, considering the coefficient of friction in the bearings to be approximately 0.95, conform well with the test data.

Tab. 4 Comparison of eigen frequencies

Eigenmode (in vertical direction)		Eigen frequency [Hz]		
		Measured	Calculated	
			friction 0.00	**friction 0.95**
1		0.89	0.79	**0.89**
2		1.96	1.80	**1.95**
3		2.31	2.13	**2.23**

CONCLUSION

Using two independent measurement methods and comparing their results with a numerical model, it can be assumed that the bearings between the concrete deck and the steel girders of Bratislava Harbour Bridge, which should move freely in the longitudinal direction, may be immobile for as of yet unknown reason. This can be caused by corrosion, dirt, poor maintenance, etc. After contacting the bridge administrator, it was found out that during the inspections, these bearings were not inspected due to their inaccessibility. It is obvious that stresses in the longitudinal direction caused by the interaction of the two parts are transmitted to the concrete bridge deck, which could cause formation of cracks in the areas of the bridge support. Only after considering such a model did the value of the measured displacements approach the calculated displacements, and the eigenfrequencies in the vertical direction coincided relatively well with the values obtained by operational modal analysis. This makes it possible to state that the tests are correct, and the numerical model is well verified and calibrated.

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