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FULL-SCALE DYNAMIC TESTING OF THE CORROSION-DAMAGED, STEEL-TRUSS STRUCTURE OF OLD BRIDGE OVER THE DANUBE IN BRATISLAVA

BADANIA DYNAMICZNE W SKALI RZECZYWISTEJ USZKODZONEGO KOROZJĄ KRATOWEGO, STALOWEGO STAREGO MOSTU PRZEZ DUNAJ W BRATYSŁAWIE

A b s t r a c t

A large number of existing bridges need to be rehabilitated due to increasing traffic and/or loading requirements and also corrosion action. In this paper, a procedure is presented for estimating the ultimate capacity of a steel bridge over the Danube in Bratislava – Old Bridge (built in 1945). The development of a simplified Finite Element Model (FEM) and basic modal parameter calculations preceded the experimental investigations of the bridge via static and dynamic in-situ loading tests, so that the main assumptions adopted in the FEM were assessed through comparison between measured and predicted dynamic and modal parameters of the bridge structure. The bridge structure computational model was then optimized by structure variables (primarily, steel structure joints mass and corrosion grade) to achieve the minimum differences between the experimental and theoretical results. The calibrated FEM with the optimal combinations of the mentioned variable values were defined and finally used for structure calculations and for strengthening the design of the real bridge structure.

Keywords: *corrosion action, structural health monitoring, system identification, FEM, experimental tests in situ, spectral analysis*

S t r e s z c z e n i e

Wiele istniejących mostów musi zostać odnowionych w związku z rosnącym natężeniem ruchu i/lub z powodu wymagań obciążeniowych, a także w skutek działania korozji. W niniejszej pracy przedstawiono procedurę szacowania nośności granicznej stalowego mostu na Dunaju w Bratysławie - Old Bridge (zbudowanego w 1945 r.). Opracowanie uproszczonego modelu MES i podstawowe obliczenia parametrów modalnych poprzedzały badania statyczne i dynamiczne mostu w skali rzeczywistej. W związku z tym główne założenia modelowania MES zostały przyjęte na podstawie porównania między zmierzonymi i przewidywanymi dynamicznymi i modalnymi parametrami konstrukcji mostu. Model obliczeniowy konstrukcji mostu został następnie zoptymalizowany przez parametry konstrukcji (przede wszystkim przez uwzględnienie masy węzłów stalowych, stopnia korozji), aby osiągnąć minimalne różnice między wynikami badań doświadczalnych i teoretycznych. Skalibrowany model MES z optymalnymi kombinacjami wymienionych parametrów został zdefiniowany i wykorzystany do obliczeń i następnie wzmocnienia konstrukcji rzeczywistej mostu.

Słowa kluczowe: *działanie korozji, monitoring konstrukcji, identyfikacja systemu, MES, badania in situ, analiza widmowa*

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1. Introduction

Riveted bridges account for the majority of steel bridges that were built in different parts of the world before the middle of the last century. A large number of these bridges are still in service today. However, some of these bridges are more than 100 years old. Therefore, it is clear that while many existing bridges are structurally adequate with respect to the maximum design axle loads, they may suffer from fatigue related to the cyclic application of modern freight equipment axle loads [1–8]. It should be noted that, generally, bridges designed within the past 50 years have considered fatigue effects, but that earlier bridge design did not include such considerations, even though, in some cases, the bridge may be found to be adequate for fatigue loads. The problems came up of how the resistance against repeated loads of the bridges is today. Usually, the authorities ask about two important issues, the first is that the bridge should be sufficiently safe for actual service conditions and if so, the second issue is, what is the expected residual life and what are the requirements for inspection and maintenance to ensure the expected residual life [6–9]. An essential part of the safety check of existing road bridges is the assessment of the static load-carrying capacity, and in some cases, a static or dynamic load test (even fatigue strength test) becomes necessary [9–12]. There are presently no regulations for the assessment of existing road bridges, expert opinions are normally used to obtain fatigue life estimates. It must be underlined; however, that fatigue is a far less relevant issue for road bridges than it is for railway bridges. As traffic density and traffic loads are constantly increasing, it is assumed that fatigue will be increasingly important for road bridges too. Because of the fatigue strength test time and financial implications, it is preferable to perform static or dynamic loading tests [13, 14] of the bridge structure as a part of the long-time monitoring to control its ultimate capacity [9, 12, 15].

