FATIGUE STRENGTH
OF AN OVER ONE HUNDRED YEAR OLD RAILWAY BRIDGE

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Abstract. The paper analyzes the fatigue strength of steel sampled from a railway bridge with a service life of over hundred years. The fatigue strength of the material was used to analyze the structural capacity of this railway bridge with such life expectancy operating under variable cyclic loading. The assessment was made using the requirements of the current design codes. It was found that the load-bearing capacity of structures made of materials similar to the analyzed steel may decrease significantly. In case of railway bridges and viaducts made of puddled steel with a service life exceeding 100 years, a significant reduction in the fatigue strength under normal stress is expected. Under shear stress, however, a significant excess of fatigue strength is predicted.

Keywords: puddled steel, old steel, railway bridge, railway viaduct, fatigue, Finite Element Method (FEM) calculation.

1. Introduction

The industrial revolution, which began in the United Kingdom (UK) in the eighteenth century, resulted in the rapid development of communication routes. The opening of the Stockton to Darlington rail link (UK) in 1825 facilitated the construction of railway networks around the world, which frequently involved overcoming various physical obstacles by means of bridges and viaducts. The Iron Bridge at Ironbridge, Shropshire, England was the first bridge in the world to be made of materials other than wood and stone. The structure, made of cast iron, was completed in 1779 and opened to service on 1 January 1781. Later bridges were built of puddled steel and cast steel. During and after the late eighteenth century, bridges and viaducts of this type were erected also in Poland. Many such structures still exist and even operate. The first iron bridge in the European continent was the bridge at Łażany (Laasan), Silesia, Poland, opened in 1796 (Fig. 1)
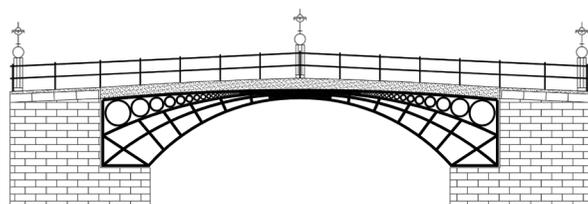
Fig. 1. A view of the first iron bridge in the European continent at Łażany (Laasan) 1796

(Katz 1928). The oldest existing iron bridge in Poland is located at Opatówek, near Kalisz (1824) (Biliszczuk, Hildebrand 2007; Biliszczuk *et al.* 2009), whereas the world's first welded road bridge was built over the river Słudwia at Maurzyce near Łowicz, Poland in 1929.

Since many of such structures are still used, they have a service life longer than 100 years. Obviously, the material properties of objects operating for such a long time have changed, due to aging. In many cases, the load-bearing capacity may have decreased considerably, making a structure unsuitable for safe use. Since railway bridges and viaducts operate under variable cyclic loading, the basic strength parameter responsible for their bearing capacity is the fatigue strength of the material used for the structural elements. Although there are many studies concerning the fatigue evaluation of steel bridges, especially railway bridges (Frýba 1980; Imam *et al.* 2007, 2008; Škaloud, Zörnerová 2005; Wang, Hao 2011) and failure of railway infrastructure (Robinson, Kapoor 2009), it is essential to analyze various problems related to the fatigue strength of old, still operating railway bridges. Studies are necessary to determine the fatigue strength of various structural steels used in Poland since the early nineteenth century and the load-bearing capacity of bridge-type structures, including the actual strength parameters of the material. The strength of steel and the load-bearing capacity of bridge-type structures with a long service life have been analyzed and described

by many Polish researchers (for instance Biliszczuk, Rabiega 1997; Konat *et al.* 2005; Lesiuk, Szata 2010; Wichtowski 2009a). The problem of fatigue life of old steel railway bridges is being examined also in other European countries, for example, by Åkesson (2010). Studies on bridges with long lifespan generally focus on material parameters as these are crucial during bridge reconstruction or renovation (e.g. Paeglitis 2013); the studies frequently involve advanced methods developed and recommended for analysis of bridge emergency states (e.g. Kossakowski 2012a, 2012b; Žiliukas, Gintalas 2011).

The objective of this study was to estimate the fatigue strength of puddled steel used for the structural elements of over hundred years old railway bridge, taking into consideration the requirements of the current design codes. The now-dismantled railway bridge was situated on the route between Radom and Tomaszów Mazowiecki, Poland. The bridge was built in 1885 and operated until 2007 for about 122 years. The results show that the fatigue strength of the analyzed steel is used for determining the load-bearing capacity of railway bridges with a similar life expectancy operating under variable cyclic loading.

