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FATIGUE ASSESSMENT OF AN 100 YEARS OLD RIVETED TRUSS RAILWAY BRIDGE

Mateusz Rajchel[1], **Tomasz Siwowski**[2]
[1]*Rzeszow University of Technology, Rzeszow, Poland*
[2]*Rzeszow University of Technology, Rzeszow, Poland*

Abstract

Fatigue is one of the most often reasons for failure in existing steel railway bridges, particularly with riveted structure. Therefore fatigue assessment is always the most important part of the complex evaluation of existing steel railway bridges. Furthermore, the reliable fatigue assessment is often decisive in the estimation of the remaining service life of a bridge. The purpose of the presented research is a fatigue assessment of an 100 years old riveted truss railway bridge. The assessment was carried out to decide whether the existing truss superstructure is still suitable for rehabilitation in the frame of the ongoing modernization of the railway line. The fatigue assessment procedure is based on the safe life method in the convention of nominal stresses.

Keywords: fatigue life, Eurocodes, safe life method, Miner's cumulative damage theory, riveted truss bridge, rehabilitation

1 Introduction

Most of the large railway bridges built in Poland after regaining independence in 1918 and immediately after the end of World War II are riveted steel trusses. Despite a significant increase in the load on railway bridges over several decades in relation to the design loads, the vast majority of these bridges are still in service thanks to the repair and strengthening works carried out during the period of operation, which were aimed at extending the remaining service life of the bridges by another 20–30 years.

Issues related to fatigue assessment and remaining service life of riveted bridges are still the subject of research and standardization in many countries. Since the beginning of the 1980s, many scientific papers have been published around the world, in which various methods of estimating the remaining service life of existing steel bridges are presented [1-8]. However, in the light of the continued development of the common European market for civil works and engineering services, there is a need to harmonise the various procedures and create acceptable recommendations for assessing the safety and remaining service life of existing bridges. The answer to this demand is the European guidelines [9], based on the Eurocodes and constituting the basis for future European standards for the prediction of fatigue life of existing bridges. The fatigue life assessment procedure recommended in the European guidelines [9] has already been used several times by the authors to evaluate steel road bridges with the riveted and welded bridge structures [9-12].

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The paper presents the fatigue assessment of an 100-years old railway riveted truss bridge, in which the procedure recommended in the European guidelines [9] is applied. The railway bridge over the Oder River in Opole (Poland) is situated along the E-30 railway line, located in the Third Pan-European Transport Corridor. The Polish Railway Lines Authority (PKP PLK) was going to modernize the railway line to increase its operating parameters to the maximum speed of passenger and freight trains up to 160 km/h and 120 km/h, respectively.The main objectives of this work is to present the applied assessment procedure and its main results, which led to a change in the decision of the railway line administrator about the remaining service life of the bridge.

2 Description of the bridge

2.1 Bridge scheme and its geometry

The double-track railway bridge over the Oder River in Opole was put into operation in the year 1928, so its steel structure is nearly 100 years old [14]. The bridge is a four-span truss structure with a static scheme of a continuous beam with hinges (the so-called Gerber system). The main geometrical parameters of the bridge are as follows: theoretical span lengths (in the axes of supports) 37.00 + 44.40 + 59.20 + 37.00 m; spacing of the main truss girders (in the axes of the girders) - 8.50 m (two tracks); the rise of the truss girders 9.27 m (main span) and 4.82 m (approaching spans) (Fig. 1).

