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Fatigue assessment of 100 years old riveted truss railway bridge

Abstract: Fatigue is one of the most often reasons for failure in existing steel bridges, particularly with riveted structure. Therefore fatigue assessment is always the most important part of the complex evaluation of existing steel bridges. The reliable fatigue assessment is often decisive in the estimation of the remaining service life of a bridge. The example of a comprehensive fatigue assessment of 100 years old riveted truss railway bridge, based on European procedures and codes, has been presented in the paper. The applied procedure is based on the safe life method in the convention of nominal stresses. The bridge assessment revealed, that the remaining fatigue strength was almost exhausted and the further service life of the bridge was close to zero. Taking into account the assessment results it was also revealed, that the existing riveted truss superstructure was not suitable for further use in the railway bridge after its rehabilitation, which aimed to extend the bridge service life by the next 50 years.

Keywords: Railway bridge; Steel structure; Truss, riveted joints; Fatigue life

Introduction

Most of the large railway bridges built in Poland in the first half of the 20th century and immediately after World War II were riveted steel trusses. Despite a significant increase in a load of railway bridges over several decades in relation to the design loads, the vast majority of these bridges are still in operation thanks to the renovation, strengthening, and modernization of their structures carried out during the previous operation, which were aimed at extending the useful life and service life of the bridges by another 20-30 years. The analysis of the maintenance documents of these bridges shows that most of them are in good technical condition, even despite advanced corrosion in some cases. These bridges easily carry much greater operational loads than those for which they were designed. The stress measurements carried out in many cases have shown that their values are significantly lower than the design strength of the steel, and detailed inspections have not detected any serious fatigue cracks in the main elements of the structure. Therefore, it would seem that the decisions to extend the service life of these bridges and the funds spent on adapting them to modern operational requirements are rational.

However, the crash of the Mississippi road bridge in Minneapolis, USA [5] and research conducted in Europe by several EU research projects (including Long Life Bridges, I-SAMCO, Sustainable Bridges) have identified the main areas of uncertainty and risk related to the prolonged operation of steel railway bridges. These include: material parameters of old steels (aging, fracture resistance), the fatigue strength of riveted joints, stability of gusset plates in joints, the technical condition of the structure (corrosion), etc. The uncertainty areas show how important the problem is to assess the fatigue life in the assessment of the technical condition and further operational usefulness of steel railway bridges with riveted structures. The lack of effective NDT methods for assessing the condition of riveted joints (e.g. detecting cracks under gusset plates) makes it difficult to reliably assess the durability and operational suitability of these bridges. As a consequence, projects involving the repair, strengthening, and modernization of existing steel rail bridges do not take into account the problems associated with their limited or exhausted fatigue life. Therefore, it is currently postulated that, before making a decision to extend the service life of the existing bridge, a detailed analysis of the fatigue life of the worn-out structure should be carried out each time. [1], [2], [6], [7].

In Poland, since the beginning of the 1980s, many scientific papers have been prepared in which various methods of estimating the durability of existing steel bridges have been presented, including [3], [4], [17], [18]. However, in view of the constant development of the common European market for construction works and engineering services, there is a need to harmonize the various procedures and create acceptable recommendations for the assessment of the safety and durability of existing bridge structures. European guidelines are the answer to this demand [8], based on Eurocodes and being the basis for future European standards for forecasting the fatigue life of existing bridges. The authors of the fatigue life assessment procedure recommended in the European guidelines have already been used several times to assess steel road bridges with a riveted truss structure [14], [15], 16].