2. Fatigue strength according to *EN 1993-1-9:2005*

Variable cyclic loadings acting on structural elements cause variable cyclic stresses and, consequently, a decrease in the fatigue strength. Although the problem of fatigue of metals is fairly well recognized in the literature (Kocańda 1985), the research into the fatigue phenomena is being continued and the methods of analysis are being improved in various fields of technology (for example Juchnevičius, Krenevičius 2011; Kala 2008; Krenevičius, Leonavičius 2008; Stonkus *et al.* 2009, 2011).

There are two ranges of fatigue depending on the load and durability:
- low-cycle fatigue, when large loads lead to low durability of elements;
- high-cycle fatigue, occurring at low loading levels, which results in high durability of elements.

Both types of fatigue are characteristic of different mechanisms of failure, according to loading. The dividing line between the low-cycle and the high-cycle fatigue is determined at the point where the fatigue curve, plotted in the natural coordinates, goes from convex to concave. The design codes do not take account of the low-cycle fatigue; only the high-cycle fatigue, whose point of origin is assumed to be $N = 10^4$ cycles, is included. The Wöhler curve, showing the relationship between the high-cycle fatigue strength $\Delta\sigma$ and the number of cycles N, was plotted in the logarithmic coordinates. The non-linear relationship is thus replaced by linear sections $\Delta\sigma(N)$.

In *EN 1993-1-9:2005 Eurocode 3: Design of Steel Structures, Part 1–9: Fatigue* it is assumed that the fatigue curve $\Delta\sigma(N)$ consists of several sections, limited by the characteristic values of the number of cycles, N (Fig. 2). The first two sections cover the ranges of the so-called limited fatigue strength $\Delta\sigma_R$, dependent on the number of cycles, N. The first section of the $\Delta\sigma_R(N)$ curve covers the range $N = 10^4 \div 2\times10^6$. For $N_C = 2\times10^6$, laboratory tests are conducted to determine the so-called standard fatigue strength $\Delta\sigma_C$, also referred to as fatigue category. It is the numerical value assigned to a specific element and a specific stress direction that indicates the curve to be used for assessing fatigue (*EN 1993-1-9:2005*).

The second section represents the range of the limited fatigue strength $\Delta\sigma_R$ for $N = 2\times10^6 \div 5\times10^6$, with the number of cycles $N_D = 5\times10^6$ corresponding to the constant-amplitude permanent fatigue strength $\Delta\sigma_D$. It is the limit for the range of variation of normal stresses under constant amplitude below which there is no fatigue damage. The last section of the fatigue curve represents the range of unlimited (permanent) fatigue strength $\Delta\sigma_L$, which is independent of the number of cycles N. Below the cut-off limit $\Delta\sigma_L$, defined by the number of cycles $N_L = 10^8$, the stress variation does not affect the accumulation of damage.

