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Performance assessment of a concrete railway bridge by diagnostic load testing

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Abstract

Diagnostic load testing can be a suitable method for condition assessment of existing bridges. This paper presents the evaluation of a concrete railway bridge built in the 1940s based on diagnostic load testing. The test was the outcome of a visual inspection and preliminary theoretical analysis that found the bridge to be in unsatisfactory condition. Unlike the other possibilities that infrastructure management bodies have in such situations, for example, structure replacement, traffic reorganization, closure of the bridge, etc., diagnostic load testing is a low cost and quick method that indicates the bridge response to real traffic load. The paper describes the test plan and implementation covering the static and dynamic load tests. The experimental methods comprise superstructure displacements measurement, strain measurement on concrete and reinforcement and determination of bridge dynamic parameters. The obtained test results analysis enabled more accurate diagnosis of the actual bridge condition. The bridge demonstrated satisfactory load bearing capacity and response to real service load in terms of registered levels of displacement and strain and proper dynamic performance.

KEYWORDS

diagnostic load testing, displacements, dynamic coefficient, dynamic tests, material properties, mode shapes, natural frequencies, railway bridge, strain

1 | INTRODUCTION

There are many strategies and tools developed for monitoring and quantitatively assessing the bridge performance.[1] One of the solutions is to perform an assessment of the bridge current status by using load testing.[2,3] Load testing implies non-destructive field testing of a bridge and is the most suitable technique for the assessment of its actual load bearing capabilities.[4] Generally, there are two types of load tests and the concerned bridge needs to be closed to traffic during their execution. Diagnostic load test[5] is conducted in order to compare the actual bridge behavior with the calculation models which can then be updated accordingly.[6] This type of load test can be used to check various bridge behavior parameters, for example, transversal distribution of load, element stiffness, strengthening efficiency, boundary conditions, etc.[7] Proof load test[8] implies higher load levels than diagnostic load and is used to obtain the maximum load capacity of a bridge with a required safety factor. This load is usually compared with the loads prescribed by the concerning code. Diagnostic load testing is primarily applicable to bridges of simpler structural systems, with known

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material properties, geometry and reinforcement plans. In contrast, proof load testing is applied in the case of bridges with significant uncertainties in this aspect.[9] Both types of load tests comprise several stages: preliminary inspection and assessment of the structure concerned, load level and layout determination accompanied by structural analysis, sensor type and layout specification, loading protocol definition, test data analysis during the test and after the test with possible updating of the used structural model.[10]

It is well known that many bridges worldwide are structurally deficient and need rehabilitation. The survey done in the frame of the project Mainline in 2015 indicated that 6,000 bridges in Europe need for strengthening or replacement.[11] However, it is believed, that a significant number of those bridges have hidden structural capacity and can carry higher loads than predicted by calculation.[12] Namely, the fact that a bridge is poorly assessed based on detailed visual inspection does not necessarily mean that the bridge is in danger of collapsing.

To confirm previous considerations, here is given a case study in Croatia about performance assessment of a skewed concrete railway bridge. The bridge owner decided to evaluate the bridge condition with the objective to increase its category due to many years of higher axle loading demand. The first step of assessment consists of visual inspection of the bridge in combination with some nondestructive tool techniques. Visual inspection of the railway bridge in combination with preliminary calculation according to current standards resulted with the decision that the bridge is in poor condition and it should be replaced with a new one. This opinion is mainly based on the deterioration of concrete such as spalling, leaking, and cracking (Figure 1). Furthermore, due to the aggressive marine environment, the chlorides were already embedded within the concrete and significant cross section loss of reinforcement was caused by corrosion. Due to the importance of the railway network and the financial constraints, it was concluded that reliable and applicable methods should be found for the assessment of bridge performance. Therefore, the bridge was subjected to a diagnostic loading test accompanied with numerical analysis. The main parameters in the bridge evaluation were data collected during static load tests such as deflections and strains. Dynamic load test resulted with inherent dynamic properties of the bridge

FIGURE 1 Concrete spalling caused by reinforcement corrosion: (a) Girder flange, (b) Girder web, (c) Slab, (d) Column

such as natural frequencies and mode shapes. Additionally, the assessment was done based on the dynamic response of the bridge induced by a passing locomotive. The positive outcome of this type of test was assessed as capable of postponing the bridge replacement with a new one for some time. Namely, this international corridor was planned to be reconstructed entirely within 10 years from the described moment.

2 | BRIDGE DESCRIPTION

The railway bridge "3. Maj" is part of Croatian railway network M502 Rijeka–state border to Slovenia (Figure 2). The bridge is crossing Liburnijska street with four traffic lanes (two in each direction) and two pedestrian lines. The bridge is not oriented perpendicular to the traffic lines, the skew angle amounts 19.1°. The bridge was built in 1947. Reinforced concrete bridge is statically indeterminate system and it consists of four spans (10.0 m + 23.7 m + 23.7 m + 10.0 m). The superstructure of the bridge is integral "U" shaped cross section with a total height of 2.5 m and width of 6.7 m. Design of the load bearing structure is an integral connection between superstructure and supports. Each support consists of two circular piers with a diameter of 100 cm. Single track railroad sleepers are laid on ballast with a depth of 0.4 m. Figure 3 shows the bridge geometry.

3 | LOAD TEST PROGRAM

Bridge displacements during the load test were measured with modified geometric leveling method in 18 relevant locations, that is, in the middle of the spans and above columns and abutments (Figure 4a). These locations were considered to have the highest displacement values during the test and were also most viable in terms of practicality of measurement. Due to symmetry conditions, strains were measured in two western spans in 16 locations with LVDT sensors (measurement range ±3 mm, sensitivity of 1,000 mV/mm) (Figure 4b).

The sensors were positioned in the outer boundary of compression and tension zones in the middle and quarter of the spans and in the columns area. Furthermore, two sensors were located in the southern S1 column area to measure the shear strain. Where possible, the sensors were positioned on reinforcement bars in order to capture strains unbiased by local concrete irregularities like initial microcracks for instance. Modal parameters of the bridge were determined based on acceleration measurements conducted in the two central spans under ambient excitation. The accelerometers (Brüel&Kjær 4508, nominal amplitude range of 70 g, sensitivity of 100 mV/g and frequency range of 0.3 Hz to 8 kHz) were glued on top of the girders in the middle and quarters of the western span and in the middle of the eastern span (Figure 5).