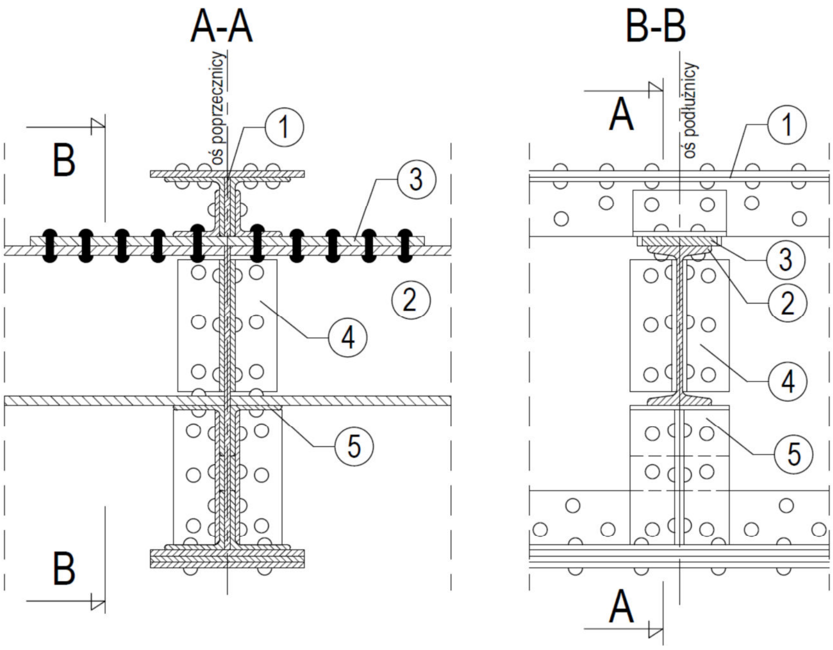
The article presents an assessment of the fatigue life and operational suitability of a 100-year-old railway bridge over the Odra River in Opole with a riveted truss structure using the procedure recommended in the guidelines [8]. The railway bridge over the Odra River in Opole is located at km 100,106 of railway line No. 132, connecting Bytom with Wrocław. This line is a part of the E 30 mainline lying in the 3rd Pan-European Transport Corridor. PKP PLK wants to raise the operational parameters of this line to a maximum speed of 160 km/h for passenger trains and 120 km/h for freight trains. The prepared design documentation for the modernization of the railway line No. 132 on the Opole Groszowice - Opole Zachodnie section assumed the use of the steel structure of the existing railway bridge over the Odra river in the planned further 50-year operation of the line, after limited modernization works, mainly related to the repair and/or reinforcement of the bridge elements.

Description of the structure and technical condition of the bridge spans

The railway bridge over the Odra River in Opole was put into operation in the 1920s, so its construction is almost 100 years old [13]. The steel structure of the bridge spans is a four-span, two-girder truss structure with a static diagram of a continuous beam with Gerber's joints, with an open platform in the form of a longitudinal and transverse grating made of riveted plate girders and rolled beams (Fig.**1**). All connections in the steel structure are riveted. The main geometric parameters of the bridge are as follows: theoretical spans (in the axes of the supports) 37.00 + 44.40 + 59.20 + 37.00 m, the spacing of the main girders (in the axes of the girders) 8.50 m.

The most damaged elements of the steel structure of the bridge spans are the joints of stringers and cross members within the platform (Fig.**2**). As a result of intense fatigue processes related to nearly 100 years of bridge operation, these connections have numerous fatigue cracks, weakening the connection and individual elements of the deck (Fig.**3a**). The consequences of these cracks and their accelerated propagation are intensified by pitting and surface corrosion, caused by the lack of a durable and effective anti-corrosion protection of steel (Fig.**3b**). An additional consequence of damage to the connections in the platform (in particular, cracks in the continuity overlays on the stringers) is the change in the static pattern of the stringers from a continuous beam to a freely supported one, thus overloading the stringers. In the connection zone of the platform elements, there is also stress corrosion of the cross members, resulting from significant material stress in the area of these nodes.

1. General view of the railway bridge over the Oder river in Opole



2. Stringer with the cross member Junction: (1) the cross member, (2) the stringer, (3) the stringer that pulls the stringer, (4) the angles connecting the stringer web with the cross member web, (5) the stool supporting the stringer

a)

b)

3. Typical damage to the joints of stringers and cross members:
(a) broken plate pulling the top chord of the stringer;
(b) intense pitting corrosion of the chords of the upper stringers together with the drag plates

With the exception of surface corrosion of steel (losses of up to 5% of the cross-section of elements) and damages in joints, the technical condition of truss girders is satisfactory. However, damage identified in the joints (broken elements, missing bolts) indicate their overload and may cause malfunction of the joints (Fig.**4**). This unfavorable condition is aggravated by corrosion of the components around the joints. It can be said with high probability that the current state of the hinges causes their blocking, which results in a change in the static pattern of the structure and an unfavorable redistribution of internal forces in individual truss elements.

4. Joint in the upper flange of the truss girder: visible crack in the top plate due to excessive belt displacement/rotation and corrosion of the joint

The performed verification calculations showed that the spans of the bridge meet the standard conditions of the temporary (operational) load capacity assuming that the bridge is loaded with real rolling stock (Model D4 according to PN-EN 15528 [9]). These conditions are met taking into account the actual material parameters of the structure (i.e. steel strength determined on the basis of tests) and the current technical condition of the steel structure, assuming an average 5% corrosion loss of the cross-section of all structural elements and cracks in the continuity overlays (change of the static diagram). The most stressed elements of the steel structure of the spans are the lower chords in the support zone (58.5%), the deck stringers (62.6%), and their connections with the cross members (62%). Taking into account the damage to the overlays, the immediate load capacity of the stools on which the platform longitudinal members rest is used in 80%.

