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EVALUATION OF PERFORMANCE INDICATOR OF RAILWAY BRIDGES USING UPDATED FINITE ELEMENT MODEL

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Abstract: Structural health monitoring (SHM) can provide information needed to make important decisions regarding the maintenance of bridge structures. However, the data collected from monitoring needs to be first translated into actionable, quantitative or qualitative based characteristics, that indicate the condition of a bridge. This paper presents a process of evaluation of such performance indicator in case of a steel railway bridge using the updated FE model and in-situ measurements of strains on selected stringers and floorbeams.

KEYWORDS: structural health monitoring, railway bridge, model updating, finite element model.

1 Introduction

Railway bridges are an important part of the civil infrastructure and substantial for the transport industry. Their condition deteriorates with increased age, therefore inspections are performed regularly to ensure their safety and serviceability. However, a decision about when, how and what part of a bridge needs to be repaired is a common and difficult management task. Additional information to support these decisions can be provided by structural health monitoring (SHM) [1]. There use to be a great amount of raw data collected from monitoring, which needs to be translated and converted into actionable information about structure condition. Such (quantitative or qualitative) information is called performance indicator [2].

In presented work, a performance indicator (PI) of a steel railway bridge is proposed. It was developed to track the state of selected stingers and floorbeams, which are usually one of the structural elements most prone to corrosion and fatigue cracks. The PI is based on results from an updated and verified FE model and response of the real structure measured by strain gauges. One of the advantages of using the FE model updating method for monitoring of railway bridges is the ability to simulate a behavior of each element of the structure [3], e.g. selected stingers and its flangers. In this way, global monitoring techniques (using accelerometers, interferometric radar…) are combined with local monitoring techniques (strain gauges) to obtain a better picture about potential local damage on individual elements, as recommended in [4]. Suitability of strain gauges for monitoring of steel railway bridges was confirmed in [5] and there are several case studies of SHM on bridges, where the strain was measured on different structural elements [6-9]. The work was also based on past experiences with test and measurements of various bridges [10,11].

2 Bridge structure and test description

The tested railway bridges are located in northern Slovakia over the river Vah. There are two bridge structures, one for each of two tracks (T1, T2). One bridge structure consists of three simple supported objects (Fig. 1): a truss with a span of 57,4m (O2) and two girder bridges at sides with a span of 29,4m (O1, O3).

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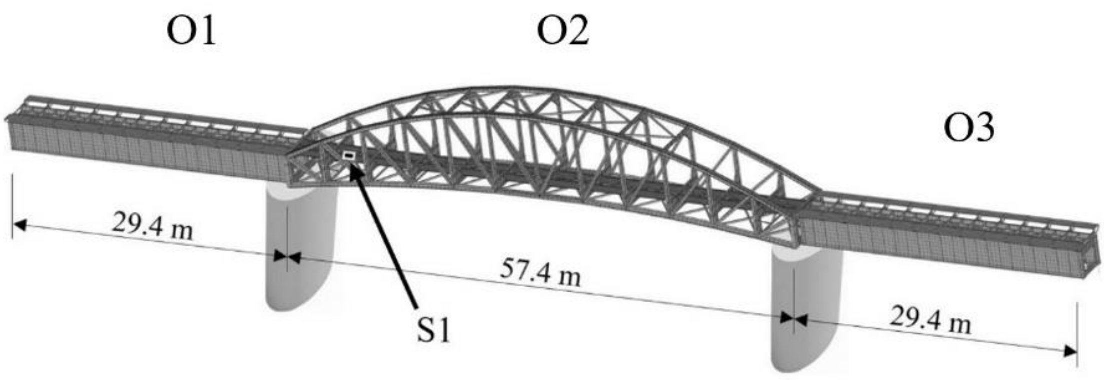


Fig. 1. One of the tested bridges (T2) and the location of sensor S1 on the upper flange of a stringer

On every bridge structure, sleepers are directly connected to stringers, whose spans are 4,2m and 4,9m (O2) or 2,45m (O1, O3). Stringers, floorbeams and main girders are attached with riveted double angle connection.

Strain gauges were applied to selected flanges of stringers and floorbeams, overall 32 sensors were used. Several measurements were done during normal operating conditions in regular monthly testing campaigns. Moreover, accelerometers, interferometric radar, temperature sensors and video recorder were used to collect data about the dynamic and static response of the bridge and passing trains.

3 Finite element model updating

To compare and evaluate test results and to perform detailed calculations, finite element models of the bridge structures were prepared (Fig. 1). Material and geometric characteristics were assumed as used in original documentation.