Accelerations were recorded using Brüel&Kjær 5-channel portable data acquisition system type 3,560 C. The sampling rate used for the testing was 1 kHz. Dynamic displacements were measured with vibrometers (type: HBM SMU 31) on two locations in the middle of the western central span (Figure 5). For the load test, the diesel-electric locomotive of type EMD GT26CW-2 and the total weight of 1,200 kN was used (Figure 6). In the original design, a composition consisting of a steam locomotive, tender and a wagon was used as a traffic load (Figure 7). In the static test, the locomotive firstly passed over the bridge at crawl speed in both directions. This enables to record maximum values of all the observed displacements and strains in a single record. Then, the locomotive was positioned in relevant locations to

FIGURE 2 Reinforced concrete railway bridge "3. Maj"; (a) side view, (b) top view

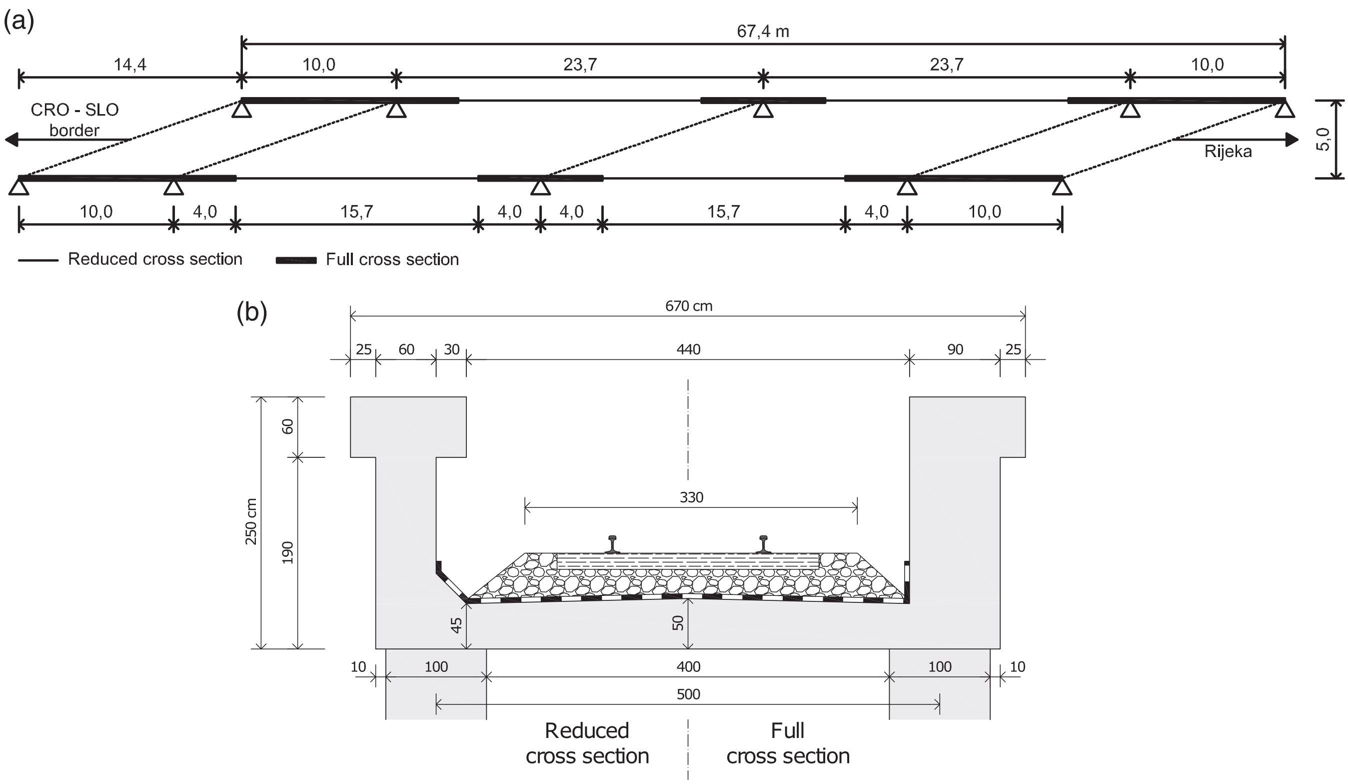

FIGURE 3 Superstructure of the bridge; (a) top view, (b) cross section

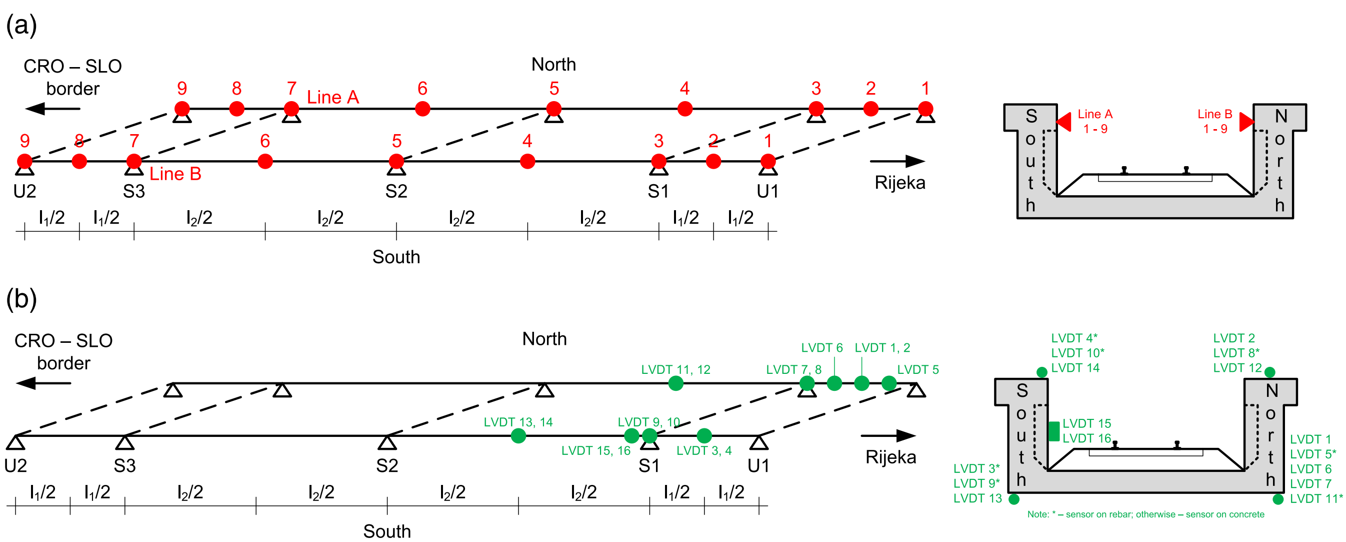

FIGURE 4 Measurement locations for displacements and strains; (a) displacement points, top view and cross section, (b) measurement points for strains, top view and cross section

produce maximum bending moments in the northern and southern main girders of each span (Table 1, Figures 8 and 9). During these phases, the stability of measured strain under constant load can be observed. In the dynamic test, the locomotive passed over the bridge at speeds of 20, 40, and 60 km/h and the bridge response in terms of all the previously described parameters were recorded. Recording of the data started before the locomotive had reached the bridge and continued for a while after it had passed over the structure.