Fig. 2 shows curves of fatigue strength for direct and shear stress ranges corresponding to the categories of
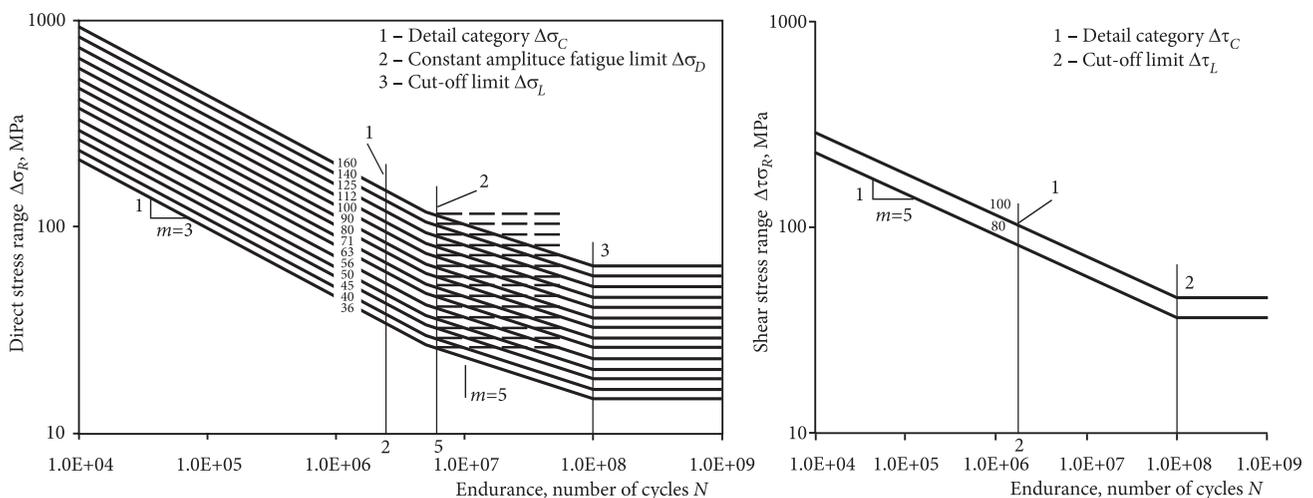
Fig. 2. Fatigue strength curves for the direct and shear stress ranges according to *EN 1993-1-9:2005*
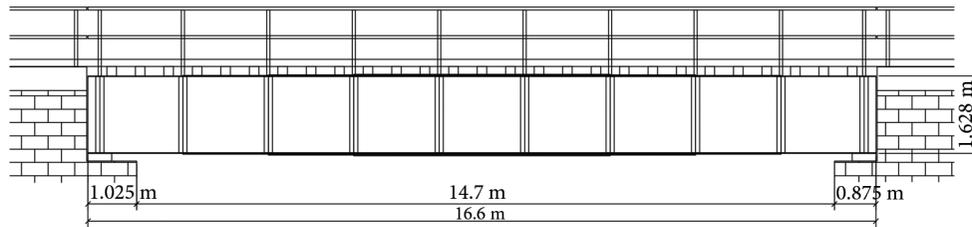
Fig. 3. A side view of the century-old railway steel bridge which operated on the Radom-Tomaszów Mazowiecki route in Poland
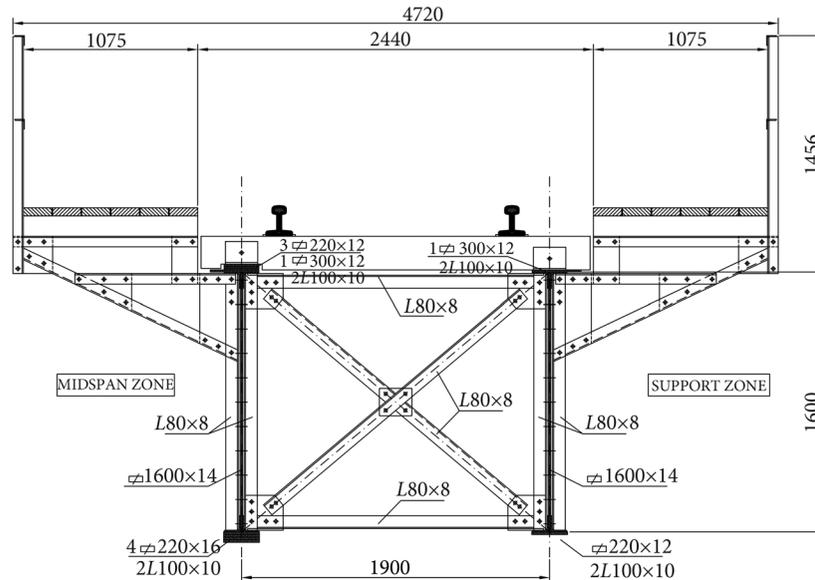

Fig. 4. A cross-sectional view of the railway bridge, units in mm

fatigue $\Delta\sigma_C = 36\text{-}160$ MPa and $\Delta\tau_C = 80\text{-}100$ MPa for various structural elements included in Tables 8.1–8.10 of *EN 1993-1-9:2005*.

Fatigue strength, expressed as a range of variation of normal stresses, is represented by the $(\log\Delta\sigma_R) - (\log N)$ curves for each category of fatigue. According to *EN 1993-1-9:2005*, the following relationships are used to define fatigue strength $\Delta\sigma_R$ under constant amplitude loading for different ranges of the number of cycles *N*:

$$\Delta\sigma_R^m \, N_R = \Delta\sigma_C^m \, 2\times10^6 \text{ with } m = 3 \text{ for } N < 5\times10^6, \quad (1)$$

$$\Delta\sigma_R^m \, N_R = \Delta\sigma_D^m \, 5\times10^6 \text{ with } m = 5 \text{ for}$$
$$5\times10^6 \leq N \leq 1\times10^8, \quad (2)$$

$$\Delta\sigma_L = 0.05^{0.2} \, \Delta\sigma_D \text{ for } N > 1\times10^8. \quad (3)$$

3. Fatigue strength of a century-old railway steel bridge

3.1. The railway bridge design

The aim of the study was to estimate the fatigue strength of steel used for a railway bridge erected in 1885 and had operated on the Radom-Tomaszów Mazowiecki route (Poland) until 2007, with a service life amounting to 122 years.

