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Condition assessment and retrofit of a historic steel-truss railway bridge

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Abstract

This paper presents an experimental and analytical study to access the condition of a historic railway steel truss bridge still in use. The task is pursued through static and dynamic field measurements, as well as laboratory tests. A validated analytical model is employed to evaluate the capacity of the bridge to carry seismic and wind loads specified by current design codes, as well as the heavier trainloads set by the owner. Strengthening and replacement measures are proposed for bridge upgrade. An estimation of the remaining fatigue life of the bridge in its present condition and after the suggested strengthening is also made.

Keywords: Steel truss; Railway bridge; Field measurements; Retrofit

1. Introduction and bridge description

At the end of the 19th century, the Greek Government developed a railway network in the southern part of Greece, an area that is mostly mountainous. In 1890 a large number of steel railway bridges were constructed along this network with spans 10 to 60 m, according to French codes and with the cooperation of a French company. In 1944, but mostly in 1963, selective strengthening and replacement of

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Fig. 1. General view of the steel bridge.

members, such as secondary beams, parts of the main beams and bracings, was performed in order to meet increased traffic loads.

This work presents salient features of the procedure, as well as the results of a condition assessment and retrofit study for a characteristic steel-truss bridge of this railway network. The two-span bridge under consideration (see Fig. 1) consists of a symmetric structure about the mid-support steel superstructure of total length 42.40 m long. The main girders are two riveted trusses 2.00 m high and 4.00 m apart, while the main truss girder consists of combined plates and L sections, as shown in Fig. 2. At the bottom chord, the main trusses are connected with transverse secondary beams (SB1), a horizontal bracing system, and the longitudinal secondary beams (SB2), as shown in Fig. 3. In Fig. 4, dashed lines have been used to indicate the new steel plates added to the members that, according to the analysis elaborated in the following sections, need to be strengthened. A representative connection of the horizontal bracing system at the lower chord is shown in Fig. 5. The 20-panel bridge superstructure is supported through two restraining bearings at the intermediate massive unreinforced masonry piers and two simple sliding bearings at each end masonry pier (Fig. 6).

Record files and practices of the Greek Railway Organization (GRO) indicate meticulous and consistent periodic inspection and maintenance of all bridges in this network. Recently, the owner of the railway network (i.e. the GRO) decided to assess their condition and identify necessary upgrading to meet modern standards, while respecting the historic fabric of the bridges.

2. Laboratory tests and field measurements

Extensive laboratory and field testing, as well as analytical work were performed to assess the condition of the steel superstructure and propose a strengthening scheme [6–8].

In situ measurements of member sizes, connections and support bearings verified the fact that the existing drawings were applied and only a few insignificant variations were observed. The in situ measurements were performed using an 800 kN