2. Steel bridge case-study

The steel bridge over the Danube in Bratislava (Old Bridge) was built in 1945. The load-bearing structure of the bridge is created partly with continuous truss main beams over the river of 75.71 m, 91.50 m, 75.64 m (spans 2, 3, 4) and partly by single adjacent truss beams of 32.42 m and 75.82 m (spans 0, 1) and 75.18 m and 32.22 m (spans 5, 6), [3]. The bridge deck is composed of a steel grate system (cross and longitudinal beams) bearing the reinforced double T–prefabricated road panels, Figure 1. The soil conditions for foundations of the two massive abutments are very similar on both sides of the river and the resistant substratum (gravel and sandy gravel) is a depth of more than 22 m.

Foundations of the pylons are big reinforced concrete blocks on the same substratum as both of the massive abutments. The 91.5 m longest span (span 3) and two ≈ 75.0 m adjacent truss main beams (spans 2, 4) was chosen for the case study presented in this paper. These three truss continuous spans are the most representative and most failing part of the 461.07 m long bridge. Most of the cracked stringer to floor-beam connections were located in this part of the bridge. Figure 2 shows a bridge cross-section and bridge schematic view.

Fig. 1. View of the Old Bridge in Bratislava

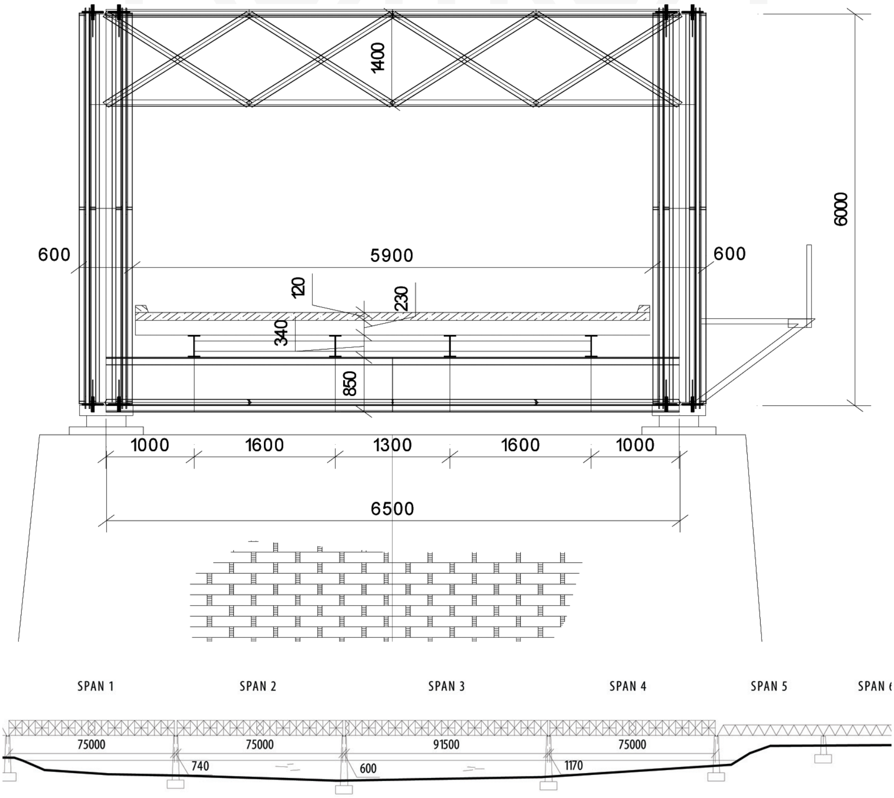


Fig. 2. Cross section and schematic view of the Old Bridge over the Danube in Bratislava

3. FEM analysis

Analysis of the bridge was performed using the IDA NEXIS software. The 3D global model incorporated all primary and secondary load-carrying members in the bridge was, however, excluded at this stage. Heavily gusseted connections, such as those between the main truss members and between wind-bracing elements and the main truss, were modelled as moment-stiff connections, while pin connections were adopted for secondary members, such as sway and cross-bracing elements. Eccentrically connected members, such as floor beams and wind–bracing elements, were coupled in the model. Longitudinal and transverse floor beams created as the built-up section from double angles with riveted steel plates were modelled using beam elements. The connections between longitudinal and transverse floor beams were made using multi-point constraints (MPC) and eccentric node-to-node gap elements were employed to simulate the contact condition between the double angles and the longitudinal floor beam web. All non-bearing elements of the truss girders and the bridge deck were included as a mass load of the structure. Four variations of the expected conditions were simulated: with and without corrosion effect; lower and higher joint mass estimation. The simplified FE model is created by 2758 joints and 5904 beam elements. A render of the computing model layout is presented in Fig. 3.