The bridge consisted of two simply-supported steel girders with a total length of 16.6 m and transverse beams spaced at 1.9 m intervals (Figs 3, 4). The bridge elements were connected by rivets. The I-beam girders were composed of 1600×14 mm webs (sheet metal panels) and flanges made up of double $L100\times10$ angle sections connected with 300×12/220×12 mm and 220×16 mm metal panels at the top and bottom, respectively. In the support zone, one panel was used per flange, while in the midspan zone 4 panels were used per flange. The girders were braced by cross members made up of $L80\times8$ angle sections in the shape of the letter X.

A visual inspection showed that the railway bridge was in very poor condition (Fig. 5). The major cause of damage was corrosion of steel elements. Material deterioration was significant and clearly visible in a cross-section. A decision was made to dismantle the bridge and analyze some of the structural elements.

3.2. The fatigue tests

The tests involved assessing the fatigue strength of steel sampled from an over-a-century-old railway bridge and reference structural steel, St3S. Two sets of samples were examined:

– railway bridge steel, marked StM ($b\times h = 20\times11$ mm),
– reference structural steel, St3S ($b\times h = 20\times12$ mm).

The loading was applied to flat, un-notched samples with a rectangular cross section *b×h*. The tensile fatigue tests were conducted under constant amplitude loading. The recorded quantities included the force, the traverse displacements and the number of cycles until failure.

The results, i.e. the stress variation in the function of the number of cycles, obtained prior to sample failure, were used to plot the fatigue curves $(\log\Delta\sigma_R)$ – $(\log N)$ for each test series. The linear regression lines were drawn based on the distributions of the measurement points, $(\log\Delta\sigma_R)$ – $(\log N)$. The test results were also used to estimate the fatigue category of the materials, i.e. to determine the values of the stress variation $\Delta\sigma_C$ for $N_C = 2\times10^6$. The analysis was carried out following the guidelines by Wichtowski (2009b), taking account of the standard deviation, and, for a small sample size, the critical values of the Student's *t* distribution with the *n*–2 degree of freedom and a significance level of 0.05.

The standard fatigue strength $\Delta\sigma_C$ was determined based on the following equation:

$$\Delta\sigma_C = \Delta\sigma_P \left(\frac{2\times10^6}{N_P} \right)^{\frac{1}{\beta}}, \qquad (4)$$

where β – the regression factor; N_P – the number of cycles, corresponding to the range of stress $\Delta\sigma_P$.

The values of the fatigue categories $\Delta\sigma_C$ were approx 166 MPa and 210 MPa for the StM and St3S steel samples, respectively. The values of $\Delta\sigma_D$ and $\Delta\sigma_L$ calculated from Eqs (1)–(3) were used to plot fatigue curves $(\log\Delta\sigma_R)$ – $(\log N)$ consisting of three characteristic sections (Fig. 6).

The fatigue category of StM steel was about 20% lower than that of St3S steel. The value of the fatigue strength established for St3S steel $(\Delta\sigma_C = 210$ MPa) was used as a benchmark for assessing the fatigue category of the railway bridge steel, StM. In order to maintain adequate level of safety, it is proposed that the corrected categories of fatigue test $\Delta\sigma_C^*$ of the railway bridge steel, StM, are based on the values of $\Delta\sigma_C$ given in *EN 1993-1-9:2005*, using a reduction factor of 0.8 according to the formula:

$$\Delta\sigma_C^* = 0.8 \, \Delta\sigma_C. \qquad (5)$$

The corrected fatigue strength $\Delta\sigma_C^*$ defined by formula (5) should be applied to assess the fatigue of the structural elements of the analyzed old railway bridge.

3.3. Microstructural analysis

The first objective of the material analysis was to obtain microstructural images of the bridge steel with a ferritic-pearlitic matrix (Fig. 7a). There were numerous non-metallic inclusions, mainly sulphides and brittle oxides. The ferrite grains (white) were uniaxial in nature, whereas, the pearlite grains (dark) were elongated, as a result of metal working (rolling). The pearlite content was estimated to be about 10–20%. The elongation of the pearlite grains indicates the direction of the rolling operation. Locally, there were some elongated grey patterns, which may be sulphide

Fig. 5. A severely corroded railway bridge element
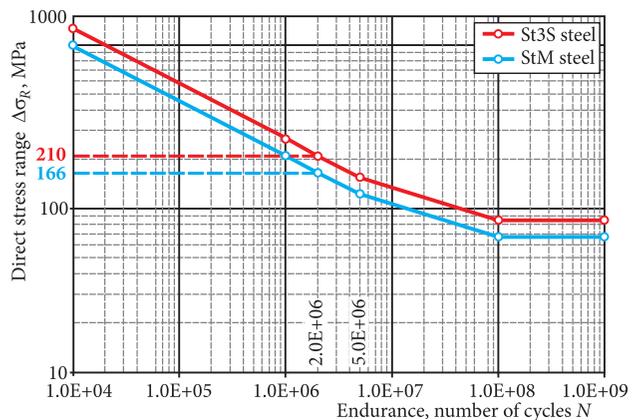

Fig. 6. Fatigue strength curves for the railway bridge steel, StM, and the reference structural steel, St3S

precipitates, strongly deformed by heat treatment. In the cross- and longitudinal sections, the structure quality was identical. The content of pearlite was lower in the surface zone than in the middle zone.