Individual elements were mainly modeled as follows: webs of element cross sections as shell elements, flanges as beam elements. Beams formed from two or four L beams (top and bottom bracing, stringer bracing…) are an exception - they were modeled using just beam elements. The weight of individual structural elements was automatically calculated according to the density of the material. The weight of other (non-structural) parts was calculated and added to the FE model as mass elements to selected nodes. Fig. 2 presents a detail of the bottom part of the FE model. Different colors of elements represent different cross-section characteristics.

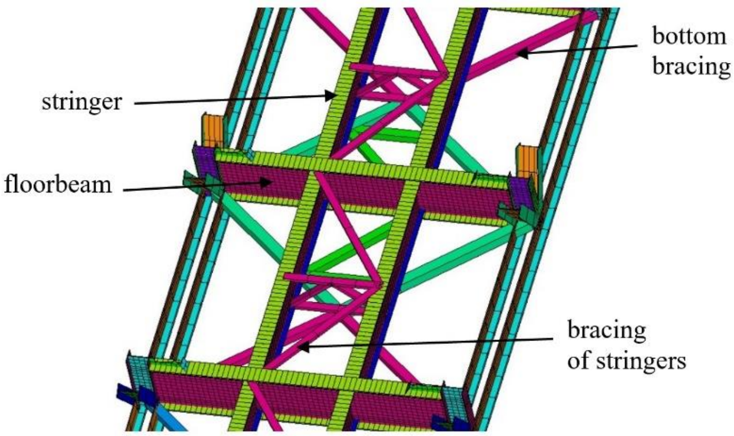


Fig. 2. Detail of selected part of the FE model (structure T2 - O2)

The FE models were updated and verified according to initial test results from accelerometers (natural frequencies, mode shapes) and interferometric radar (natural frequencies, vertical displacement in the middle of a span). In this process, several features were adjusted, such as stiffness of the cross-section of the bridge, Young modulus, mass distribution and cross-section dimensions of individual structural elements were checked with real dimensions obtained from in-situ measurements. Finally, the FE model was calibrated so that the agreement with test results was sufficient. A comparison of the first natural frequencies acquired from the FE model, accelerometers and radar is in Table 1. The updated and verified FE model was subsequently used for evaluation of the following measurements.

Table 1. Comparison of the natural frequencies (structure T2 - O2)

Mode shape no.	Frequency [Hz]			Description of mode shape
	Calculation (FE model)	Measurement (Accelerometers)	Measurement (Interfer. radar)	
1	1,97	1,88	1,87	vertical
2	4,21	4,74	4,74	transverse
3	5,46	5,63	5,60	torsional
4	5,78	5,81	5,84	vertical

4 Evaluation of performance indicator

During each measurement, a strain time history was collected from the sensors. As mentioned above, the measurements were carried out during normal operating conditions with a high diversity of passing trains (passenger, regional, inter-city, freight…). The loading on the bridge hence depended on several factors, such as speed, axle weight, wheelbase, type of locomotive and carriages of the passing trains. As a consequence, the amplitudes of strain varied between measurements and their values couldn't be simply compared.

The aim of the work was to find a methodology how to compare measured time histories and detect changes in structural response. The first step was to transform the values of strains to dimensionless quantity, which would be independent of the characteristics of passing train. As shown in Fig. 3, the static loading forces from a train vehicle can be defined when the weight on axle and distances between axles are known. This information was available, therefore it was possible to determine the loading of the bridge during each measurement. The load was subsequently used to calculate stress using influence lines of normal stresses in locations of sensors. These were calculated from static analysis of the FE model, updated according to data from accelerometers and interferometric radar. As a result, the peak values of normal stresses from a passing train were obtained.