The first fifteen natural frequencies and modes of natural bridge vibration were calculated to compare with their experimental values from the DLT measurements. As an example, some of them are shown in Fig. 4. Variations of expected conditions and the comparison of results are explained in Figure 5. From comparison of these FEM natural frequency results it seems that the numerical natural frequency values are affected by the bridge steel structure joints mass, corrosion grade and FE–model degree of accuracy. Bridge deflection calculation for static and dynamic loading test: The maximum static vertical deflection values in the relevant points of the spans, positions of measured points, load positions and the effectiveness of the testing loads (SCANIA lorry of 15.7 ton mass) according to the Slovak standards [13, 14] for the static loading test (SLT) were taken into account and also calculated via IDA Nexis software package. Results from the calculations of static deflection were also used for dynamic loading test (DLT) load effectiveness.

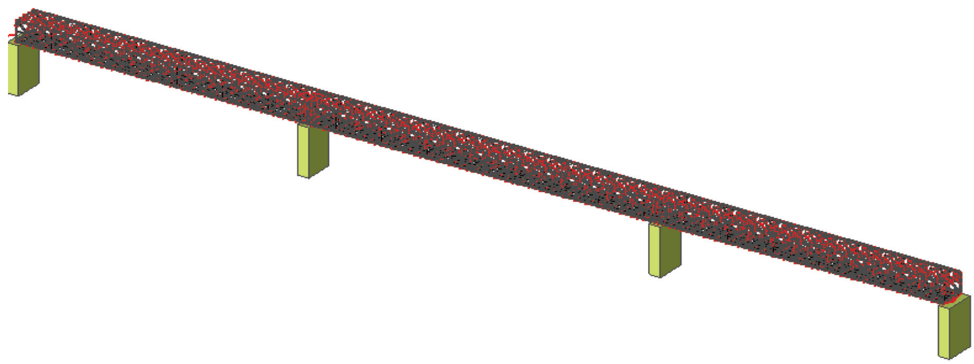


Fig. 3. Global FEM layout

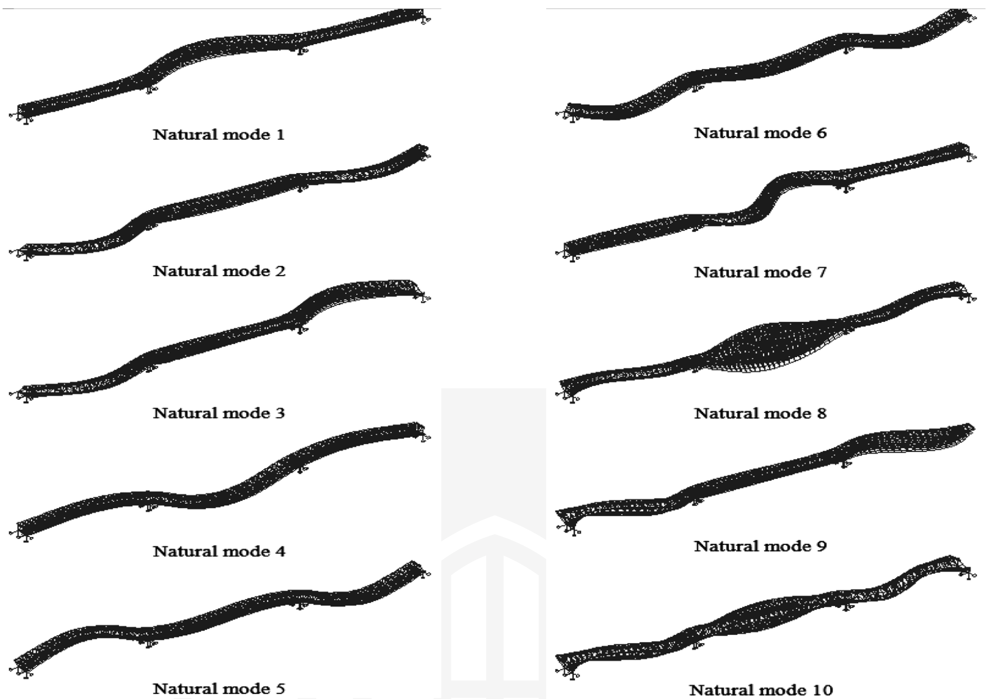


Fig. 4. Calculated modes of the bridge natural vibration

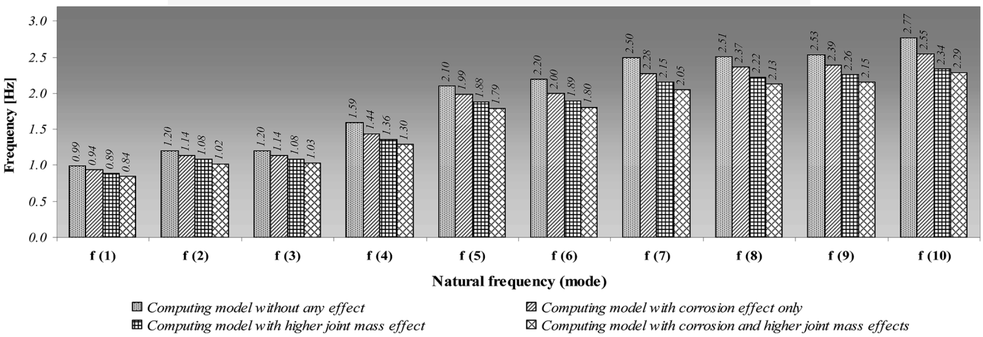


Fig. 5. Computational models natural frequencies results comparison

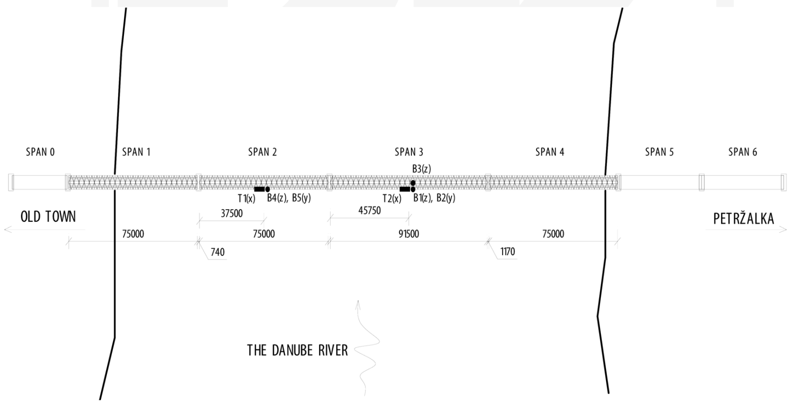
4. Dynamic loading test

Before the bridge dynamic loading test performance, the static loading test was carried out using the SCANIA load vehicle with mass of 15 700 kg. The deflection values in the relevant points of the tested spans (3, 4 and 5 spans) were measured using the precise geodesy levelling method with Leica equipment [9].