Procedure for fatigue life assessment of existing steel bridges

The procedure includes four basic phases of fatigue life assessment of existing steel bridges. The aim of phase I is to identify the critical elements of the structure due to the exhaustion of the fatigue life. Phase II includes detailed fatigue calculations and quantitative NDT tests of those elements for which phase I has shown an insufficient level of fatigue life. The purpose of the calculations in this phase is, inter alia, defining the so-called unconditional fatigue life, i.e. the expected time period to fatigue failure for the assumed load (stress) spectrum. Such a calculation is performed using the Palmgren-Miner linear accumulation method. In the event of possible serious consequences caused by the exhausted fatigue life of the bridge, it is possible to take additional measures with the use of specialized methods and tools (phase III). These methods and tools are mainly: (a) methods of fracture mechanics in the assessment of unconditional fatigue life, (b) probabilistic methods in the assessment of the impact of the variability of basic parameters on fatigue life, and (c) qualitative NDT tests. If the results of the actions envisaged in phases I to III do not confirm that the bridge can be left in operation with the standard level of current maintenance, it is necessary to introduce protective and/or repair works. As part of these works, which constitute Phase IV of the actions recommended in the procedure, it may be necessary: intensification of inspections and/or introduction of constant monitoring of the bridge, reduction of the weight and number of vehicles traveling on the bridge, repair / replacement of broken elements, reinforcement of selected elements of the bridge, and in the worst case now - closing and demolition of the bridge.

The preliminary assessment of the fatigue safety level of the elements (phase I) is performed using the formula (1):

$$\mu_{fat} = \frac{\Delta\sigma_c}{\gamma_{Ff}\gamma_{Mf}\Delta\sigma_{E,2}} \quad (1)$$

where:

- μ_{fat} – fatigue safety level,
- $\Delta\sigma_c$ – standard fatigue strength for N_C=2 million cycles (the so-called component fatigue category),
- $\Delta\sigma_{E,2}$ – equivalent constant amplitude stress variation range, related to 2 million cycles,
- γ_{Mf} – partial safety factor for fatigue strength $\Delta\sigma_c$,
- γ_{Ff} – partial safety factor for the equivalent range of stress variation with constant amplitude $\Delta\sigma_{E,2}$.

The values of individual quantities are adopted according to the PN-EN 1993-1-9 standard [10]. The equivalent range of stress variation with constant amplitude, related to 2 million cycles, is calculated by the formula (2):

$$\gamma_{Ff}\Delta\sigma_{E,2} = \lambda \times \Delta\sigma(\gamma_{Ff,}Q_k) \quad (2)$$

where:

- $\Delta\sigma(\gamma_{Ff,}Q_k)$ – range of stress variability due to fatigue loads according to PN-EN 1991-2 [11],
- Q_k - value of loads characteristic for the Model 71 of railway bridge loads according to PN-EN 1991-2 [11],
- $\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4$ ale dla $\lambda \leq \lambda_1$ - damage equivalence factor according to PN-EN 1993-2 [12],
- λ_1 – the coefficient expressing the effect of traffic damage depends on the length of the influence line,
- λ_2 – a coefficient expressing the impact of traffic volume,
- λ_3 – factor expressing the impact of the life of the bridge,
- λ_4 – coefficient used in structural members loaded on more than one track.

The range of stress variability due to the fatigue load of Model 71 is determined as the algebraic difference of the maximum and minimum stresses in the analyzed element according to the formula (3):

$$\Delta\sigma_p = |\Delta\sigma_{p,max} - \Delta\sigma_{p,min}| \quad (3)$$

When the fatigue safety level is determined according to the procedure described above $\mu_{fat} \geq 1{,}0$ then it should be assumed that there is no risk of fatigue in the element (joint). When $\mu_{fat} < 1{,}0$ then it is necessary to proceed to phase II of the fatigue life assessment procedure.

Phase II of the fatigue life assessment consists of detailed calculations and tests of those elements for which phase I showed an insufficient level of fatigue safety (i.e. $\mu_{fat} < 1{,}0$). The calculation in phase II is performed using the Palmgren-Miner's linear accumulation method according to the formula (4):

$$D_d = \sum \frac{n_{Ei}}{N_{Ri}} \leq 1 \text{ and } T_s = \frac{1}{D_d} \quad (4)$$

where:

- D_d – total fatigue damage in the analyzed service life,
- n_{Ei} – number of cycles related to the range of stress variability $\gamma_{Ff}\Delta\sigma_i$ in the i-th band of the computational load spectrum,
- N_{Ri} – design life (as the number of cycles) obtained on the basis of the computational fatigue curve,
- $\Delta\sigma_c/\gamma_{Mf}$ – for the range of variation $\gamma_{Ff}\Delta\sigma_i$,
- T_s – unconditional fatigue life expressed in years.