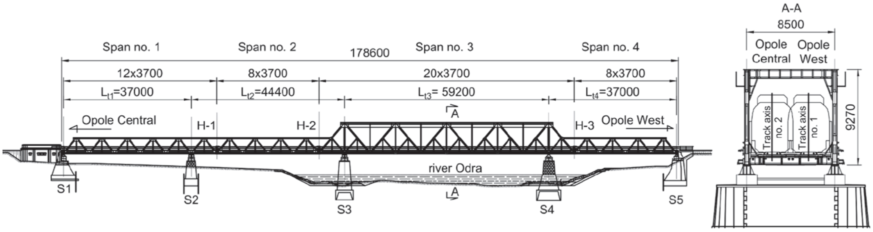

Figure 1 Scheme of the railway bridge over the Oder River in Opole

The truss girders have parallel chords and a "V" truss system of diagonals with additional verticals. The main span has an additional secondary "V" truss system. The bridge has an open deck in the form of a steel grid made of riveted plated crossbeams 0.90 m deep and longitudinal rolled stringers INP 340 type. The stringers are connected to the crossbeams by a system of vertical angles (webs) and horizontal angles (flanges). The top flanges of the stringers are pulled over the crossbeams by means of steel overlays. All joints in the steel superstructure (truss girders, deck grid, bracings) are riveted. All massive supports of the bridge are made of concrete with a stone facade. The steel riveted superstructure of the railway bridge over the Oder River in Opole is shown in Fig.2.

Figure 2 The steel riveted superstructure of the railway bridge over the Oder River in Opole

2.2 Technical condition of steel superstructure

The detailed inspection of the steel superstructure was carried out to identify any damages, particularly fatigue cracks, which may have appeared in the structure during the 100 years of operation of the bridge. Except damages in Gerber hinges and surface corrosion of steel (losses of up to 5% of the cross-section of members), the technical condition of the truss girders was satisfactory. However, the most damaged members of the steel structure are the connections of stringers and crossbeams within the bridge open grid deck. As a result of intensive fatigue associated with nearly 100 years of bridge service life, numerous fatigue cracks occur in these joints, weakening both the connection and individual members of the deck (Fig. 3).

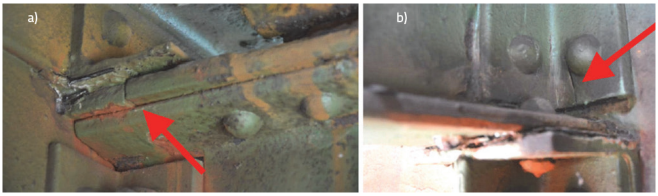

Figure 3 Typical failures of stringer and crossbeam joints: (a) cracked continuity pad of the stringer's upper flange; (b) crack in the connecting Angle

2.3 Material characteristics

Archival data on the structural steel used in the construction of the bridge were obtained from the publication [14]. The sampling procedure and the assessment of the material properties of the steel were carried out in accordance with the guidelines [9]. A total of 16 samples were taken from the bridge structure. All spans and all main members of truss girders as well as the deck were tested. The chemical composition of StSi steel was determined using the Q4 Tasman spark emission spectrometer from Bruker. The mechanical tests of StSi steel were carried out according to reference standards [15], [16]. The tests determined the strength properties (R_e or $R_{0.2}$, R_m), plastic properties (L_u, A_5) and fracture toughness (U) at -20°C (table 1).

Table 1 Design yield strength of steel determined from material testing

Mean yield strength	Standard deviation	Coefficient of variation	Conversion factor (for 30 tested samples)	Material safety factor	Design yield strength
fy,mean	Ss	Vx	kn	γM	fd
[MPa]	[MPa]	[-]	[-]	[-]	[MPa]
391.47	22.42	0.057	1.67	1.106	354.04

3 Structural and load models for fatigue assessment

3.1 Numerical model of superstructure

All calculations needed for the fatigue assessment of the bridge superstructure were performed using the finite member method (FEM) and the structural model built in *Sofistik* code. The geometry of the steel superstructure was modelled using a 3-D numerical model, the dimensions of which were determined on the basis of inventory drawings (Fig. 1).

Moreover, using data from the site inspection the actual cross-sections of members were assumed with regard to identified corrosion losses. Also due to corrosion of structural connections, the modelling was not based on a simplified truss model with pinned connections but the full end fixity of riveted members was assumed for the truss girders and deck grid. Thus the effects from secondary moments due to the stiffness of the connection was taken into account. The truss girders and deck members were modelled with two-node beam members discretized individually with a maximum mesh size of about 30 mm.