The dynamic response tested spans of the bridge was also induced by passing load vehicle SCANIA in the both directions with various speed. The sensor positions are shown in Fig. 6. The operating dynamic loading test started with a load speed of v = 5 km/h (crawling speed) which increased up to the maximum achievable speed v = 62 km/h.

A computer-based measurement system (CBMS) was used to record the dynamic response of the bridge excitations induced by the testing vehicle over the DLT period. The investigated vibration acceleration amplitudes were recorded at selected points with the maximum calculated deflection in each of the investigated four spans (Fig. 7). At the same points, the vibration amplitudes in both horizontal (longitudinal and perpendicular to bridge) directions were also recorded. Output signals from the accelerometers (Brüel–Kjaer, BK8306) were preamplified and recorded on two PC facilities with A/D converters and software packages NI and DISYS, The experimental analysis was carried out in the Laboratory of the University of Zilina. Natural frequencies were obtained using spectral analysis of the recorded bridge response dynamic components of the structure vibration, which are considered ergodic and stationary [15, 16, 17, 18]. The vibration ambient ability has been investigated by means of the correlation and spectral analysis in order to obtain cross-spectral densities $G_{xy}(f)$ and coherence function $\gamma_{xy2}(f)$, see also [9, 16]. The frequency response spectrum has also been obtained by using a two-channel, real-time analyzer BK–2032 in the frequency range 0–10 Hz. The output signal in the form of the Fourier frequency spectrum (power spectrum – $G_{xx}(f)$) was also recorded by computer and printed by laser printer and x-y plotter. Spectral analysis was performed via National Instruments software package NI LabVIE.