Durability (fatigue life), i.e. expected number of cycles to fatigue failure at stress amplitude $\Delta\sigma_i$ determined according to the formula (5):

$$N_{Ri} = 2 \times 10^6 \times \left(\frac{\Delta\sigma_C}{\Delta\sigma_i}\frac{1}{\gamma_{Ff}\gamma_{Mf}}\right)^{m_i} \tag{5}$$

To obtain the durability value N_{Ri} for each band of the load spectrum, the applied stress variation ranges are multiplied by the load factor γ_{Ff}, while the value of the fatigue strength is divided by the material factor γ_{Mf}. The fatigue life of an element is not endangered when the total fatigue failure occurs $D_d \leq 1{,}0$.

Fatigue strength continued at amplitude $\Delta\sigma_D$, i.e. the limit value of the stress variability range at a constant amplitude, below which no fatigue damage occurs, was adopted according to PN-EN 1993-1-9 [10] for $N_D = 5 \times 10^6$ cycles. At the same time, due to the possibility of occasional occurrence of values greater than N_D, for the number of cycles in between 5×10^6 and 10^8 it is possible to modify the standard fatigue curve for the value $m_2 = m_1 + 2$.

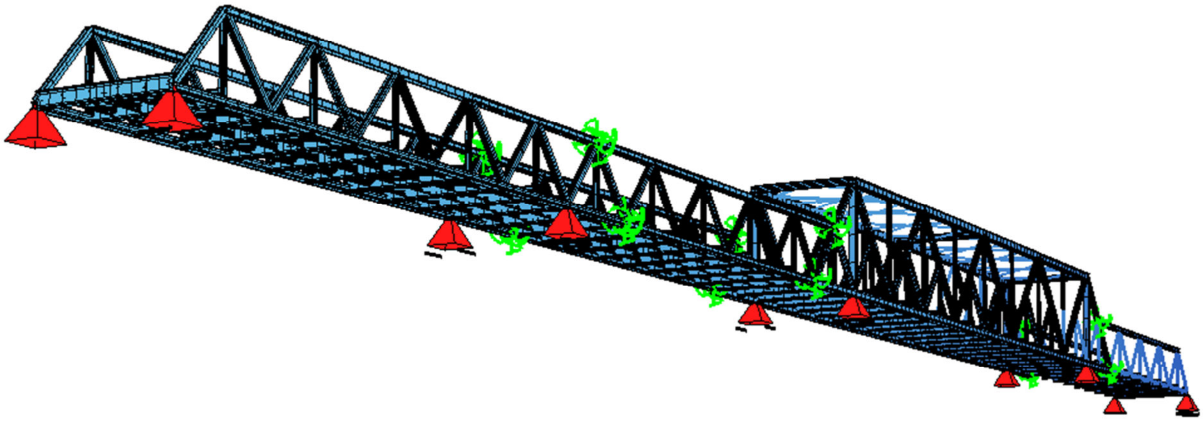
In the presented procedure, among the parameters for the calculations, the most uncertainty is related to the load spectrum data during the bridge operation period. Therefore, these data must necessarily be updated and supplemented.

Calculation models of the bridge structure and fatigue load

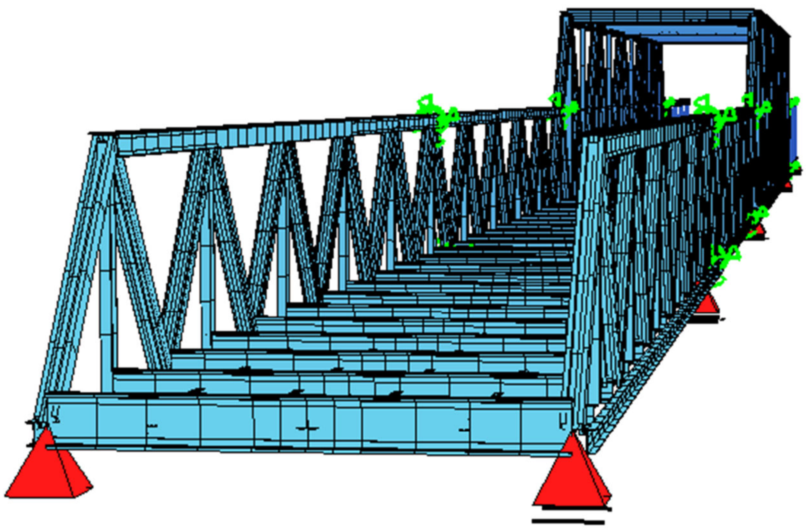
The computational analysis necessary to assess the fatigue life of the bridge structure was performed using the finite element method using numerical models made in the Sofistik 2014 system. The detailed geometry of the steel structure of the spans was mapped using a three-dimensional numerical model of the class (e1, p3), the dimensions of which were determined on the basis of inventory drawings. Elements of the truss girders and the deck were modeled with two-node *beam* elements of the beam type, and the joints with spring elements of the *spri* type in three orthogonal directions, not transmitting bending moments. In line with the accepted practice, all riveted nodal connections were modeled as rigid. All beam elements were discretized individually with a maximum mesh size of approximately 30 mm. An ordered method of meshing was used. The model shows the support of the steel structure of the spans on articulated-sliding and articulated-non-sliding bearings by removing the degrees of freedom from the nodes lying in the places of translational supports in accordance with the actual bearing arrangement of the structure. Structural steel was characterized in the calculations as a linear-elastic isotropic material with two engineering constants E = 210 GPa and ν = 0.3. Due to the type of structure (riveted structure) and a large number of additional elements (rivets, overlays, gusset plates), the own weight of the steel structure was assumed on the basis of the inventoried gross bar sections and increased by 15%. The numerical model of the bridge spans structure is shown in Fig.**5**.