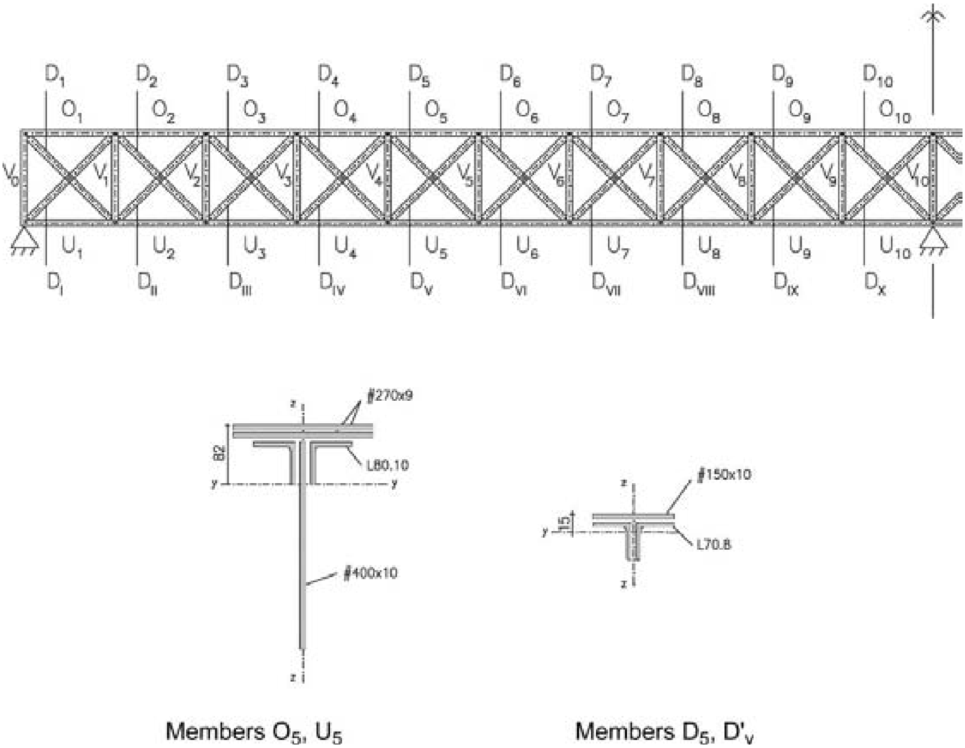

Fig. 2. Main truss girder and representative cross-sections.

engine track and carried out by a team headed by Prof. P. Karydis (Director of the Laboratory for Earthquake Engineering at NTUA). Specifically, the bridge was instrumented with strain gauges placed at selected locations to measure normal stresses, as shown in Figs. 7 and 8. In addition, the vertical vibrations were recorded with accelerometers having a sensitivity of 10 Volts/g (Fig. 9), placed at

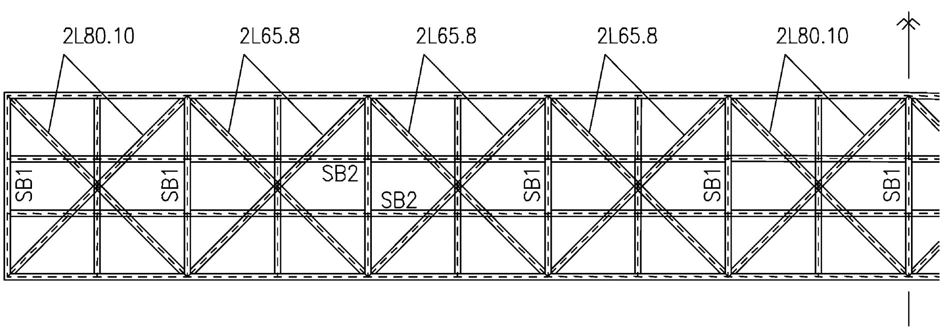

Fig. 3. Horizontal bracing system in the bottom chord and secondary beams (SB1, SB2).

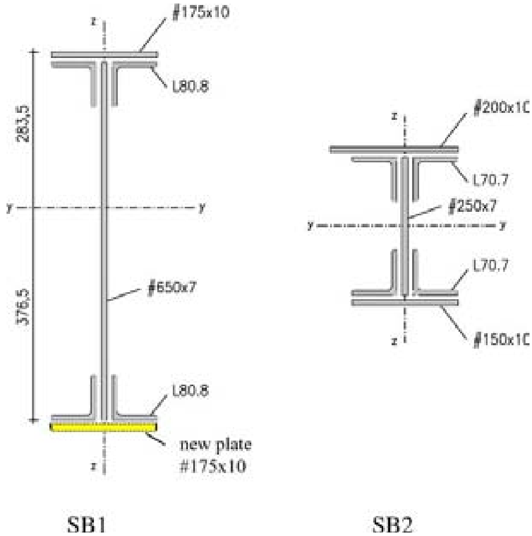

Fig. 4. Cross sections of secondary beams (SB1 and SB2).

point 5 (see Fig. 7). In order to measure the free-vibration, accelerations were recorded after the 800 kN engine had crossed the bridge. Strains were also recorded for trains that crossed the bridge in order to assess the behavior of the superstructure for currently used trains. These measurements were performed for each direction of the traveling train [11].

Laboratory tests, such as tension tests, chemical analysis and fatigue tests, were also carried out on specimens extracted from representative members that are prone to fatigue [12]. More specifically, a sufficient number of specimens were extracted from the webs of the upper and lower chord of the main girder and from the web of the transverse beams, and the members were fully restored with riveted plates as shown in Fig. 10.