The analysis also showed that fractures occurred in areas where plastic deformation was considerable, its extent being comparable to the sample size in the cross-section. The fatigue fractures of the railway bridge steel, StM, were flat and their plane was oriented to the load direction. The residual fractures, however, had a rough non-uniform structure with clear granulation (Figs 7b and 7c). The surface of the whole residual fracture did not form a single plane; it consisted of several areas of valleys and peaks. The planes were oriented at different angles in relation to the load direction. The angles were small and ranged α=0–10°. The residual part of the fractures of the StM steel was classified as brittle fracture, according to suggestions by Kocańda (1985). In case of the railway bridge steel, StM, delamination was clearly visible (Figs 7b, 8a); it covered the entire width of the samples and the cracks extended from the surface in the direction parallel to the direction of the load to a depth of several millimetres.

A qualitative analysis was also carried out to determine chemical composition of the bridge steel. In addition to iron and carbon, the material contains significant amounts of manganese and a small amount of sulphur. Considering
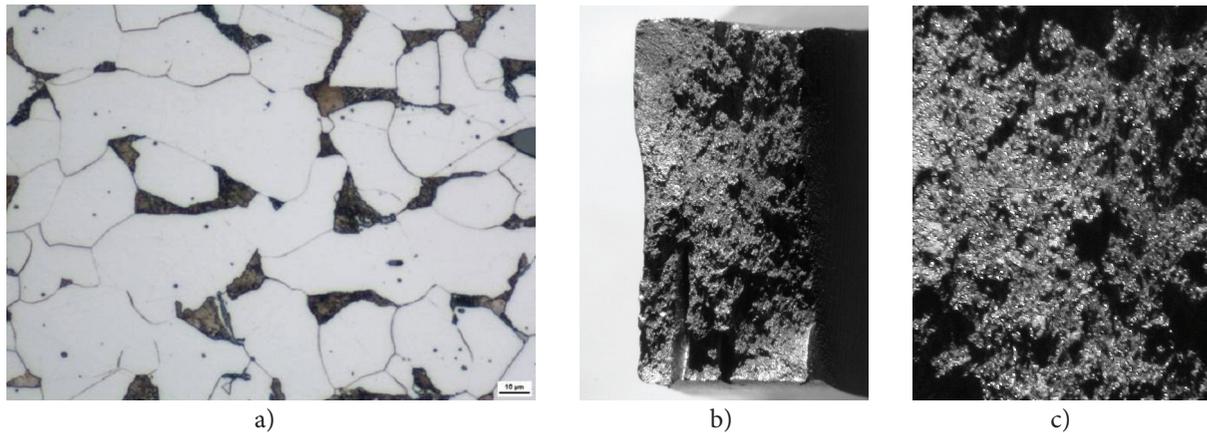
Fig. 7. Microstructure (a), fracture (b) and macrostructure (c) of the railway bridge steel, StM
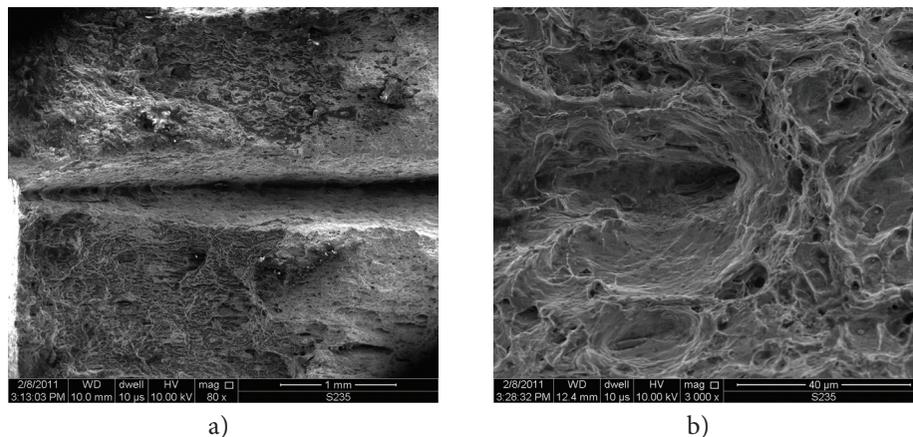