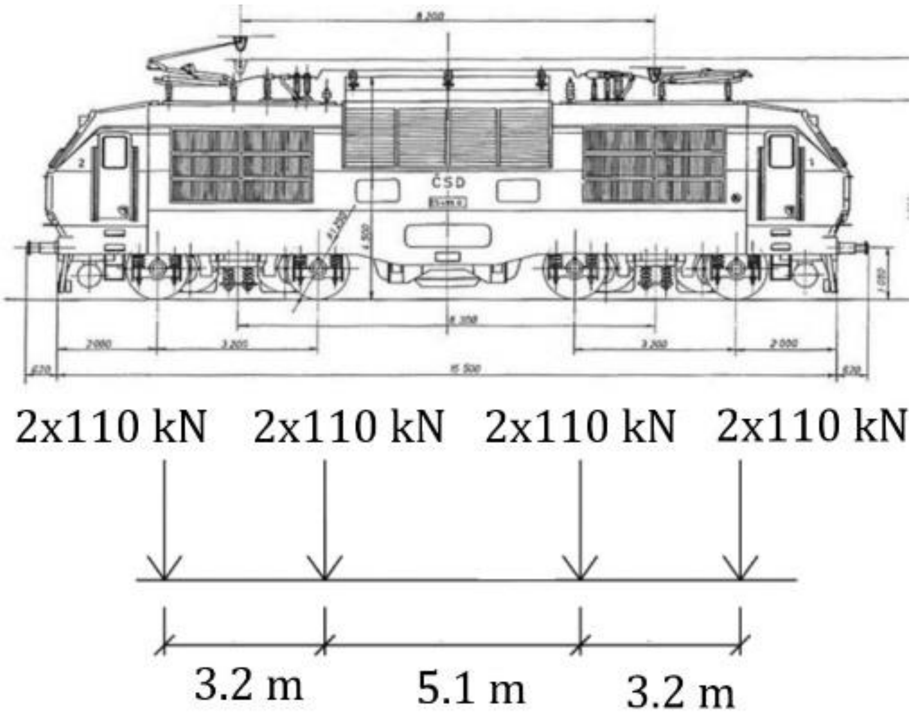


Fig. 3. A scheme of loading forces from locomotive (type 350)

These values were then compared with peak values of stresses gained from strain data. Finally, a performance indicator (PI) is expressed as:

$$PI = \frac{\sigma_{max,calc}}{\sigma_{max,test}} = \frac{\frac{F_a}{2} \cdot (\eta_1 + \eta_2)}{\varepsilon_{max,test} \cdot E} \tag{1}$$

where the numerator is a peak value of normal stress calculated from influence lines and the denominator is a peak value of normal stress obtained from the measurement. Quantity $F_a/2$ refers to load on a wheel, η_1 and η_2 are ordinates of influence line in a location of axles (Fig. 4). Measured strain values $\varepsilon_{max,test}$ are multiplied by Young's modulus E to acquire stress values. The result of this division is dimensionless and it includes the influence of different axle load and wheelbase of passing trains.

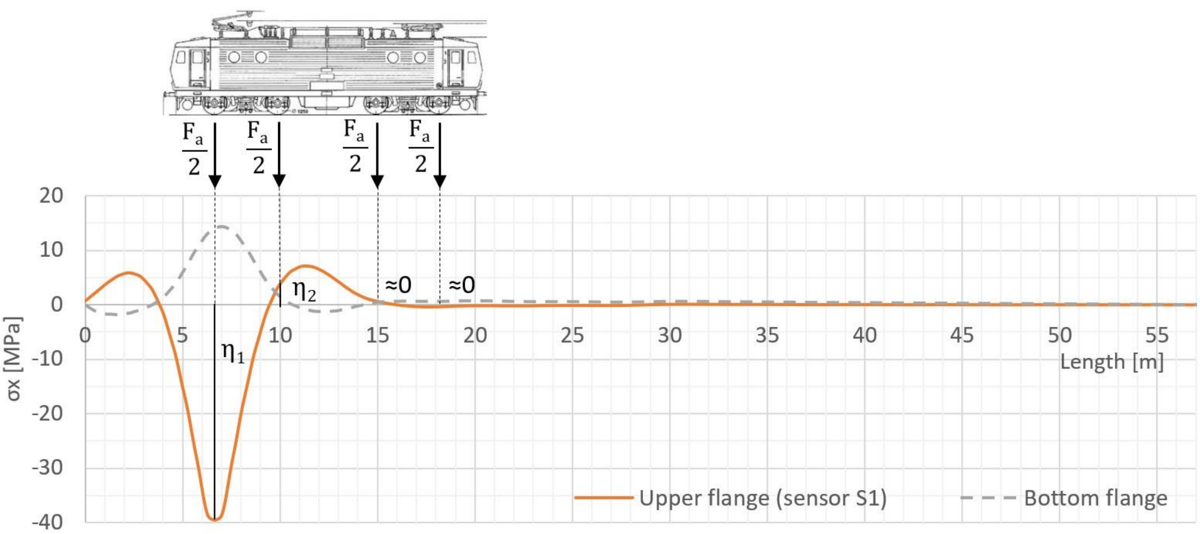


Fig. 4. Influence lines of normal stress for the location of sensor S1 (bridge T2-O2) and its evaluation

The value of the proposed PI was evaluated and monitored regularly, after each testing campaign. It was assumed that a significant decrease in its value would occur in case of substantial damage of selected stringers and floorbeams. Ideally, in undamaged case, the value of PI should be close to 1. However, it requires a complex, precisely calibrated FE model that accurately represents real behavior of a structure. As seen in Table 2 presenting the results of regular measurements for one of the sensors, the PI takes values close to 1. This proves, that such model was successfully prepared.