Fig. 5. Connection of the bracing system at the lower chord.

Fig. 6. Sliding bearing at end masonry piers.

Mechanical devices were used to measure the vertical deflections at point 4, see Fig. 7. Table 1 lists the measured and calculated vertical deflections at point 4 of the bridge caused by the 800 kN engine. The test-engine is placed on eight characteristic positions (Φ_i) as shown in Fig. 7. Table 2 lists the lowest three eigen-periods extracted from the records during free vibration in the vertical direction.

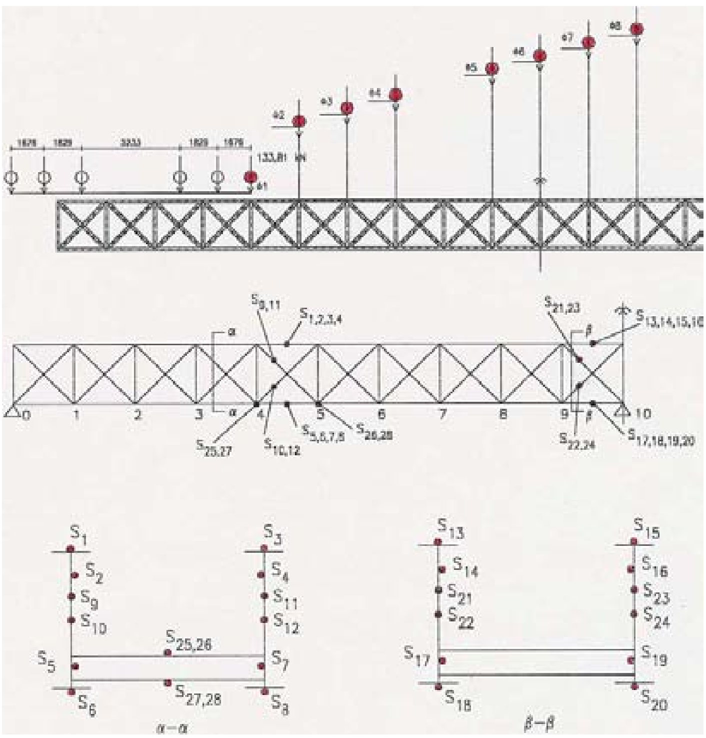

Fig. 7. Locations of the trial tests (Φ_i) and strain gauges (S_i).

Fig. 8. Strain gauges placed at locations S_1 and S_2.

Tension tests indicated that the material of the members partially complied with the St37-2 requirements [12]. Inadequacies were observed regarding the ultimate tensile strength and corresponding elongation. Chemical analysis indicated use of different steel grades for the main girder trusses and the secondary beams, the former being of superior quality than the latter. The results from the chemical analysis are presented in Table 3.

Table 3 indicates that specimens 1 and 2, which were extracted from the truss girder, correspond to an old material, while specimens 3 and 4, which were extracted from the transverse beams, correspond to a new material that replaced the old one 30 to 40 years ago. Moreover, metallographic tests showed that member material is perlite-ferritic steel with several oxides and sulfides [12].

Several specimens were used to perform fatigue tests. Figs. 11 and 12 depict the results of the fatigue tests together with the Woehler curve corresponding to detail category 112 of Eurocode EC3 [3]. In both figures, the fatigue strength of the tested material could be classified at least according to this detail category.

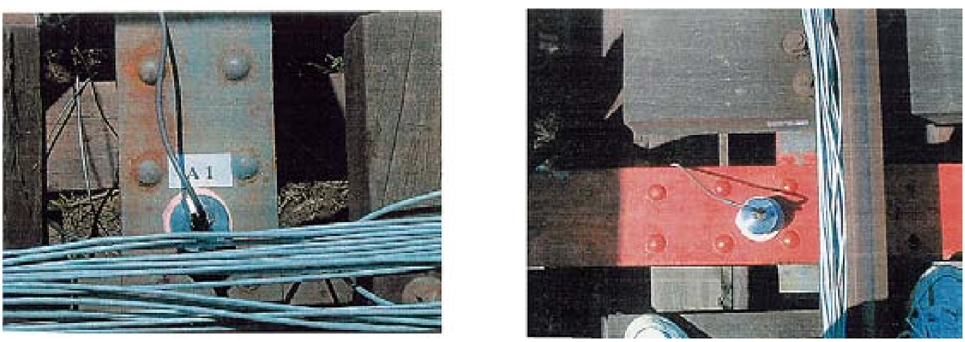

Fig. 9. Vertical accelerometers.