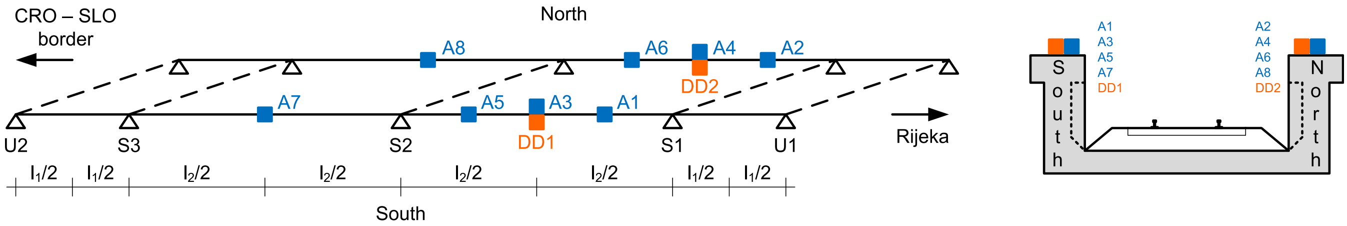

FIGURE 5 Measurement positions for accelerations and dynamic displacements

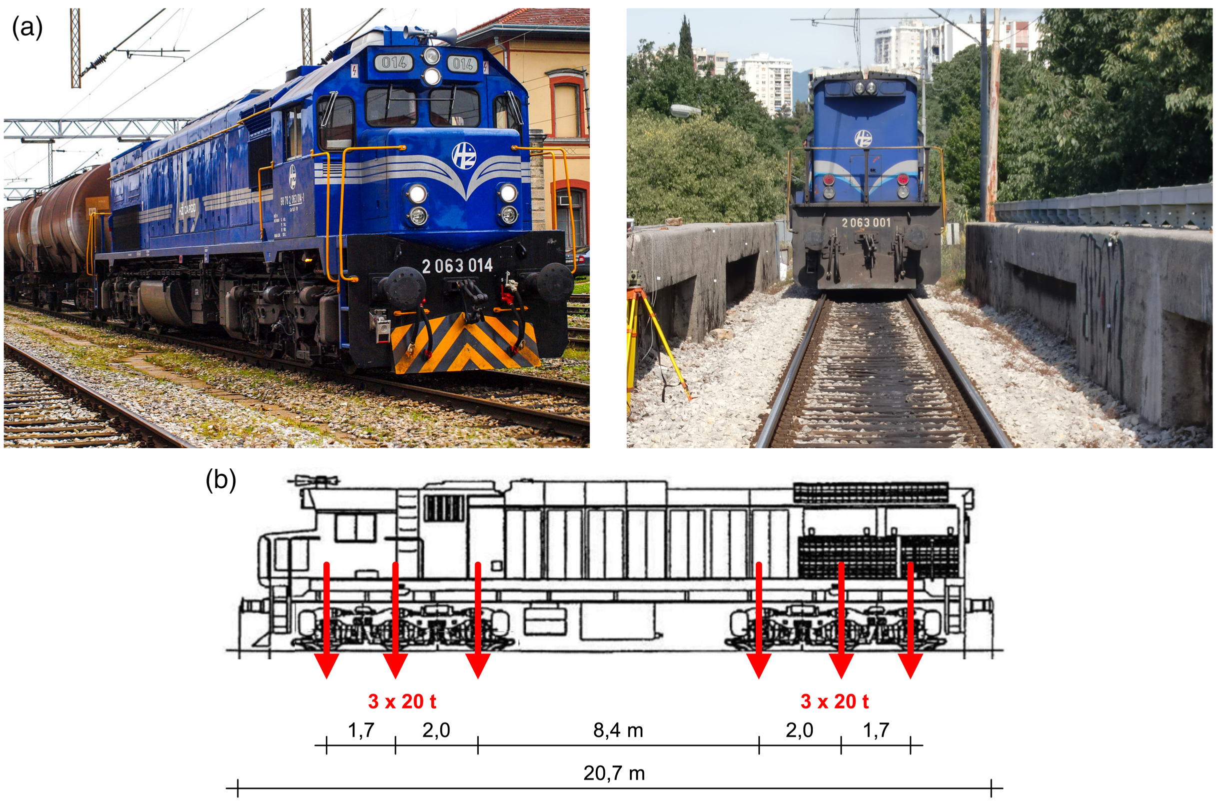

FIGURE 6 Locomotive used for the load test; (a) diesel-electric locomotive of type EMD GT26CW-2, (b) locomotive axle loads and locations

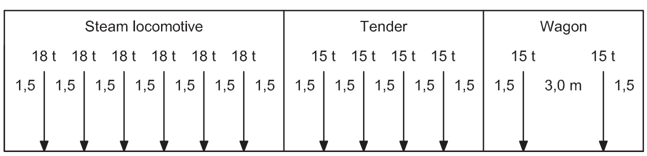

FIGURE 7 Original design traffic load with a steam locomotive, tender and wagon loads

4 | MATERIAL PROPERTIES

To determine the quality of concrete, cylindrical cores were taken from the main girders at five locations (Figure 10). The obtained compressive strengths and densities are shown in Table 2.

Based on the experimentally obtained concrete compressive strengths and the calculated mean value, concrete modulus of elasticity was determined by using the following expression (1) from Eurocode 2[13]:

$$E_{cm} = 22 \times (f_{cm}/10)^{0.3} \times 1{,}000 \tag{1}$$

The exact value obtained by the above expression was 32,303 MPa and the value of 32,000 MPa was used in the numerical simulations.

TABLE 1 Loading phases for the static load test of the bridge

Phase no.	Load phase description
1	Bridge unloaded
2	Locomotive positioned for M_{max} in northern girder in span U1–S1
3	Locomotive positioned for M_{max} in southern girder in span U1–S1
4	Bridge unloaded
5	Locomotive positioned for M_{max} in northern girder in span S1–S2
6	Locomotive positioned for M_{max} in southern girder in span S1–S2
7	Bridge unloaded
8	Locomotive positioned for M_{max} in northern girder in span S2–S3
9	Locomotive positioned for M_{max} in southern girder in span S2–S3
10	Bridge unloaded

5 | NUMERICAL MODEL

For the preliminary investigation and the load testing program, a simplified model was created by using Scia Engineer software. This model was built based on available documentation, which was very poor and including only geometrical properties. The columns and the superstructure of the bridge are modeled as reinforced concrete with Young's modulus of 35,000 MPa, a Poisson's ratio of 0.2 with compressive strength of 35 MPa and material density 2,500 kg/m^3. The steel reinforcement is assigned to cross-sections with a Young modulus of 210,000 MPa, a Poisson ratio of 0.3, and a material density value of 7,850 kg/m^3. All columns are modeled as 1d beam elements with fixed boundary conditions. The superstructure of the bridge is modeled as 4-node shell elements with previously mentioned materials properties. The bearings at abutments of the superstructure are fixed in both horizontal directions. Both material and geometrical properties were included as a linear analysis of a model, simulating a global static and dynamic behavior of the