As an example, Figure 8 (a, b), shows a part of the spectral analysis procedure results PSD $G_{xx}(f)$ of the dynamic vertical components structure vibration from the bridge DLT. Figure 8 also shows corresponding cross-power spectral density $G_{xy}(f)$, Figure 8 (c), with its phase spectrum $\theta_{xy}(f) \rightarrow$ (d), coherence function $\gamma_{xy2}(f) \rightarrow$ (e) and transfer function $H_{13}(f) \rightarrow$ (f). In Table 1, values of the calculated and measured natural frequencies are introduced for comparison.



LEGEND:

- **B1, B2, B3, B4, B5** **Accelerometrs BRUEL-KJAER 8306 (0-450Hz)**
- **T1, T2** **Strain gauges KISTLER (0-500Hz)**

Fig. 6. Sensor positions layout

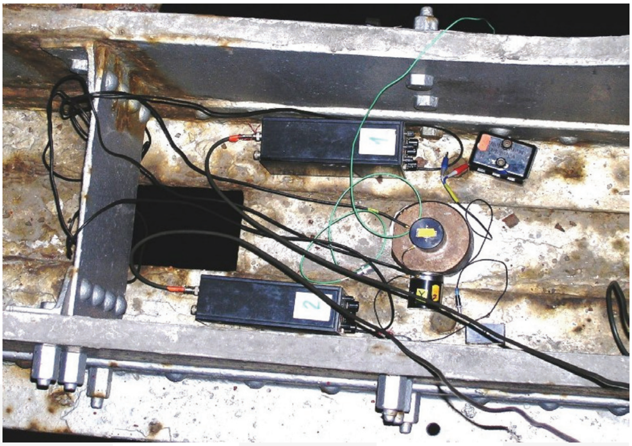
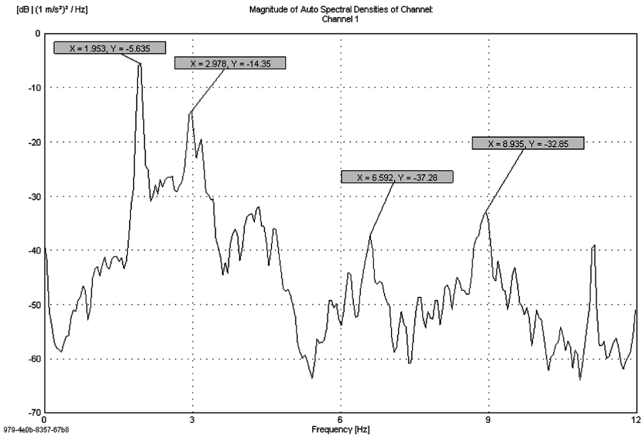


Fig. 7. Acceleration sensors with amplifiers

a)



b)

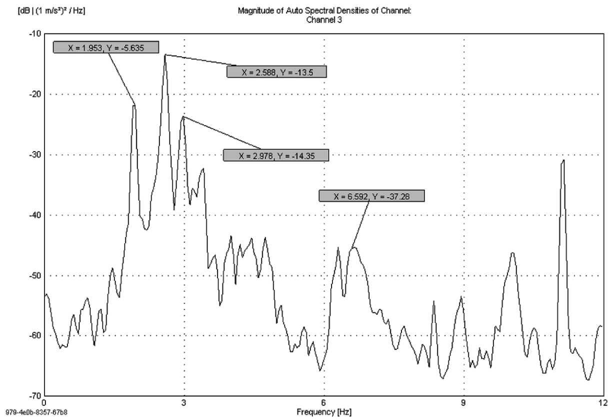
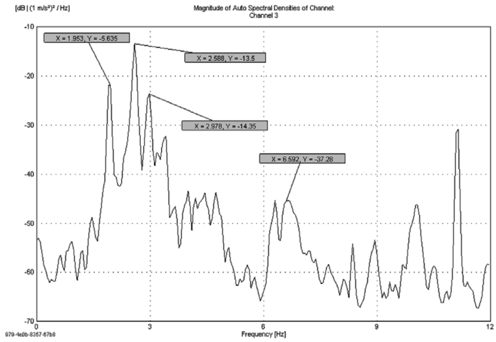
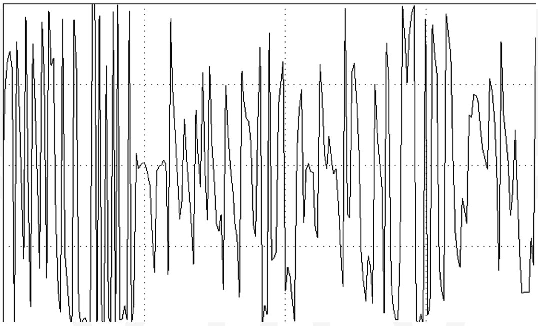