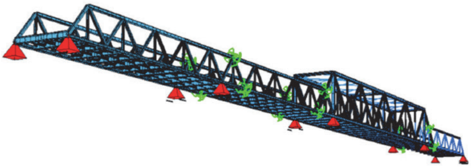

Figure 4 The numerical model of bridge superstructure

The hinges were modelled with spring members in three orthogonal directions that do not transmit bending moments. Due to the riveted type of the superstructure having a large number of connection members (rivets, overlays, connection pads, gusset plates, etc.), the self-weight of the steel members was assumed on the basis of the inventoried gross cross-sections and increased by 15%. In the numerical model structural steel was characterized as a linear-elastic isotropic material with two engineering constants E = 210 GPa and v = 0.3. The 3-D numerical model of the bridge superstructure was applied in both phases of fatigue assessment.

3.2 Fatigue loads of bridge superstructure

In phase I the basic model of fatigue loads, the Load Model 71 according to Eurocode 1-2 [21]. Since the existing bridge had two tracks, the load was set successively on both tracks in the most unfavourable positions. In phase II the composition of two fatigue load models FLM type 2 and FLM type 8 according to Eurocode 1-2 [21] corresponding to passenger and freight trains, respectively, was applied. The selected fatigue models are very similar in terms of the load per axle to the Load Model D-4 according to the railway standard EN 15528 [19], used to assess the actual carrying capacity of the existing bridge (see p.2). Therefore, the railway administration PKP PLK considered them to be the most adequate to simulate the current traffic on the bridge. The full spectrum of the bridge's load with rolling stock in the years 1930 – 2020. Calculated on the basis of the prepared load spectrum, the total number of all trains that crossed the bridge during its period of operation between 1930 and 2020 is N_{obs} = 2,904,694 trains per bridge and N_{obs} = 1,452,347 trains per track. During this period, the total share of passenger trains was 1,075,890 and freight trains was 376,457. The numbers of both trains determined in this way were used to assess the fatigue life of the bridge in phase II.

4 Fatigue assessment results and discussion

There are different methods for fatigue assessment and different approaches are selected for different circumstances to ensure the greatest possible accuracy and applicability. The most commonly used fatigue assessment methods include infinite-life design, safe-life design, damage-tolerance design, and durability design. The European guidelines [9] based on the Eurocodes and safe-life design method were applied by the authors to assess the fatigue life of existing bridge. This method is based on Miner's linear cumulative damage theory and the S–N curves.

4.1 Phase I

In calculations of stress ranges $\Delta\sigma_p$ the appropriate fatigue load model LM 71 was set successively on both tracks in the most unfavourable positions along the span lengths. The stresses in individual bridge members were calculated for net cross-sections, i.e. taking into account the existing holes for rivets and the loss of the cross-sections due to corrosion. In the calculations the load safety factors were omitted, but the dynamic coefficients φ_2 for particular members were applied according to formula (5). For preliminary evaluation of the bridge according to formula (2) the following values were applied: $\Delta\sigma_C = 71$ MPa – for riveted joints according to the European guidelines [9]; $\Delta\sigma_C = 160$ MPa – for the rolled members according to Eurocode 3-1-9 [20]; $\gamma_{Ff} = 1{,}0$ – fatigue safety factor according to Eurocode 3-2 [17] and $\gamma_{Mf} = 1{,}35$ – partial safety factor for high consequences of failure according to Eurocode 3-1-9 [20]. Damage coefficient λ_2 for the bridge was determined using the volume of rail traffic on the relevant section of the railway line provided by the railway authority in the period of 14 years, i.e. from 2006 to 2020. On the other hand, the planned service life of the bridge after its modernization was assumed to be 50 years. Other damage coefficients λ_i depending on the geometry of the bridge were adopted according to Eurocode 3-2 [17]. The calculations of stresses in the individual members of the truss girders and the deck were made using the numerical model (fig. 4). The maximum and minimum stresses in the net cross-section of each member were determined and the stress ranges $\Delta\sigma_p$ were calculated. For partially or fully compressed members (top chord in the span zones, bottom chord in the support zones and compressed diagonals and verticals) an effective stress range was calculated by adding the tensile portion of the stress range and 60% of the magnitude of the compressive portion of the stress range [14]. As a result of the calculations in Phase I the members of truss girders and deck grid at high risk of fatigue (i.e. critical construction details for which the $\mu_{fat} < 1.0$) were determined. The exemplary results of preliminary evaluation in phase I are summarised in Table 2, where the calculated values of μ_{fat} for the selected bridge members in the main span no. 3 are given.