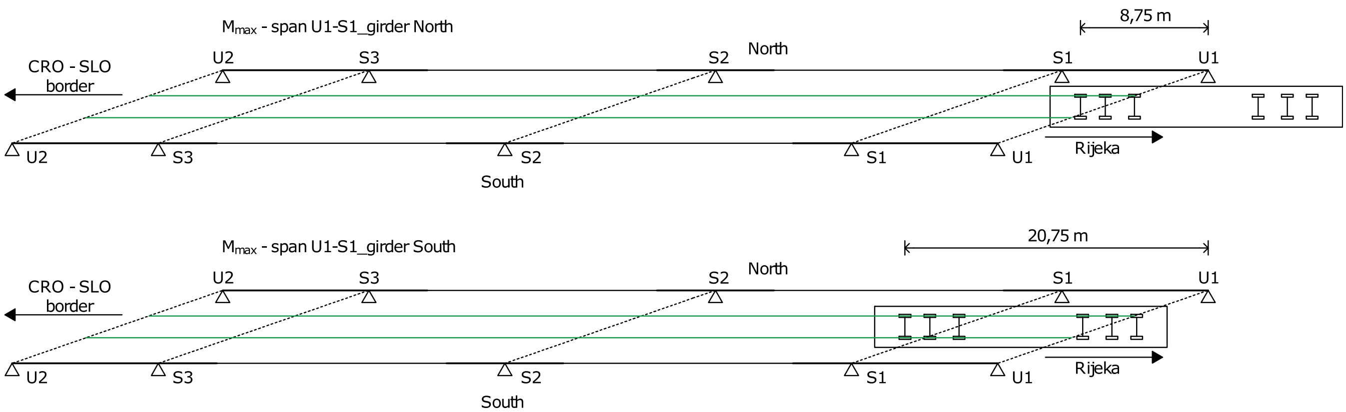

FIGURE 8 Position of the locomotive in loading phases 2 and 3

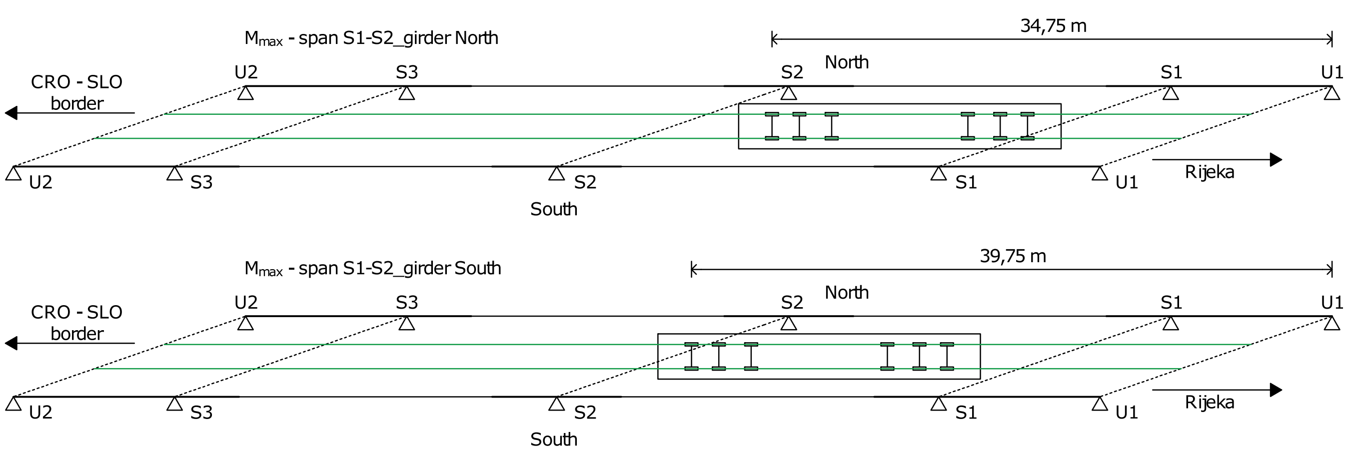

FIGURE 9 Position of the locomotive in loading phases 5 and 6

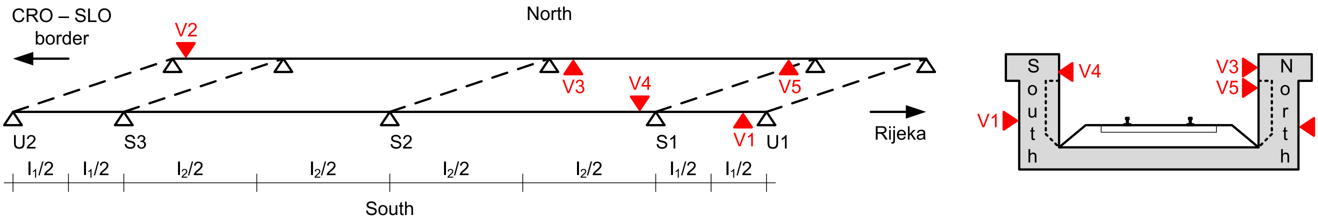

FIGURE 10 Locations of drilled concrete cores

TABLE 2 Concrete properties obtained from drilled cores

Spec. ID	Compressive strength (MPa)	Density (kg/m^3)
V1	32.5	2,336
V2	30.2	2,325
V3	46.3	2,300
V4	38.1	2,332
V5	32.8	2,377
Mean	35.98	2,334
SD	6.45	27.81
CV	17.94%	1.19%

bridge. The simplified model was created before any site investigations. This model was used for calculating the load efficiency by comparing the influence of the selected locomotive used in the static test and the design load model. The traffic load on the bridge is on the line carrying heavy rail traffic, therefore, Load Model SW/2 was selected for the design load model. The maximum level of load efficiency[14] according to bending moment for the main spans was 40%. For railway bridges, generally, it is very hard to achieve load level greater than 50% of the characteristic live load present in design codes.

After performing the diagnostic load test and additional tests regarding the material properties of the bridge, a new model was developed based on the previous one. The dynamic behavior of the structure can be influenced by stiffness, material properties, boundary conditions, and mass of the structure. Material properties are assigned to an updated model from the observed test on concrete (Table 2). Further, the mass of the structure is updated by additional simulating ballast support and track as an added mass in the model. The mass of the structure and boundary conditions are updated based on the experimentally measured dynamic properties of structure (natural frequencies, mode shapes). The stiffness and geometrical properties assigned to the model were considered as invariant. The mass of the structure and boundary conditions were varied iteratively. Those parameters were compared based on the numerical model and the experimental data, taking into account the first three natural frequencies. The acceptable criterion is considered as a difference between the measured and analytical deflection values within ±15% for reinforced concrete bridges. Vertical loads for Load Model SW/2 and locomotive are simulated as equivalent vertical loading uniformly distributed per area considering distribution through ballast (Figure 11). Values of displacement and strains are presented as envelope representation given from numerical model loaded with locomotive and Load model SW/2 (Figure 12).

Natural frequencies and modal shapes were also considered in numerical analysis and results are presented in Figure 13. Torsion shapes are dominated due to heavy main girders and very skew type of bridge.