In phase I of the fatigue life assessment, a basic model of railway fatigue loads was adopted, the so-called Model 71 according to PN-EN 1991-2 [11] (Fig.6a). As the existing bridge has two tracks, the load was successively placed on both tracks in the most unfavorable positions. Before performing the calculations in Phase II, analyzes and/or measurements and tests were carried out in order to update and supplement the information on the loads. In phase II, the operational railway loads were simulated using the fatigue load model, containing the composition of normal and light traffic corresponding to freight and passenger trains, respectively. Fatigue loads of type 2 (locomotive pulling a passenger train) and type 8 (locomotive pulling a freight train) were applied according to PN-EN 1991-2 standard [11] (Fig.**6b** and **6c**). The applied fatigue loads are similar in terms of the value of the axle load to the Model D4 according to PN-EN 15528 [10], used to assess the immediate load capacity of the bridge.

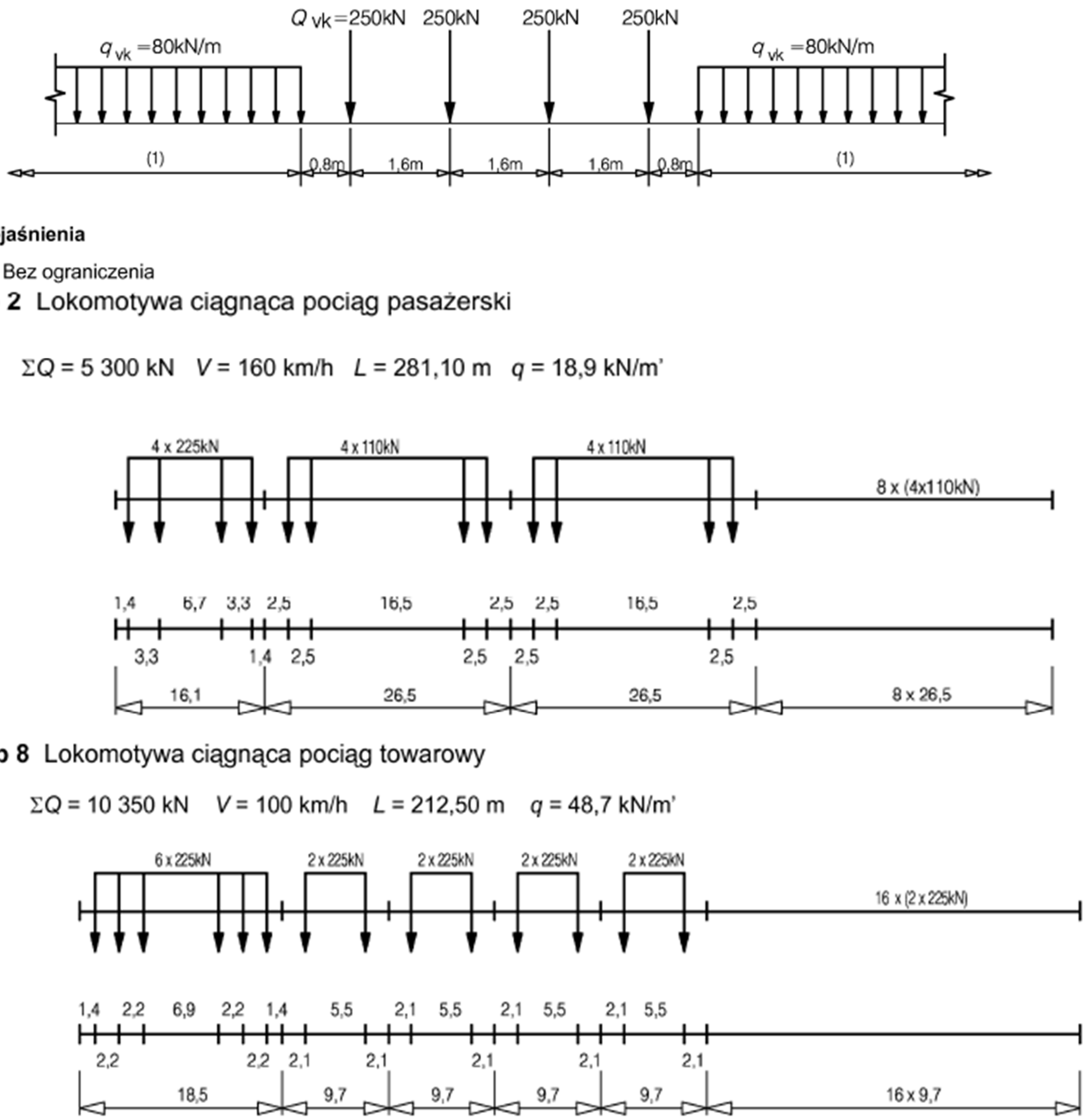
a)