Fig. 10. Details from the locations of the extracted pieces.

3. Development of validated analytical model

A finite element analysis has also been employed to study the response of the structure. The bridge truss is modeled with three-dimensional beam elements conforming to the guidelines given in Ermopoulos [4] and Spyrakos [10]. Although the structural system is a truss system, all joints of the model are modeled as rigid connections according to the design specifications DS804 [2]. A representative connection of vertical studs, diagonal ties and bottom beam of the truss system that

Table 1
Vertical deflections at point 4 (see Fig. 7)

Loadcase (Φ_i)	Measured (cm)	Analytical (cm)
Φ_1	0.600	0.630
Φ_2	0.800	0.773
Φ_3	0.900	0.915
Φ_4	1.000	1.021
Φ_5	1.000	0.976
Φ_6	0.900	0.809
Φ_7	0.600	0.585
Φ_8	0.400	0.353

Table 2
Eigenperiods for free vibration

Normal modes	Recorded (sec)	Analytical (sec)
1	0.114	0.142
2	0.100	0.096
3	0.086	0.067

validates the rigid joint modeling is shown in Fig. 5. As a consequence, moments are transferred through joints and are proven to be of interest only for the top and bottom beams of the trusses, as well as for the floor and deck beams. Moments that develop in the vertical studs and diagonal ties are negligible.

The dynamic analysis of the system has been based on a lumped mass formulation [10]. For the development of the finite element model the following assumptions are made: (1) the structure is pinned to vertical supports; (2) the loads and reactions are applied only at joints; and (3) besides the lumped masses generated at the nodes to simulate the inertia of the members, additional masses are introduced at the deck beams to account for the deck loads.

The material properties of the steel structure have been obtained by in situ and laboratory measurements [12]. The measured modulus of elasticity is $E = 2.1 \times 10^5$ MPa, the yield stress is $f_y = 285$ MPa, and the ultimate stress is $f_u = 308$ MPa.

The results from the analytical calculations were compared with those obtained from the field measurements in order to assess the accuracy of the 3D-model used in the analysis. For this reason, the normal stresses were calculated at the same locations where the strain gauges have been placed (see Fig. 7, locations S_i). For each position of the engine truck (Φ_i) ($i = 1$ to 8) shown in Fig. 7, the corresponding stresses and the vertical deflections at point 4 were calculated analytically. Table 1 lists the deflections computed by the finite element model, as well as the in situ measured values for various positions of the test engine truck. As shown in Table 1, the measured and computed deflections are practically identical.

The first three eigenperiods of the 3D finite element model of the truss bridge are listed in Table 2 and the corresponding mode shapes are shown in Fig. 13. The correlation between the measured and the calculated eigenperiods and modal shapes indicates the validity of the finite element model.

Table 3
Results from the chemical analysis (%)

Specimens	C	Si	Mn	P	S
No. 1	0.056	4.59	0.19	>0.17	>0.10
No. 2	0.031	3.62	0.42	>0.17	>0.10
No. 3	0.024	1.48	0.15	>0.17	>0.10
No. 4	0.035	1.40	0.15	>0.17	>0.10

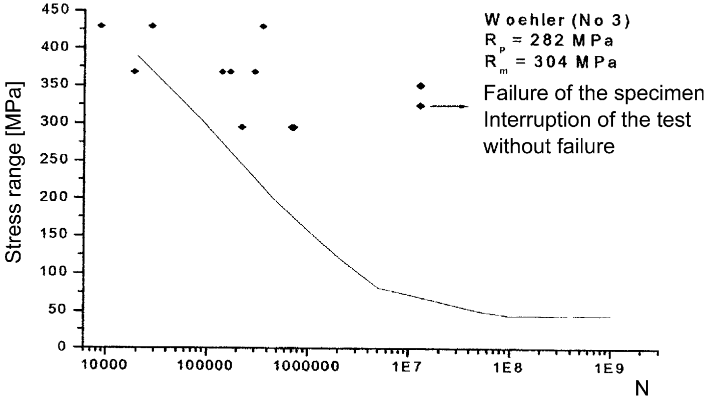

Fig. 11. Results from fatigue tests for the main girder.