6 | RESULTS AND DISCUSSION OF STATIC TESTS

6.1 | Displacement measurement

The following Table 3 shows deflections measured in spans S1–S2 and S2–S3 during the respective phases of static load test and compared with those from the numerical model. Displacements measured in side spans were under the measurement accuracy (approx. 0.2 mm) and therefore are not considered in this analysis. The presented results confirm that the calibration of the numerical model was done accurately, regarding the stiffness of the structure.

6.2 | Strain measurement

The strains were measured continuously during all phases of the static load test. The following Figure 14 shows strains record in relevant middle span S1–S2 while the locomotive was slowly passing (crawl speed) over the bridge in the direction from east to west. Figure 15 presents registered strains during maximum loading phases. According to recordings, it can be recognized that strains are crossing from negative to positive values (and vice versa) due to changes of negative and positive bending moment induced by the locomotive position.

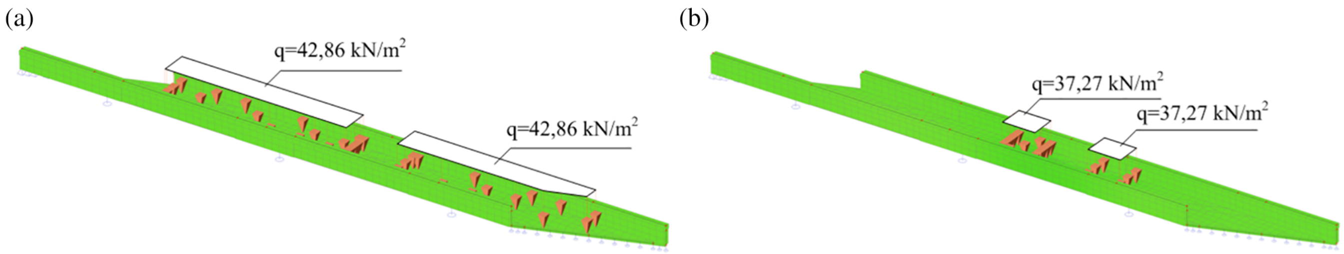

FIGURE 11 Numerical model of bridge with loadings; (a) Load Model SW/2, (b) Locomotive

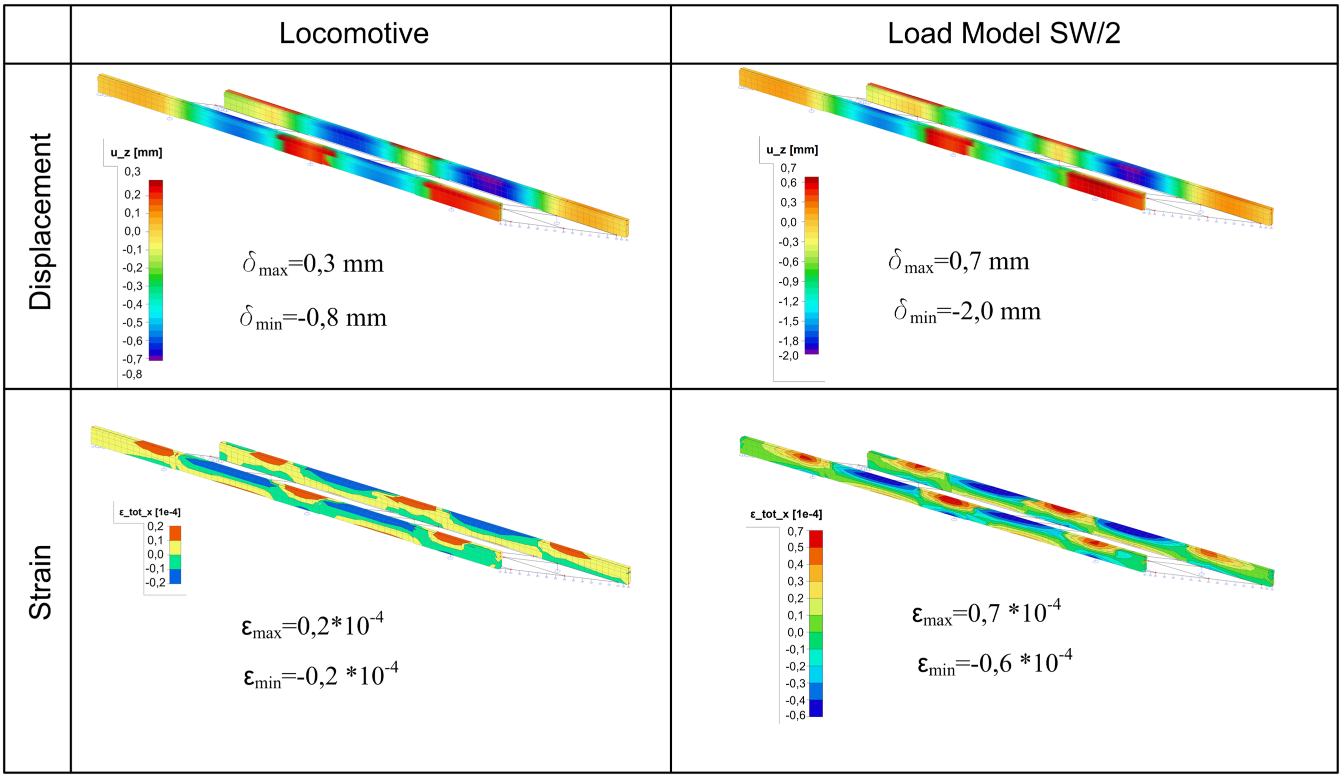

FIGURE 12 Envelope representation of displacements and strains from numerical model loaded with locomotive and Load model SW/2 (Table 1)

Table 4 presents comparison between the strains on concrete obtained experimentally and the ones computed with the calibrated numerical model. Differences are acceptable (approx. 5%) and that proves that the calibrated model fits the field observations.

7 | RESULTS AND DISCUSSION OF DYNAMIC TESTS

Apart from the static tests, diagnostic load testing consists of additional dynamic testing. These tests are used to evaluate the dynamic performance of the structure and they can serve as a measure of stiffness and mass of the structure. Experimentally determined dynamic properties of the bridge can be used for the update of the structural model together with static tests. Furthermore, another application of these data may be used for damage assessment of the structure over time. Therefore, it is essential to determine dynamic properties of the bridge at an early age so that afterward they can be observed periodically or continuously by using the structural health monitoring system.[15] Variation of dynamic properties over time may be significantly caused by changes in temperature[16] and humidity. This effect depends on the structural system and it has to be considered while evaluating the results from dynamic tests.