Fig. 8. Spectral analysis procedure

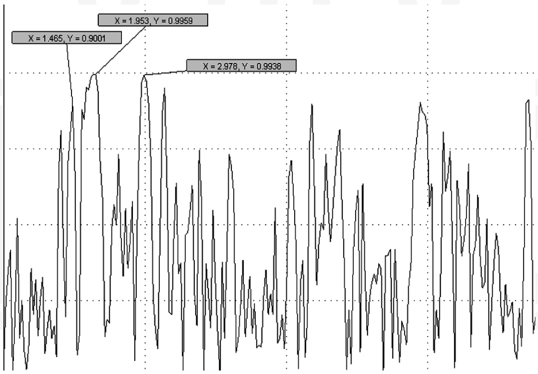
c)



d)



e)



f)

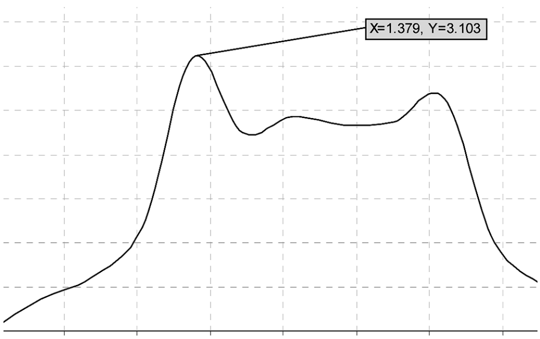


Fig. 8 (cont.). Spectral analysis procedure

Table 1

Calculated and measured natural frequencies

Natural frequencies ofthe bridge				
Natural mode	Numerical calculation *		Experimental analysis **	Vibration tendency
	Model 1	Model 2	2 – 3 span	
1	0.993	0.844	-	horizontal bending
2	1.200	1.024	-	horizontal bending
3	1.204	1.027	-	horizontal bending
4	1.589	1.298	-	vertical bending
5	2.104	1.788	1.53	horizontal bending
6	2.199	1.801	1.85 (1.82)	vertical bending
7	2.501	2.049	1.93 (1.95)	vertical bending
8	2.514	2.129	~ 1.78 (1.79)	horizontal bending
9	2.531	2.152	~ 1.78 (1.79)	horizontal bending
10	2.767	2.291	2.55 (2.58)	torsion
11	2.934	2.681	~ 2.95	torsion
12	2.937	2.693	~ 2.97	torsion
13	3.091	2.977	~ 3.00	axial
14	3.154	2.994	2.58	horizontal bending
15	3.754	3.551	2.97	horizontal bending
16	3.772	3.568	3.42 (3.44)	horizontal bending

* FE Model 1 – without corrosion and higher joint mass effects. FE Model 2 – with corrosion and higher joint mass effects.

** Natural frequencies from PSD with dominant peaks.

5. Conclusions

Theoretical and experimental investigation of the Old Bridge structure over the Danube in Bratislava is briefly described in the paper. The following conclusions can be drawn:

- The predicted dynamic behaviour of the bridge by a simplified FEM analysis calculation was compared to the measured one. Despite both the complex structural layout of the bridge and simplifying assumptions of the model, results showed strong agreement for all identified frequencies in the basic frequency range 0–5 Hz;
- The bridge dynamic loading test results are proof of the truss main beams continuousness which is in strong agreement with the adopted computational model and applied input bridge parameters;