The stresses calculated from the analysis and those obtained from measurements at indicative locations are shown in Figs. 14 and 15. No significant differences are observed. However, conservatism in the calculations with the finite element model is preserved, since in most cases the analytical results are greater than the corresponding measured values.

4. Analysis for higher loads and strengthening measures

The validated 3D model was used to analyze the bridge for loading combinations involving the train types specified by the owner, in order to assess the strength, stability and functionality of the bridge.

The loads for the analysis, such as wind loads, traction and braking forces, nosing force and eccentricity of vertical loads, were applied according to the German Codes DS804 [2] and Eurocode 1, Part 2 [9]. The earthquake load was applied

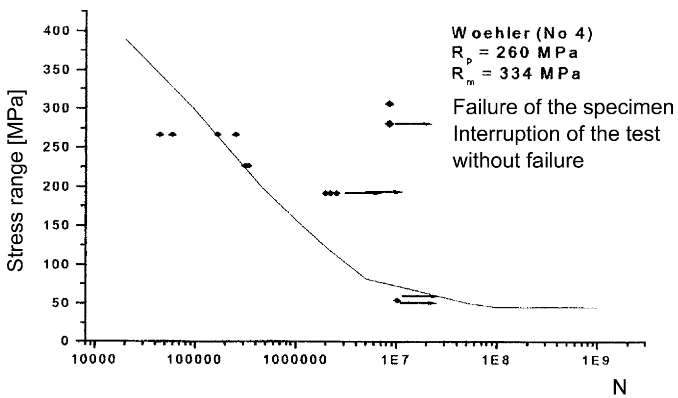

Fig. 12. Results from fatigue tests for the transverse girder.

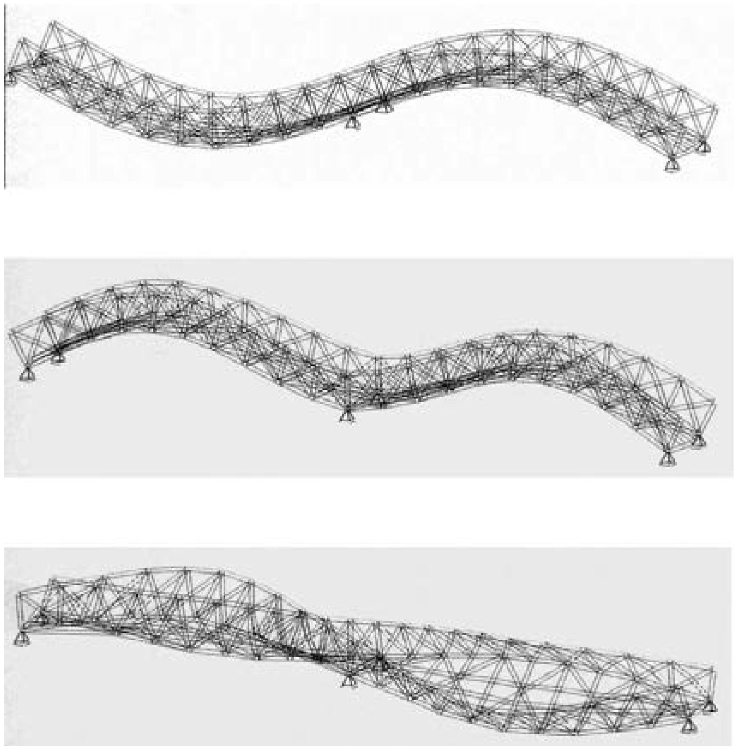

Fig. 13. First, second, and third vertical mode shapes.