Fig. 8. Outer delamination (a) and microstructure (b) of the railway bridge steel, StM
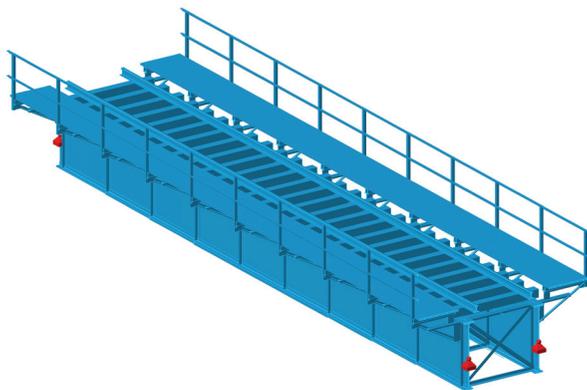

Fig. 9. A FEM model of the analyzed bridge

the age, structure and chemical composition of the material as well as the distribution of lamellar fractures, it is certain that the tested steel corresponds to puddled steel, which was commonly used in bridge engineering in Poland in the late nineteenth century.

4. Determination of the fatigue stress range spectrum

The fatigue strength analysis required static calculations. The bridge was modelled as a beam structure using Autodesk Robot Structural Analysis 2010, which included a FEM analysis (Fig. 9).

The Load Model 71 was used to perform a fatigue analysis according to *EN 1991-2:2003 Eurocode 1: Actions*

on Structures – Part 2: Traffic Loads on Bridges. The Load Model 71 represents the static effect of vertical loading due to normal rail traffic. The characteristic values of loads shall be multiplied by factor α on lines carrying rail traffic, which is heavier or lighter than normal rail traffic. In this case, the movement of trains corresponds to normal rail traffic, thus factor α was assumed to be 1.0. The load distribution and the characteristic values of vertical loads applied to rails are shown in Fig. 10.

The damage effects of the stress range spectrum represented by the damage equivalent stress range related to 2×10^6 cycles was defined as:

$$\Delta \sigma_{E,2} = \lambda \, \Phi_2 \, \Delta \sigma_p, \qquad (6)$$

where λ – the damage equivalence factor; Φ – the damage equivalent impact factor.

The damage equivalence factors λ for railway bridges were defined as follows:

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4, \text{ but } \lambda \leq \lambda_{max} = 1.4, \qquad (7)$$

where λ_1 – the factor used for different types of girder; it takes account of the damage effect of traffic and depends on the length of the influence line or area; λ_2 – the factor that takes account of the traffic volume; λ_3 – the factor that

takes account of the design life of the bridge; λ_4 – the factor to be applied when a structural element is loaded by more than one track.

For standard rail traffic (EC Mix) and a simply supported beam with the span length L_Φ = 16.0 m, λ_1 was determined to be equal to 0.736. The traffic volume per year was assessed as 25×10^6 t / track, thus, factor λ_2 was 1.0. The service life of the bridge was almost 122 years, thus factor λ_3 was assessed as 1.04. As only one train was able to cross the bridge at a time, factor λ_4 was assumed to be 1.0. To sum up, the damage equivalence factors λ for the analyzed bridge were:

$$\lambda = 0.736 \times 1.0 \times 1.04 \times 1.0 = 0.765 \ \leq \lambda_{max} = 1.4. \quad (8)$$

Determining the dynamic factor required taking account of the dynamic magnification of stresses and vibration effects in the structure; the resonance effects were not considered. The dynamic factor Φ_2, which enhances the static load effects under Load Models 71 for a simply supported beam and the span length L_Φ = 16.0 m, is:

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi - 0.2}} + 0.82 = 1.20 \ \text{and} \ 1.0 \leq \Phi_2 \leq 1.67. \quad (9)$$

Thus, the damage effect of the stress range spectrum $\Delta\sigma_{E,2}$ was defined as:

$$\Delta\sigma_{E,2} = \lambda\,\Phi_2\,\Delta\sigma_p = 0.918\,\Delta\sigma_p. \quad (10)$$

In non-welded or stress-relieved welded elements, the effect of the mean stress on the fatigue strength is taken into consideration. Fatigue assessment will require determining the reduced effective stress range $\Delta\sigma_{E,2}$ for part of or all of the compressive stress cycles. The effective stress range may be calculated by adding the tensile portion of the stress range and 60% of the magnitude of the compressive portion of the stress range. In case of the analyzed bridge, the mean girders were totally compressed in the upper part, and the effective stress range $\Delta\sigma_{E,2}$ for this part was reduced.