b)



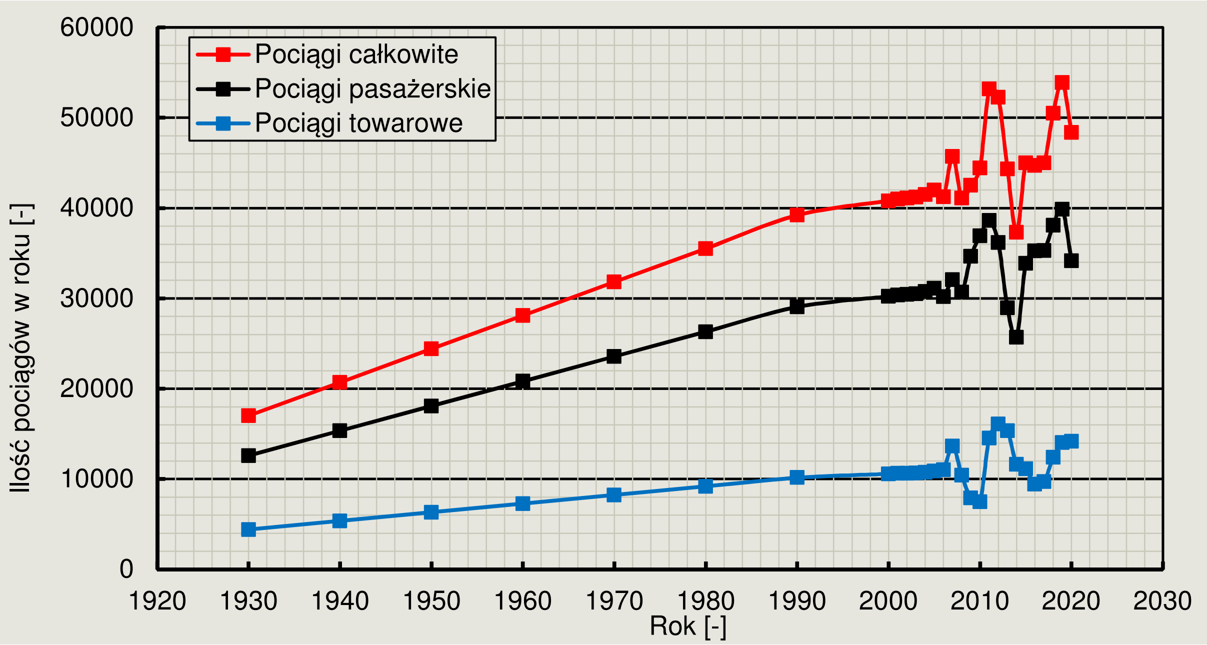
5. Numerical model of the bridge spans structure



6. Railway load models adopted in the fatigue life assessment of the bridge according to PN-EN 1991-2 [11]: a) Model 71; b) type 2 fatigue action model (passenger train); c) Type 8 fatigue interaction model (freight train).

In order to build the spectrum of the bridge loads, it is necessary to know the composition of railway traffic on the bridge during its current operation. The facility manager had such data only for the period from 2006 to 2020. On the basis of these data, rail traffic was extrapolated (backward), assuming the European model of rail load increment in the period 1930 - 2006 according to the guidelines [8]. On the basis of these assumptions, the full spectrum of railway loads of the bridge was determined analytically, divided into passenger and freight trains. The percentage share of freight and passenger trains in the total number of trains for the period before 2006 was adopted as the average value from the 2006-2020 measurement period. For the period 1930-2005, average values over 10-year periods were adopted. The load on one track was taken as half of the total load on the bridge.