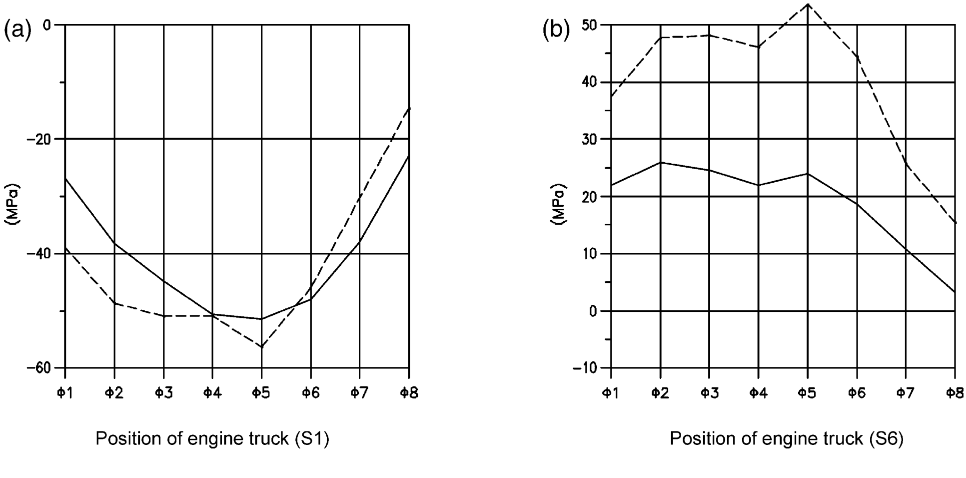

Fig. 14. Locations S1 and S6: Stress versus loading (Φi) (continuous line: tests, dashed line: analytical).

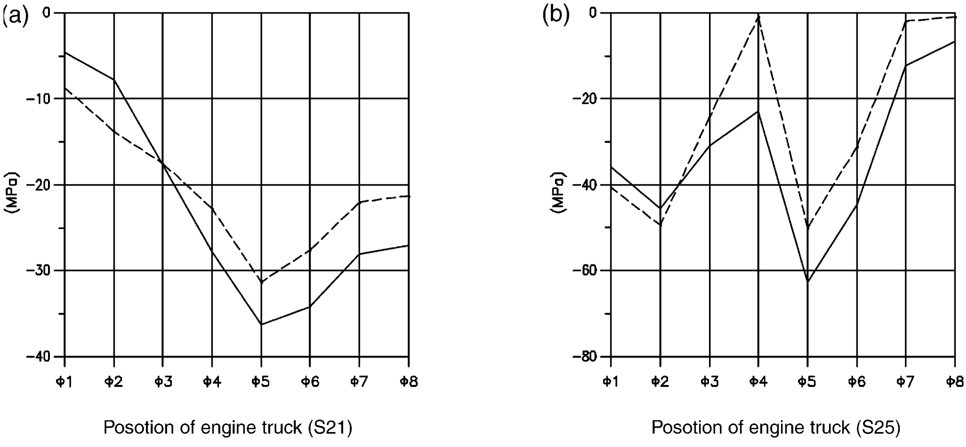

Fig. 15. Locations S21 and S25: Stress versus loading (Φi) (continuous line: tests, dashed line: analytical).

according to the Greek Aseismic Code (EAK 2000) [5]. The design acceleration spectrum, $\Phi_d(T)$, for the bridge site is shown in Fig. 16. The spectra correspond to a soil class that characterizes weathered rocks and to a seismic behavior factor, $q = 1$, which corresponds to elastic behavior. The railway loads, specified by the owner, are shown in Fig. 17. Specifically, the first load model (Train 1961) consists of either one or two engines (a) followed by a series of railroad cars (b), while the second (RAIL-BUS) consists of a minimum of one to a maximum of three railroad cars (c) [13].