Extensive field testing was performed in order to assess the dynamic parameters of the superstructure. In the first phase, main dynamic parameters (i.e., eigenfrequencies, modal shapes, damping ratios) are determined by means of operational modal analysis

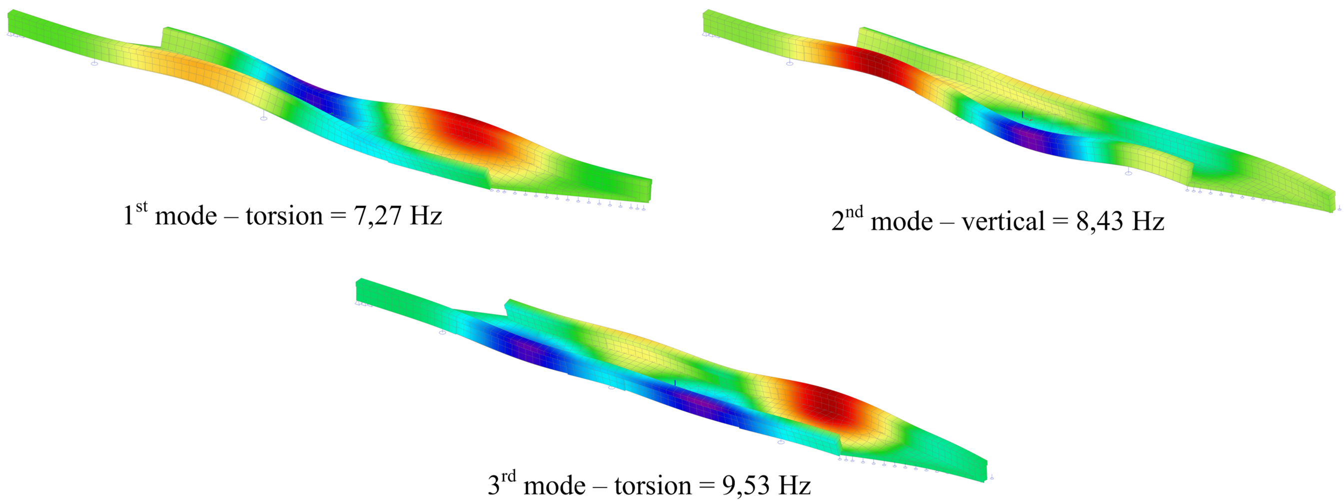

FIGURE 13 Numerically estimated natural frequencies and mode shapes

TABLE 3 Maximum measured deflections during the phases of the static load test

Load phase no. (see Table 1)	Measurement location	Measured deflection (mm)	Calculated deflection from numerical model (mm)
5	4A	0.6	0.65
6	4B	0.7	0.75
8	6A	0.8	0.80
9	6B	0.6	0.60

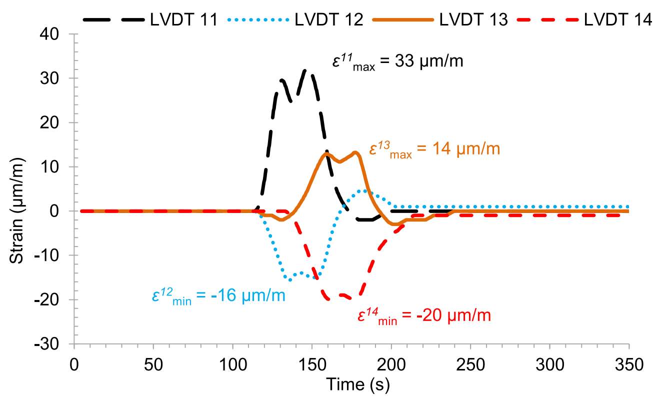

FIGURE 14 Strains record in span S1–S2 during locomotive drive over the bridge in direction east–west

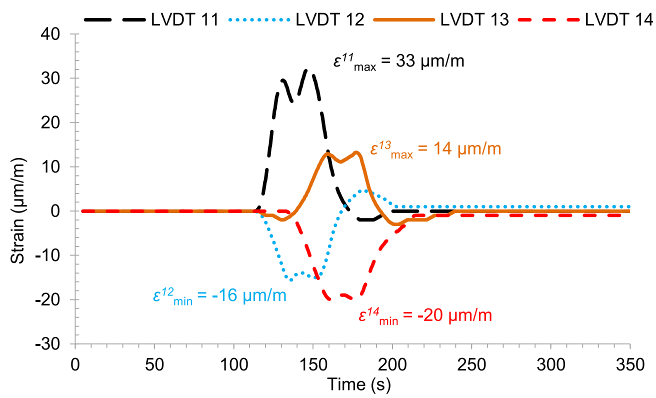

FIGURE 15 Strains record during phases 2, 3, 5, and 6

(OMA). In the second phase, the increments of dynamic displacements and strains caused by locomotive passing over the bridge at different speeds are measured.

7.1 | Identification of dynamic properties

Operational modal analysis (OMA) uses ambient environmental and traffic excitation and there is no need for controlled dynamic excitation of the structure.[17] Unmeasured and uncontrolled ambient excitation is assumed to have the characteristics of the Gaussian white noise process. This simplifies the testing procedure, especially for CE structures as only response measurement are required for determination of natural frequencies, modal shapes and damping ratios.

Identification of natural frequencies, modal shapes and damping ratios was conducted using the methods of frequency domain decomposition (FDD). The procedure is based on singular value decomposition (SVD) of Power

TABLE 4 Maximum measured strains in concrete during phases of static load test

Strain location	Load phase no. (see Table 1) with maximum strain	Measured strain (μm/m)	Calculated strain from numerical model (μm/m)
LVDT 12	Phase 5	−17	−17.9
LVDT 13	Phase 6	14	14.8
LVDT 14	Phase 6	−19	−20.0