Table 2 Results of preliminary evaluation (phase I) – selected members of the span no. 3

Bridge members	**Truss girders**				**Deck grillage**			
	BC3	**BC5**	**D1**	**D2**	**CB1**	**CB2-20**	**S1-20**	**RC1-20**
$\Delta\sigma_p$ [MPa]	224.99	127.05	143.57	166.81	123.48	129.88	115.485	186.48
λ	0.431	0.431	0.431	0.431	0.594	0.594	1.064	0.920
Φ_2	1.007	1.007	1.007	1.007	1.203	1.203	1.280	1.280
$\Delta\sigma_{E2}$ [MPa]	97.071	54.814	61.940	71.966	73.371	77.176	122.876	171.629
$\Delta\sigma_c$ [MPa]	71.000	71.000	71.000	71.000	71.000	71.000	160.000	71.000
γ_{Ff}	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
γ_{Mf}	1.350	1.350	1.350	1.350	1.350	1.350	1.350	1.350
μ_{fat}	0.542	0.959	0.849	0.731	0.717	0.681	0.965	0.306
$\mu_{fat} \geq 1$	No	No	No	No	No	No	No	No

BC – bottom chord, D – diagonals, CB – crossbeams, S – stringers, RC – riveted connections in the deck

The obtained results of phase I showed that a significant part of the bridge's steel superstructure is at high risk of fatigue. A total of 25 members at risk of fatigue were identified in the truss girders along the bridge, including 15 sections of the bottom chord and 10 diagonals. In total, for both truss girders, these numbers are twice as big. On the other hand, in the deck grid all cross-beams (52), stringers (192) and their connections were identified at risk of fatigue. For all individual members identified in phase I as fatigue prone, a detailed investigation (phase II) was performed.

4.2 Phase II

The assessment of fatigue life in phase II of bridge members for which phase I showed a fatigue hazard was carried out taking into account fatigue load models and fatigue strength curves specified in Eurocode 3-1-9 [20]. The stress ranges $\Delta\sigma_i$ were calculated for passenger (FLM type 2) and freight (FLM type 8) trains for each critical member identified in phase I. Determined values of $\Delta\sigma_i$ was used in the calculation of the total fatigue damage D_d according to formula (5) for each critical member in the planned 50-year service life of the bridge. The calculated values of the total fatigue damage level D_d and the fatigue life T_s for the critical bridge members in the main span no.3 are given in Table 3.

Table 3 Fatigue damage level Dd and fatigue life Ts calculation for critical members of the span no. 3

Critical member	Type of trains	Δσi [MPa]	Δσc [MPa]	ni [x10^6]	m	NRi [x10^5]	ni/Nri	Dd	Ts
BC3	Type 2	81.74	71	1,075	3	5,327	2.019	6.95	0.14
	Type 8	156.24	71	0,376	3	0,762	4.934		
D1	Type 2	61.69	71	1,075	3	12,393	0.868	1.44	0.69
	Type 8	76.58	71	0,376	3	6,479	0.581		
	Type 8	145.53	71	0,376	3	0,943	3.988		

According to the performed calculations, the vast majority of the checked bridge members have already exceeded significantly acceptable fatigue damage level ($D_d > 1.0$) and almost exhausted fatigue life ($T_s < 1$ year). This means that the process of initiation and/or propagation of fatigue cracks can begin in these members at any time, what was confirmed during the detailed inspection. The most endangered members are the deck and the bottom chords of the truss girders. In the case of the deck, the cracks are initiated in riveted connections between the stringers and the crossbeams, while in the case of the bottom chords – in the net cross-sections under gusset plates of the nodes. Other components examined in phase II are also at high risk of fatigue and cracking, but to a slightly lesser extent.

5 Final conclusions

The case study described in the article showed that in the case of old riveted steel bridges, even in spite of their sufficient actual load carrying capacity, the fatigue assessment should always determine the possibility and economic sense of bridge rehabilitation. Fatigue assessment is important because steel rivets have the lowest fatigue life.The applied fatigue assessment method, based on the European guidelines [9], seems to be an effective tool to facilitate the final decision.

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