- From comparison of the FEM natural frequency result, it follows that the numerical natural frequency values are mainly affected by the bridge steel structure mass with adding joints mass, corrosion grade and FE–model degree of accuracy (e.g. 2D, 3D, etc.). If such effects are neglected in the FEM analysis, the resulting theoretical and experimental dynamic properties can be different, mainly during modal identification procedures;
- Calculations of the natural frequencies with several combinations of the input parameters (with and without corrosion effect, lower and higher joint mass estimation, etc.) were performed in the numerical FEM analysis. The computational model with acceptation of the 96% cross-section area (4% corrosion loss) and also including the steel structure higher joints mass effect yield very close natural frequencies values in comparison with experimental ones;
- This computational model can be applied for the bridge fatigue and resistance analysis [5, 19, 20] before starting a decision making process regarding bridge strengthening or general reconstruction.

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References

[1] Proulx J., Hebert D., Paultre P., *Evaluation of the dynamic properties of a steel arch bridge*, Proc. 10th Int. Modal Analysis Conference, USA, San Diego 1992, 2, 1025-1031.

[2] Inaudi D., Bolster M., Deblois R., French C., Phipps A.. Sebasky J., Western K., *Structural health monitoring system for the new I–35W St. Anthony Falls Bridge*, Proc. 4th International Conference on Structural Health Monitoring on Intelligent Infrastructure (SHMII–4), Switzerland, Zurich 2009.

[3] Adamson D., Kulak G., *Fatigue tests of riveted bridge girders*, Structural Engineering Report No. 210, University of Alberta, August 1995.

[4] Akesson B., *Fatigue life of riveted steel bridges*, CRC Press/Balkema, London 2010.

[5] JRC Scientific and Technical Reports. Assessment of existing steel structures: recommendations for estimation of remaining fatigue life, Report No. 43401, Joint Research Centre, European Commission, Luxembourg 2008.

[6] Wysokowski A., *Durability of steel bridges as function of fatigue and corrosion phenomena's*, IBDiM, Studies and Materials 53, CD–ROM, 2001 (in Polish).

[7] Bień J., *Failures and diagnostics of bridges*, WKŁ Press, Warszawa 2011 (in Polish).

[8] Zhou Y.E., Yen B.T., Fisher J.W., Sweeney R.A.P, *Examination of fatigue strength (S–N) curves for riveted bridge members*, Proc. 12th Annual Bridge Conference, Canada, Montreal 1995, 102-109.

[9] Bencat J., *Static and dynamic test results of the Old Bridge – over the Danube in Bratislava*, Technical Report: No. 6–3–21/SvF/09, ZU/Zilina, 2009.

[10] Bata M., Plachy V., Sykora J., Bily V., Polak M., *Static and dynamic tests of bridges*, Proc. of Czech – US Bridge Conference, Prague 1994.

[11] Williams C., *The effects of vibration on large structures and the determination of dynamic characteristics*, Proc. International Conference on Engineering Integrity through Testing, UK, Birmingham 1990, 441-452.

[12] Williams C., *Testing of large structures using vibration techniques, Structural integrity assessment*, Elsevier Applied Science, UK, London 1992.

[13] Slovak Standard STN 73 6209: Loading test of bridges, (in Slovak).

[14] Slovak Standard STN EN 1991 – 3: Traffic loads on Bridges, (in Slovak).

[15] Ren W. X., Zhao T., Harik I. E., *Experimental and analytical modal analysis of a steel arch bridge*, Journal of Structural Engineering, ASCE, Vol. 130(7), 2004, 1022-1031.

[16] Bendat J.S., Piersol A.G., *Engineering applications of correlation and spectral analysis*, (2nd edition), Wiley Interscience, New York 1993.

[17] National Cooperative Highway Research Program (USA). Dynamic Impact Factors for Bridges, NCHRP Synthesis 266; National Academy Press, DC, Washington 1998.

[18] SIA 269/3 – Existing Structures: Steel Structures, Swiss Society of Engineers and Architects (SIA), Switzerland, Zurich 2011.

[19] Eurocode 3. Design of Steel Structures,Part 1– 9: Fatigue; European Standard EN 1993–1–9, Brussels 1993.

[20] European Community: Guidline for Load and Resistance Assessment of Existing European Railway Bridges. European Commission (EC) Sixth Framework Programme, Sustainable Bridges, Sweden, Malmö 2007.