The full spectrum of the bridge load with rolling stock in the years 1930-2020 is presented in Fig.**7**. Calculated on the basis of the prepared spectrum, the total number of all trains that passed through the bridge during its operation in the years 1930-2020 is N_{obs} = 2 904 694 trains per bridge, and N_{obs} = 1 452 347 trains per track. In this period, the total share of freight trains was 376 457, and passenger trains 1 075 890. The number of trains determined in this way was used to assess the fatigue life of the bridge in phase II.



7. The spectrum of the actual loads on the bridge in the years 1930 - 2020

Assessment of the fatigue life of the bridge structure

Calculated ranges of stress variability $\Delta\sigma_p$ and $\Delta\sigma_i$ were determined for net cross-sections, i.e. taking into account the rivet holes and taking into account the weakening of the cross-sections due to corrosion. In the calculation of stresses in individual bridge elements, the appropriate load (Fig.6) were placed successively on both tracks in the most unfavorable positions along the length of the bridge. In fatigue calculations, the load factor (characteristic load) was omitted, but dynamic factors were used according to PN-EN 1991-2 [11].

The assessment of the fatigue life in phase I was made taking into account the Model 71 fatigue loads and according to formula (1), assuming the following values:

- $\Delta\sigma_c = 71\ MPa$ – for a joint/rivet element according to the guidelines [8],
- $\Delta\sigma_c = 160\ MPa$ – for an element of rolled sections according to PN-EN 1993-1-9 [10],
- $\gamma_{Ff} = 1{,}0$ – recommended according to PN-EN 1993-2 [12],
- $\gamma_{Mf} = 1{,}35$ – recommended according to PN-EN 1993-1-9 [10] for significant consequences of object destruction.

Damage equivalence factors λ_2 for the bridge, the railway traffic intensity on line section 132 was determined based on data provided by the facility manager over the turn of 14 years, i.e. from 2006 to 2020. However, the planned life of the bridge was assumed to be 50 years (according to the assumptions of the bridge modernization project). The remaining damage factors depending on the geometry of the bridge were assumed according to PN-EN 1993-2 [12].

The calculations of stresses in the elements of the main girder and deck were performed using the described numerical model. The maximum and minimum stresses in the net cross-section of each element were determined and the ranges of stress variability were calculated $\Delta\sigma_p$. For partially or fully compressed elements (upper flange in the span zone, lower flange in the support zone, and compressed diagonals and posts), a reduced equivalent range of stress variability was assumed. $\Delta\sigma_{E,2}$ assuming 60% of the compressive component according to PN-EN 1993-1-9 [10].

For such assumptions and the calculated stress ranges, all-steel structure elements for which μ_{fat} < 1,0. In total, 25 elements at risk of fatigue were identified in the truss girders of the bridge, including 15 sections of the bottom flange and 10 diagonals. In total, for both girders, these numbers are respectively 30 for the sections of the lower chord and 20 for the diagonals. On the other hand, all cross-members (52), stringers (192), and their interconnections were identified in the deck at risk of fatigue. **Therefore, the calculations in phase I showed that a significant part of the steel structure of the bridge is endangered by fatigue.** For the elements indicated in phase I, an extended assessment of the fatigue life (phase II) was carried out, the total fatigue damage was determined and the unconditional fatigue life was checked.

Assessment of the fatigue life in phase II of the bridge elements for which phase I showed a risk of fatigue was made taking into account the models of fatigue loads (type 2 and type 8) and the fatigue strength curves specified in PN-EN 1993-1-9 [10]. Stress variability ranges were determined $\Delta\sigma_i$ on loads from passenger (type 2) and freight (type 8) trains for each critical structure element, indicated in phase I. Determined values $\Delta\sigma_i$ used in the calculation of total fatigue damage D_d according to formula (4) for each of the critical elements in the assumed 50-year life of the bridge.

Sample summary fatigue damage D_d and unconditional fatigue life T_s critical elements of the steel structure of the girders and deck, calculated on the basis of the procedure for Phase II, are given in Tables **1 – 3**. As it results from the calculations, the vast majority of the tested bridge elements have already exhausted their fatigue life (D_d > 1,0) and almost zero unconditional fatigue life (T_s < 1 year). This means that the process of initiation and propagation of fatigue cracks can literally begin in these elements at any moment. The most endangered by this process are the deck connections and the bottom chords of the truss girders. In the case of the deck, the point of crack initiation is the riveted joints of the stringers with the cross members, and in the case of the chords of the lower girders - the net cross-sections in the nodes. The remaining elements checked in phase II are also endangered by the fatigue process and cracks but to a lesser extent.