The design calculations for the bridge members was based on DIN18800 [1]. The connections between the members were assumed to be rigid, and an analysis taking

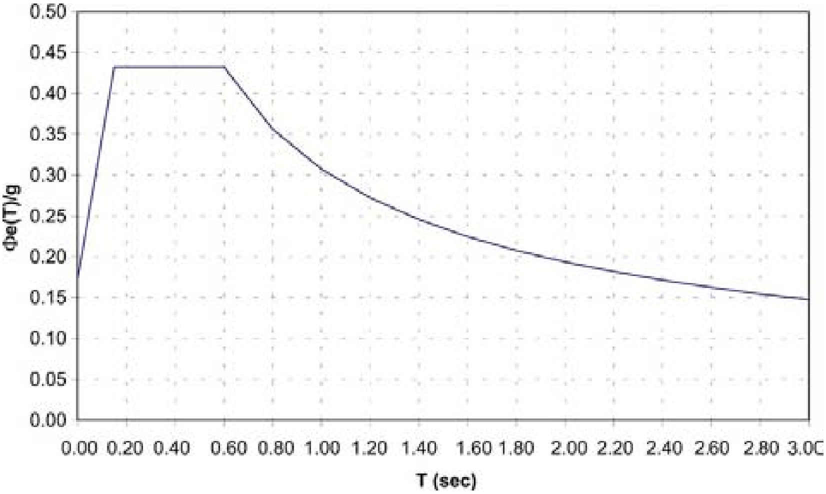

Fig. 16. Design spectrum.

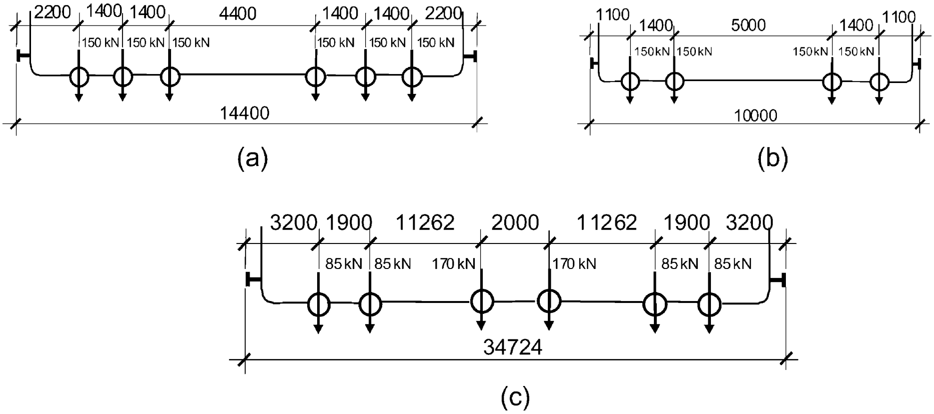

Fig. 17. Train 1961, (a) engine and (b) railroad car, (c) RAILBUS.

into account all the load combinations was carried out. Fatigue calculations were based on real material properties obtained from the tests using the load history, as well as future loads, given by the owner. Since data on the load history of the bridge were not available before the Second World War, mean values were adopted for this period.

The analysis and the design calculations showed that for the new train types, i.e. Train 1961 and RAILBUS, strengthening in some members was necessary. These members are the transversal secondary beams SB1 located in the lower chord, see Fig. 3. The type of strengthening used is indicated in Fig. 4 with dashed lines, that is addition of a new steel plate riveted to the lower flange. Riveting is suggested for aesthetics.

Finally, Table 4 presents the results of two schemes regarding the remaining fatigue life of the various parts of the bridge after the suggested strengthening of the members.

5. Final results and conclusions

The most significant conclusions that can be drawn from this study are the following:

Table 4
Remaining fatigue life

Scenarios of future traffic	Secondary beams (BS2)	Secondary transversal beams (BS1)	Main girders
Train 1961: 10 passages per day	20 years	10 years	40 years
RAILBUS (three cars): 10 passages per day	20 years	10 years	40 years

- The analysis and design calculations for the new train types specified by the owner showed that the main truss system can carry the new loads, while strengthening is required only in the transverse secondary beams SB1, Fig. 3.
- Thorough inspection and in situ measurements showed that the original drawings have been accurately followed and are in full agreement with the existing geometry and dimensions of the bridge.
- The systematic and periodic inspection and maintenance has primarily attributed to the relatively good condition of the bridge.
- A procedure to minimize interruption of traffic during strengthening works on the bridge must be developed since the transverse secondary beams SB1 that need strengthening are directly connected to the rail tracks.
- In conclusion, the bridge can safely fulfill the design requirements and future needs of the GRO, provided that the suggested strengthening is materialized.

Acknowledgements

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