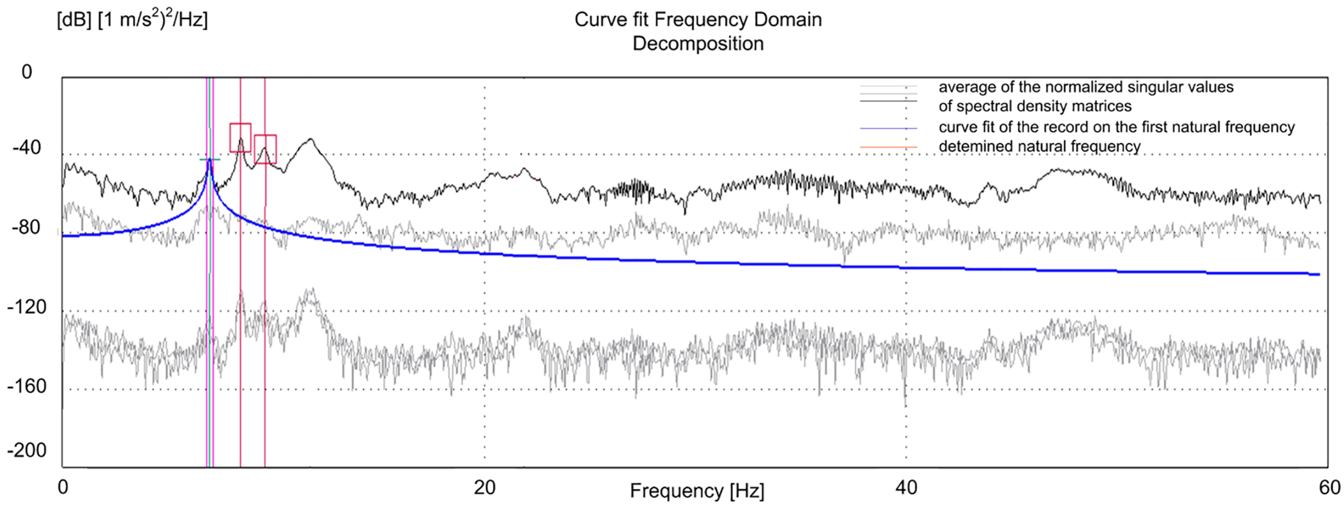

FIGURE 16 Spectral recording of curve fit frequency domain decomposition

Spectral Density (PSD) matrix of the measured responses. We have to assume that the loading is white noise process, the structure is lightly damped, and close mode shapes are geometrically orthogonal.[18,19]

Results of experimentally determined natural frequencies are shown in Figure 16 and together with results of experimentally determined damping ratios are presented in Table 5. Determined mode shapes of the second and third span are shown in Figure 17.

7.2 | Dynamic displacements analysis

Train speed is the most important parameter influencing the dynamic stresses in a railway bridge. In order to estimate the dynamic displacements, the same locomotive passed over the bridge with three different speeds with a constant velocity of motion. Based on vertical dynamic displacement recordings, the dynamic coefficient is calculated as the ratio of dynamic displacement and static displacement, according to the equation:

$$\varphi = \frac{y_{\mathrm{din}} + y_{\mathrm{st}}}{y_{\mathrm{st}}} \tag{2}$$

Results of experimentally determined dynamic coefficients together with the calculated value based on Equation (2) are presented in Table 6. Dynamic displacement was measured by using vibrometer displacement instrument (HBM SMU31) and the digital oscilloscope connected to a computer. Figure 18a shows the variation of vertical displacement under the moving load at the speed of 40 km/h and horizontal displacement during braking. It may be observed for vertical displacement, that when the load reaches to the southern girder and then to the northern girder fast fluctuation with relatively large amplitude occurs. As the locomotive passes the bridge, fluctuations disappear very soon. Such characteristic was observed in all dynamic tests under moving train.

Experimentally detected dynamic coefficients are compared to those one defined in Eurocode[13] for the track with standard maintenance ($\emptyset_3$). It can be calculated according to equation:

$$\emptyset_3 = \frac{2.16}{\sqrt{L_\emptyset} - 0.2} + 0.73 \tag{3}$$

where $L_\emptyset$ is the determinant length in (m) according to Table 6 in Eurocode 2.[13]

Furthermore, at the same locations horizontal longitudinal displacement of the structure was measured during the locomotive braking. Measured maximum horizontal displacement was 0.13 mm for locomotive

TABLE 5 Experimentally determined natural frequencies and damping ratios

No.	Mode shape	Natural frequency (Hz)	Standard deviation (Hz)	Damping ratio (%)	Standard deviation (%)	Natural frequency from numerical model (Hz)
1	Torsion	6.98	0.02	0.76	0.31	7.27
2	Vertical	8.45	0.01	0.65	0.14	8.43
3	Torsion	9.57	0.01	1.02	0.47	9.53

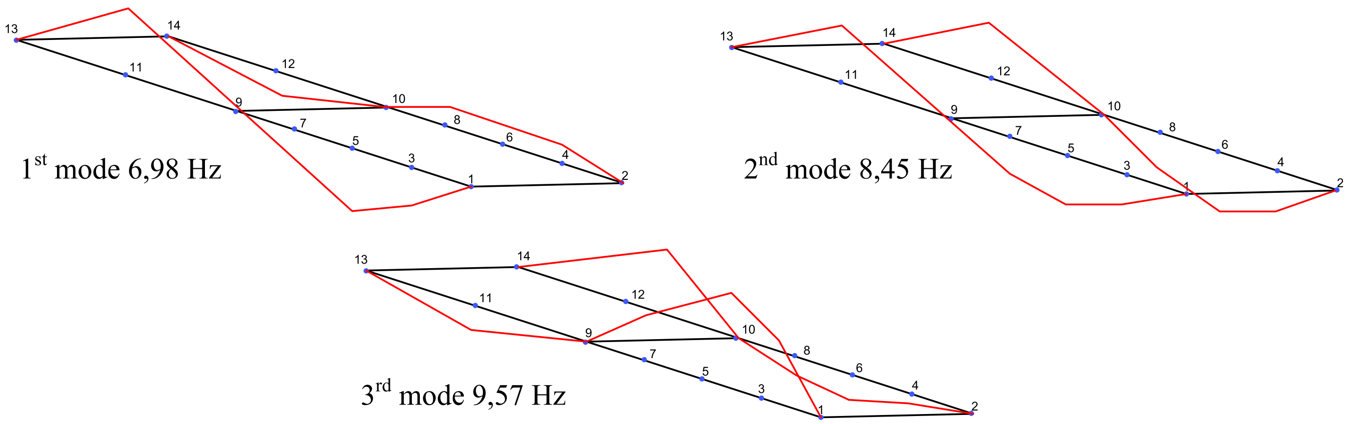

FIGURE 17 Experimentally obtained modal shapes of the bridge

TABLE 6 Experimentally determined dynamic coefficients

Speed (km/h)	Measured dynamic displacement y_{din} (mm)	Dynamic coefficient φ (Equation (2))	Calculated value based on Equation (3)
20	0.023 (north girder)	1.038	
40	0.050 (south girder)	1.071	1.19
60	0.072 (south girder)	1,103	

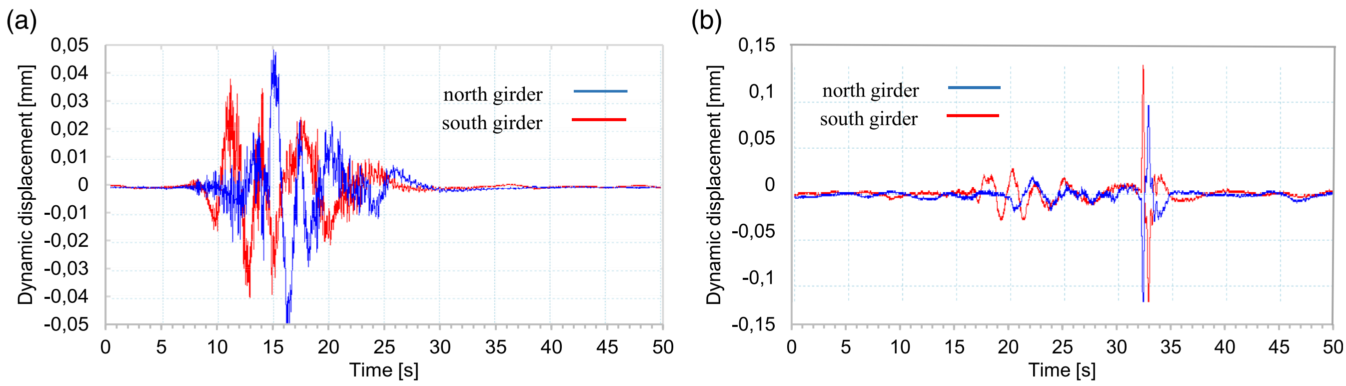

FIGURE 18 Record of dynamic displacement versus time during locomotive passage and braking; (a) vertical displacement during locomotive passage at a speed of 40 km/h, (b) horizontal longitudinal displacement during braking