Tab. 1. Total fatigue damage D_d and unconditional fatigue life T_s critical elements of the bottom chord in the structure of span No.1

<table>
<tr><th>Bottom belt element</th><th>Type of train</th><th>$\Delta\sigma_i$ [MPa]</th><th>$\Delta\sigma_c$ [MPa]</th><th>n_i [-]</th><th>N_{Ri} [-]</th><th>n_i/N_{Ri}</th><th>D_d</th><th>T_s</th></tr>
<tr><td rowspan="2">D1</td><td>Type 2: passenger</td><td>80,86</td><td>71,00</td><td>1075890</td><td>550272</td><td>1,955196</td><td rowspan="2">4,0709</td><td rowspan="2">0,2456</td></tr>
<tr><td>Type 8: freight</td><td>117,81</td><td>71,00</td><td>376457</td><td>177933</td><td>2,11572</td></tr>
<tr><td rowspan="2">D4</td><td>Type 2: passenger</td><td>64,66</td><td>71,00</td><td>1075890</td><td>1076364</td><td>0,99956</td><td rowspan="2">1,6193</td><td rowspan="2">0,6176</td></tr>
<tr><td>Type 8: freight</td><td>78,24</td><td>71,00</td><td>376457</td><td>607466</td><td>0,619717</td></tr>
<tr><td rowspan="2">D6</td><td>Type 2: passenger</td><td>58,41</td><td>71,00</td><td>1075890</td><td>1460088</td><td>0,736867</td><td rowspan="2">3,6535</td><td rowspan="2">0,2737</td></tr>
<tr><td>Type 8: freight</td><td>131,12</td><td>71,00</td><td>376457</td><td>129075</td><td>2,916586</td></tr>
</table>

Tab. 2. Total fatigue damage D_d and unconditional fatigue life T_s critical diagonals in the structure of span no. 3

Diagonals	Type of train	$\Delta\sigma_i$ [MPa]	$\Delta\sigma_c$ [MPa]	n_i [-]	N_{Ri} [-]	n_i/N_{Ri}	D_d	T_s
K1	Type 2: passenger	61,69	71,00	1075890	1239339	0,868116	1,4491	0,6901
	Type 8: freight	76,58	71,00	376457	647914	0,581029		
K2	Type 2: passenger	54,07	71,00	1075890	1840949	0,584421	0,9402	1,0636
	Type 8: freight	65,03	71,00	376457	1058114	0,355781		
K2	Type 2: passenger	43,90	71,00	1075890	3438462	0,312899	1,1713	0,8538
	Type 8: freight	87,21	71,00	376457	438576	0,858363		
K1	Type 2: passenger	56,61	71,00	1075890	1603997	0,670755	1,4955	0,6687
	Type 8: freight	86,06	71,00	376457	456472	0,82471		

Tab. 3. Total fatigue damage D_d and unconditional fatigue life T_s critical crossbars in the structure of span no. 1

Crossbars	Type of train	$\Delta\sigma_i$ [MPa]	$\Delta\sigma_c$ [MPa]	n_i [-]	N_{Ri} [-]	n_i/N_{Ri}	D_d	T_s
P1-P13	Type 2: passenger	60,67	71,00	1075890	1302674	0,825909	1,8698	0,5348
	Type 8: freight	93,09	71,00	376457	360622	1,043909		

As the inspection of the technical condition of the bridge showed, the process of initiation and propagation of fatigue cracks in the pier has already started. Potential knot cracks in the chords of the bottom truss girders may not yet be visible as they are hidden under the gusset plates. However, at any time, after exceeding the critical length, under the influence of operational loads, they may turn into a brittle fracture, leading to sudden failure of the element/spar. This risk is particularly high in the case of the nearly 100-year-old bridge structure, the structural steel of which is characterized by very low impact strength.

Summary and final conclusions

The assessment of the riveted steel structure of the railway bridge over the Odra river in Opole showed that the fatigue life of the steel structure of the bridge is practically exhausted and that the further fatigue life of the structure is zero. This applies to virtually all elements of the platform and most elements of the lattice girders, mainly the bottom chords. The signs confirming the exhausting fatigue life of the bridge are the identified fatigue cracks of the connection elements of the cross-members and stringers. It can be said with a high degree of probability that the fatigue crack initiation process also started in the nodal zones of the lower chords. The lack of their visual identity is due to the fact that these cracks usually develop under the gusset plates. The exhausted fatigue life of the structure combined with the low impact strength of steel can lead to sudden, brittle fractures, threatening the safety of both the bridge structure and its users.

Taking into account the above-mentioned results of the railway bridge over the Odra River in Opole assessment, clearly showed that the existing steel structure **is not suitable for use in the planned modernization/reconstruction** of the bridge and for further operation in the planned 50-year technical life of the structure. Therefore, it was recommended to commence works related to the replacement of the existing steel structure with new spans.

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