The values of the normal stress range spectrum $\Delta\sigma_{E,2}$ and the shear stress range spectrum $\Delta\tau_{E,2}$ for the main girders of the analyzed bridge are shown in Figs 12 and 13.

The extreme values of the stress range spectrum were:

$$\Delta\sigma_{E,2} = 57 \ \text{MPa}, \quad (11)$$

$$\Delta\tau_{E,2} = 18 \ \text{MPa}. \quad (12)$$

These values are reference values necessary to estimate the fatigue of the bridge structural elements, which will be presented in the next section.
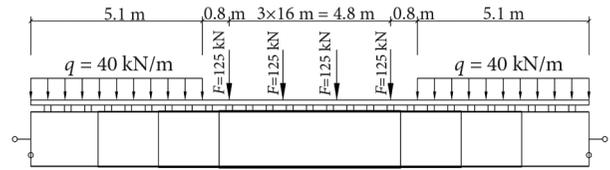

Fig. 10. Load Model 71 applied to the analyzed bridge model
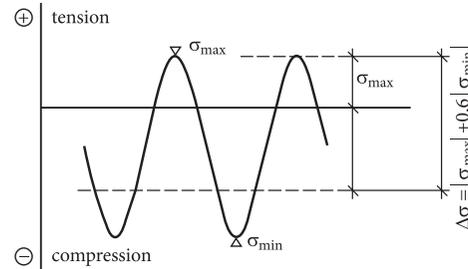

Fig. 11. Stress reduction of the compressive portion of the stress range according to *EN 1993-1-9:2005*

5. The fatigue strength of the analyzed railway bridge

The fatigue was assessed as follows:

$$\frac{\gamma_{Ff}\,\Delta\sigma_{E,2}}{\dfrac{\Delta\sigma_c}{\gamma_{Mf}}} \leq 1, \quad (13)$$

$$\frac{\gamma_{Ff}\,\Delta\tau_{E,2}}{\dfrac{\Delta\tau_c}{\gamma_{Mf}}} \leq 1, \quad (14)$$

where γ_{Ff} – the partial factor for the equivalent constant amplitude stress ranges $\Delta\sigma_E$, $\Delta\tau_E$; γ_{Mf} – the partial factor for fatigue strength $\Delta\sigma_C$, $\Delta\tau_C$.

According to *EN 1993-1-9:2005*, the partial factor for the equivalent constant amplitude stress ranges should be assessed as γ_{Ff} = 1.0. For the 'safe life' assessment with high consequence of failure, the partial factor for fatigue strength $\Delta\sigma_C$ should be equal to γ_{Ff} = 1.35.

As mentioned above, the fatigue strength of the tested material, $\Delta\sigma_C$ and $\Delta\tau_C$, should be reduced according to formula (5). Consequently, fatigue conditions (13) and (14) are simplified to:

$$\frac{\gamma_{Ff}\,\Delta\sigma_{E,2}}{\dfrac{\Delta\sigma_c}{\gamma_{Mf}}} = \frac{1.0\,\Delta\sigma_{E,2}}{\dfrac{0.8\,\Delta\sigma_c}{1.35}} = \frac{1.69\,\Delta\sigma_{E,2}}{\Delta\sigma_c} \leq 1, \quad (15)$$

$$\frac{\gamma_{Ff}\,\Delta\tau_{E,2}}{\dfrac{\Delta\tau_c}{\gamma_{Mf}}} = \frac{1.0\,\Delta\tau_{E,2}}{\dfrac{0.8\,\Delta\tau_c}{1.35}} = \frac{1.69\,\Delta\tau_{E,2}}{\Delta\tau_c} \leq 1. \quad (16)$$