braking starting from a maximum speed of 40 km/h. The characteristic feature of horizontal longitudinal forces during braking is presented on the Figure 18b, and it shows that the displacement slowly increases at the beginning and it has sudden drops at the moment of stopping. On the basis of experiments, the magnitude of the horizontal longitudinal force in the bridge and rails can be calculated as[20]:

$$H = \gamma_f \mu_0 F \tag{4}$$

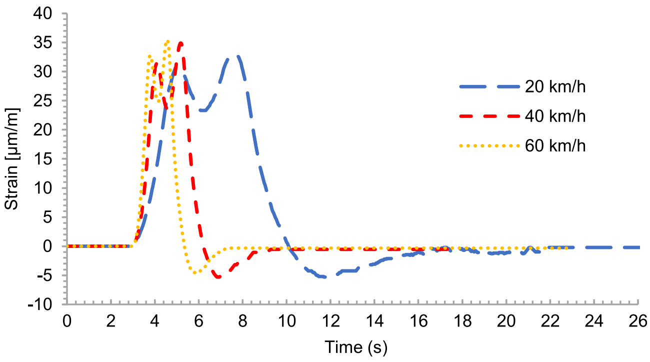

FIGURE 19 Registered strain for LVDT 11 in time during the dynamic load test for three different speeds

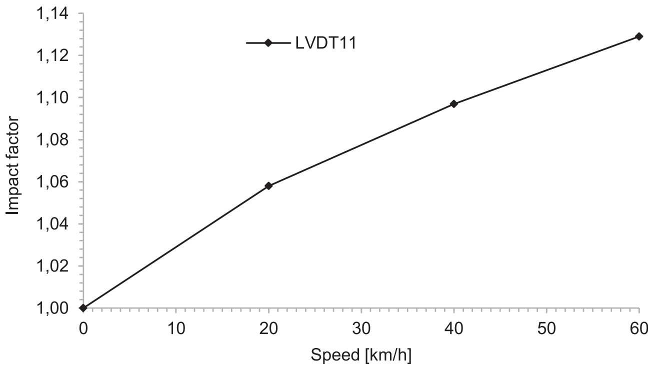

FIGURE 20 Impact factor for LVDT 11

where γ_f is the load factor 1.3; μ_0 is the coefficient of adhesion (0.2 for braking); F is the sum of vertical forces. According to Equation (4), the magnitude of the horizontal longitudinal force equals to 310 kN.

7.3 | Dynamic stress analysis

Strains were recorded for a locomotive crossing the bridge with three different speeds with a constant velocity of motion in order to assess the response of the superstructure. Dynamic stress analysis was focused on the maximal measured strain—LVDT 11 positioned on the reinforcement at the middle of the span S1–S2. Recordings of strains induced by a locomotive for different speed is shown in Figure 19. Extreme peaks were caused by two groups of three axles. A positive value represents positive bending moment when the locomotive is in the mid-span S1–S2, a negative value is caused by the location of the locomotive in the adjacent span causing negative bending moment in the observed span.

In the dynamic test, the results of stress were analyzed as an impact factor (relative dynamic effect, % at different speeds. This factor mainly depends on the static system, length of the span, boundary conditions, material properties and load properties. The maximum strain for LVDT 11 at different speed was normalized by the maximum strain at the same location obtained due to the static test of the bridge (Figure 20).

8 | BRIDGE ASSESSMENT

After performing the diagnostic load test, it is necessary to evaluate the bridge according to the measured data, numerical analysis and standards. It has been shown that during the diagnostic load test the bridge behaved without signs of distress. Furthermore, in each stage displacement increments stabilized relatively fast and there was no permanent displacement value observed. As presented, the difference between the field measurements (strains and displacements) are minimized by updating the numerical model and results are within 10%. Thanks to model updating, it is possible to compare the results of any load model with standards. Therefore, the maximum calculated deflection form Load model SW/2 is 2.0 mm which is equivalent to the value of l/11,850, where l represents the observed span. According to UIC standard, the recommended value of maximum deflection for the maintained railway is l/2,600. This means that in term of stiffness, the bridge has enough capacity to withstand live load. Furthermore, results of measured strains on reinforcement and concrete confirm that the bridge behaves in an elastic manner. According to the field test and the presented results, effect of the speed on the dynamic coefficient (displacement and strain) on the bridge is increasing. Generally, the dynamic effect for displacement and strain increase with increasing speed up to some level, and afterward decrease with the higher speed. It can occur, that dynamic effect increases constantly which can be caused by track irregularities. The calculated dynamic coefficient value is greater than the experimentally measured. These results confirm that the structure behaves in good manner during dynamic tests. It was demonstrated that the bridge behavior is still valid for operational loads and bridge fulfills the standards with regard to prescribed live loads. However, as presented in the introduction, the bridge is faced with durability problems such as corrosion of reinforcement and delamination of the concrete cover. Regardless of sufficient carrying capacity, it is required to perform repair measures as soon as possible.

9 | CONCLUSIONS

This paper presents a diagnostic static and dynamic load test on a concrete railway bridge built in the 1940s. The test load of the static test consisted of the locomotive with known weight and axle configuration, which was placed at pre-selected locations to obtain maximum load effects. During the static load tests, displacement and strains were measured in relevant cross sections. Additional to static tests, the dynamic testing was used to evaluate the performance of the structure. Free vibrations of the structure were used to identify the dynamic properties of the bridge. Furthermore, dynamic factors (displacement, strain) were observed due to locomotive passing over the bridge. Finally, a numerical model was updated based on the experimental research. Diagnostic load test is here presented as a useful tool to get precise information about the realistic behavior of a bridge. Although the visual inspection indicated that the bridge is in poor condition, the load test provided many additional information for giving a more accurate diagnosis of the bridge. The main result of the diagnostic load test at a service-load level is that the railway bridge has enough structural capacity to carry the applied loads. The load carrying capacity is one of the key structural performance indicators together with durability. As presented, the bridge has deficient structural durability and therefore it is required to perform repair measures. With respect to the structural durability and enough structural capacity, at the end, it was concluded that the owner has enough time to prepare a project for rehabilitation and bridge traffic does not need to be closed.

DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available on request from the corresponding author. The data are not publicly available due to privacy or ethical restrictions.

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