Table 2. Evaluation of strain measurements - sensor S1 (bridge T2-O2)

Test no.	Train no.	Type of locomotive	Wheelbase [m]	Axel weight [t]	Velocity [km/h]	PI [-]
1	R607	361	3.2	21	N/A	**1,03**
2	IC523	350	3.2	22.4	81	**1,03**
3	R953	757	2.4	18.85	75	**0,96**
4	IC523	383	3.0	22.25	78	**1,03**
5	IC523	350	3.2	22.4	N/A	**1,01**

5 Calculated vs. measured strain time histories

Moreover, a program was prepared to automatically calculate strain time history using influence lines obtained from the precisely calibrated FE model. The axle loads, distances of axles, a velocity of a train and time step for calculation were needed as input parameters. The velocity of passing trains was evaluated from video recording and confirmed with the calculation from strain time histories. Therefore, not just peak stress values were calculated to compare with measurements, but also the complete strain time histories. As seen in Table 3 and Table 4, the agreement between calculated and measured time histories is within engineering accuracy and deviations are small. These time histories can be used in the future as a basis for evaluation of other features, which might be sensitive to local damage.

Table 3. Measured strain time histories - sensor S1 (bridge T2-O2)

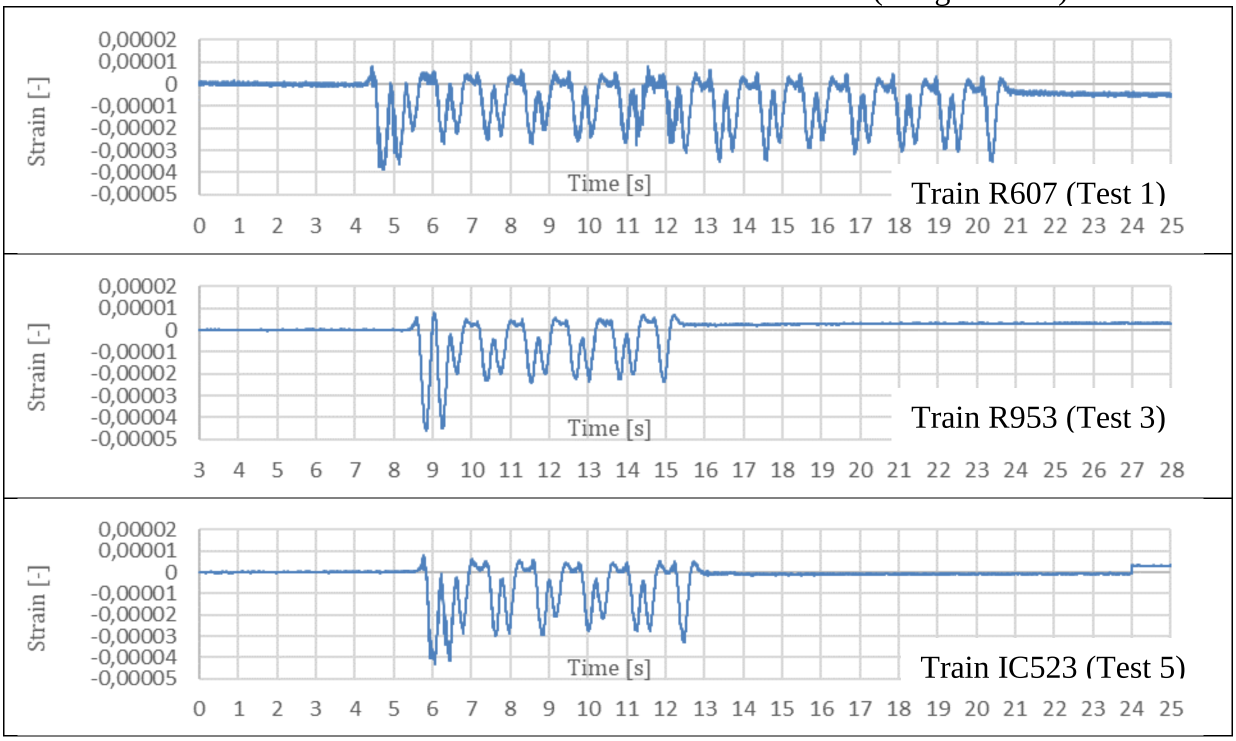
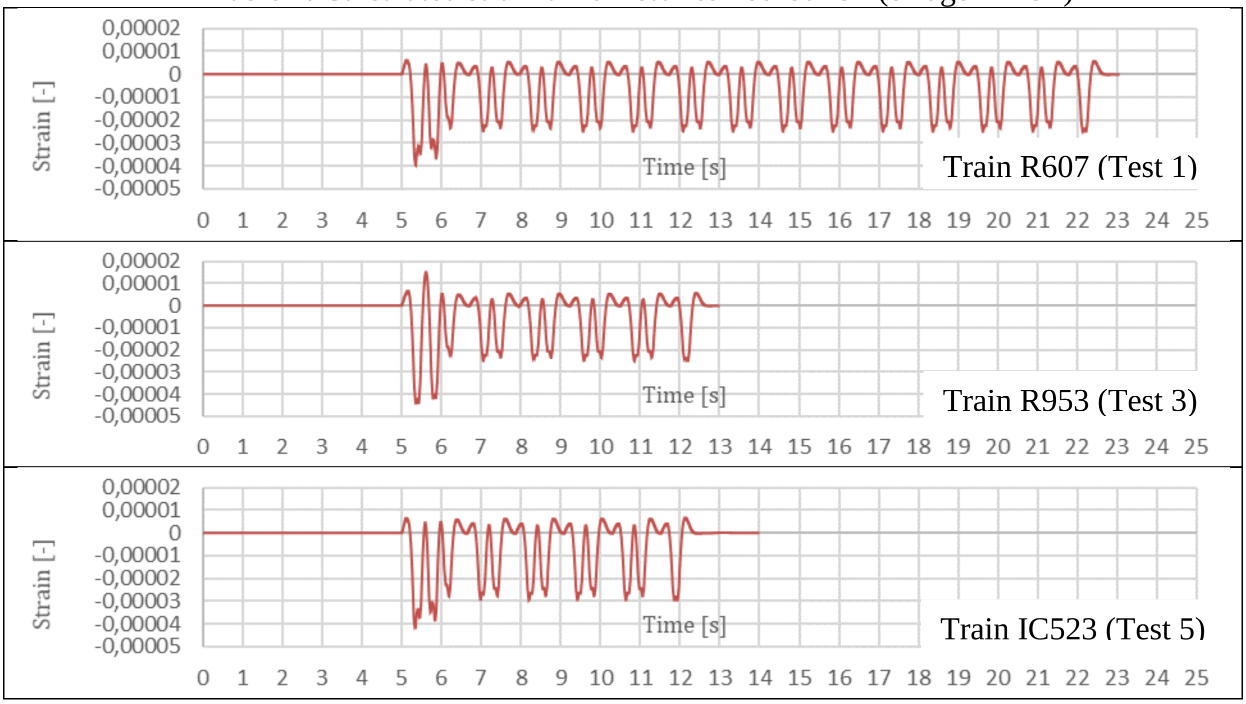


Table 4. Calculated strain time histories - sensor S1 (bridge T2-O2)



CONCLUSION

Measuring strains allowed us to get an overview of the response of local structural elements caused by passing trains. After initial tests with accelerometers and radar interferometry, a detailed FE model was updated and verified. Slight changes and inaccuracies of the model were revealed, which would not be possible without in-situ measurements.

The performance indicator was proposed by comparing calculations with strain measurements. It allowed us to compare measured time histories from regular testing campaigns and monitor potential changes in the structural response of selected stringers and floorbeams. Also, calculating strain time histories using influential lines is not as time-consuming as FEM calculations and can be adjusted for any type of train vehicle. Another benefit is, that this method is rather accurate, because influence lines are evaluated using a precisely calibrated FE model.

The resulting comparisons show good agreement between calculations and measurements. The methodology for monitoring local elements was implemented. In the future, presented practical knowledge and methodology could be used for structural health monitoring. However, it would be beneficial to use additional sensors to obtain a response from more structural elements on the bridge. Also, further research should be aimed at the evaluation of the proposed performance indicator on damaged structures to define relevant threshold values.

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