For a structural element with one-sided bolted connection, the detail category corresponds to the reference value of the fatigue strength $\Delta\sigma_C$ = 80 MPa. The fatigue condition is:
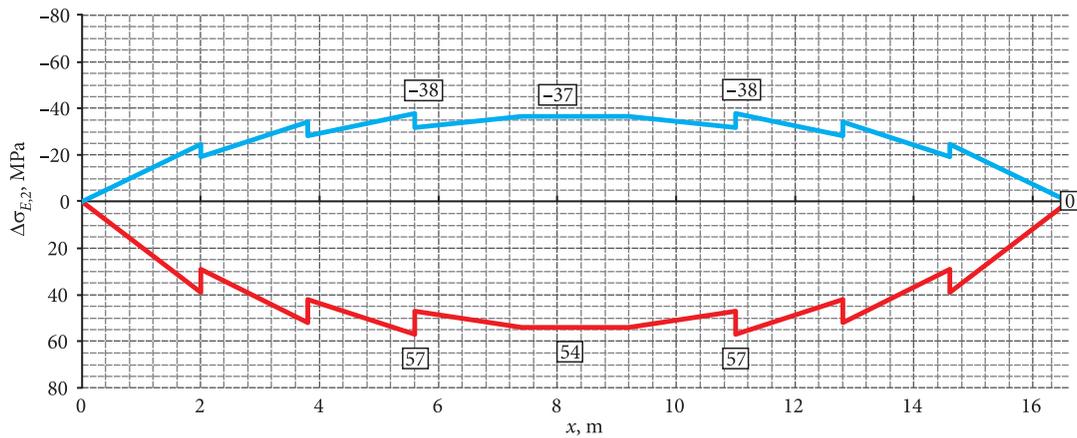
Fig. 12. Distribution of the normal stress range spectrum $\Delta\sigma_{E,2}$ for the main railway bridge girders
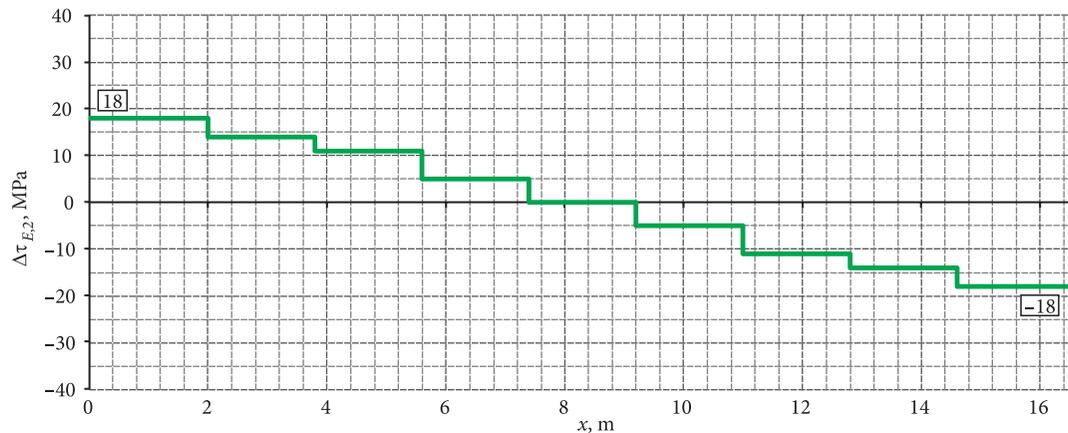

Fig. 13. Distribution of the shear stress range spectrum $\Delta\tau_{E,2}$ for the main railway bridge girders

$$\frac{1.69\,\Delta\sigma_{E,2}}{\Delta\sigma_c} = \frac{1.69\times 57}{80} = 1.20 > 1. \tag{17}$$

For shearing, the detail category is $\Delta\tau_C = 80$ MPa. The fatigue condition is:

$$\frac{1.69\,\Delta\tau_{E,2}}{\Delta\tau_c} = \frac{1.69\times 18}{80} = 0.38 < 1. \tag{18}$$

In case of normal stress, fatigue condition (13) was exceeded by over 20%. If the bridge had continued to operate, a failure and disaster would have been inevitable. For shearing, there was no excessive fatigue. The extra fatigue strength was over 60%.

6. Conclusions

The paper presents results of the research on the fatigue strength of steel sampled from a railway bridge with a service life of over a hundred years. The following conclusions were drawn.

1. The standard fatigue strength of the tested material is used for structures with a similar service life operating under cyclically variable loads. The results indicate that the fatigue strength of structures made of material with a capacity similar to the tested material should be reduced significantly, i.e. by about 20%, using the detail category, according to *EN 1993-1-9:2005*.

2. A significant decrease in the fatigue strength under normal stress is expected in case of railway bridges and viaducts made of puddled steel with a service life exceeding 100 years. For the analyzed bridge, the fatigue condition under normal stress was exceeded by over 20%.

3. Railway bridges and viaducts with a service life exceeding 100 years should be monitored continuously. As demonstrated above, the analyzed bridge would have failed or collapsed if it had continued to operate.

4. Under shear stress, the fatigue condition was not exceeded. For railway bridges with a similar design, a significant excess of fatigue strength is expected. Shearing should not cause any fatigue failure.

5. For steel grades similar to those analyzed here, it is predicted that the structure will delaminate and that will lead to fatigue failure. This observation may be helpful in the inspection and assessment of safety of old railway facilities.

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