Fatigue performance evaluation of steel arch bridge based on experimental tests in the light of increased operating loads

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Abstract. The paper presents a case study of experimentally supported fatigue performance evaluation of over 22-year old composite (steel-concrete) arch bridge. The bridge is a landmark road structure crossing the Narew River in Ostrołęka in Poland. It's main span is composed of through arch static system. The structure was designed to carry out the local town traffic. The bridge due to the emergency condition of the neighboring bridge located along national roadway was temporarily subjected to increased live loads. Thereby it became to play an important role in the transportation system of the whole region being intensively exploited by heavy vehicles in many cases neglecting compliance with restrictions for total weight of heavy trucks. Thus, taking into account limited load capacity and intensity of the traffic crossing the bridge, some doubts about the fatigue performance of bridge critical members were formulated. Fatigue performance evaluation of the steel superstructure was executed according to general assumptions of the Eurocodes and the *European Recommendations for Estimation of Remaining Fatigue Life of Existing Steel Structures*. Essential input data used for estimation of fatigue damage accumulation in critical bridge components as well as for prediction of residual lifetime of the whole structure was based on experimental vibration tests performed under operating loads.

## 1. Bridge structure and objectives of the study

The investigated structure called the Antoni Madaliński Bridge is located over the Narew River in Ostrołęka city. It consists of 4 spans with the lengths: 32 + 32 + 110 + 32 meters. Shorter spans are built of beam type superstructures, each constructed of 4 steel plate girders, 2,5 m high, composed with typical RC slab. The main span is the form of through arch static system constructed of composite–steel-concrete deck connected with the central steel arch by means of cable stays. The structure was designed to carry out the local town traffic. According to the system of the Polish national standards, which was used for the bridge design, the total weight of single vehicle crossing the bridge should not exceed 30 tonnes. The structural form of the investigated bridge is presented in figure 1 and figure 2.

In the years 1996-2016, due to its limited load capacity and in order to keep the heavy traffic out of town, the bridge was used only for carry out vehicles of total mass do not exceeding 3,5 tonnes. However, in the end of 2016 the neighboring bridge located along national road No 61 was closed for service and reconstructed due to its emergency condition. Thereby, the considered structure was subjected to increased live loads and become a very important element in the transportation system of the whole region. Thus, the bridge owner temporarily increased admissible weight of vehicles to 30

tonnes (designed load capacity). Taking into account intensity of the heavy traffic crossing the bridge – in many cases apparently neglecting compliance with weight restrictions for heavy vehicles introduced by bridge owner – some doubts about the fatigue performance of bridge critical members were formulated [1].

In September of 2017 intensity of the traffic on the bridge has been significantly decreased as admissible weight of vehicles was again limited to 3,5 tonnes when the alternative crossing was put to service.
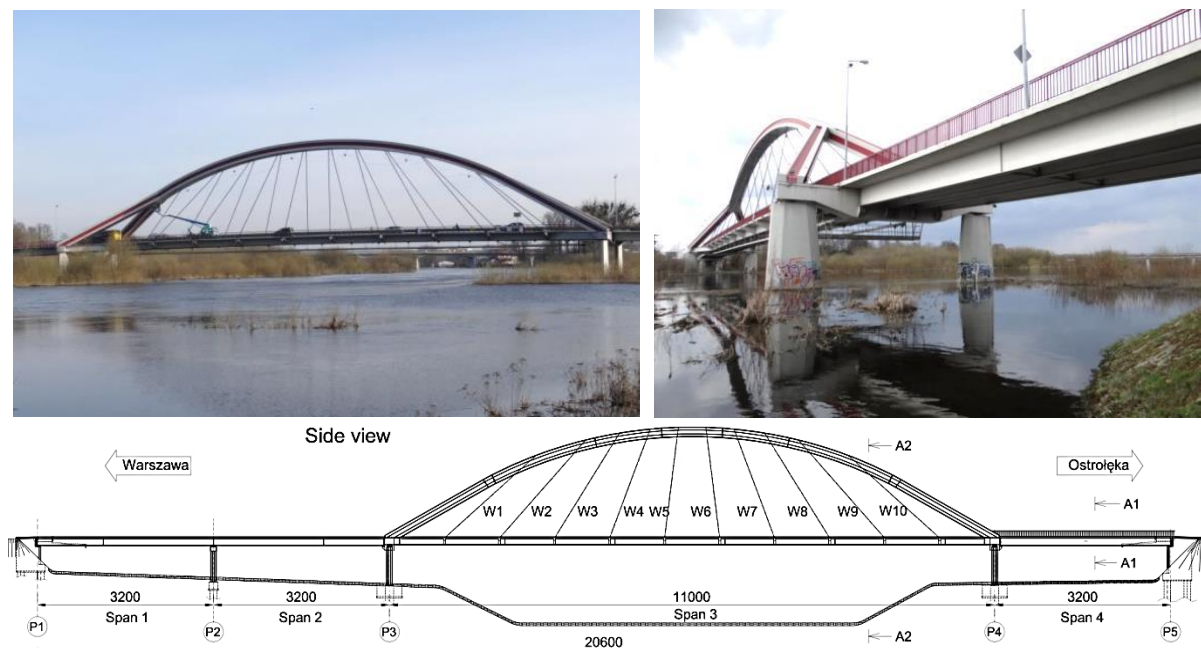
Figure 1. Structural form of the Antoni Madaliński Bridge over the Narew River in Ostrołęka.
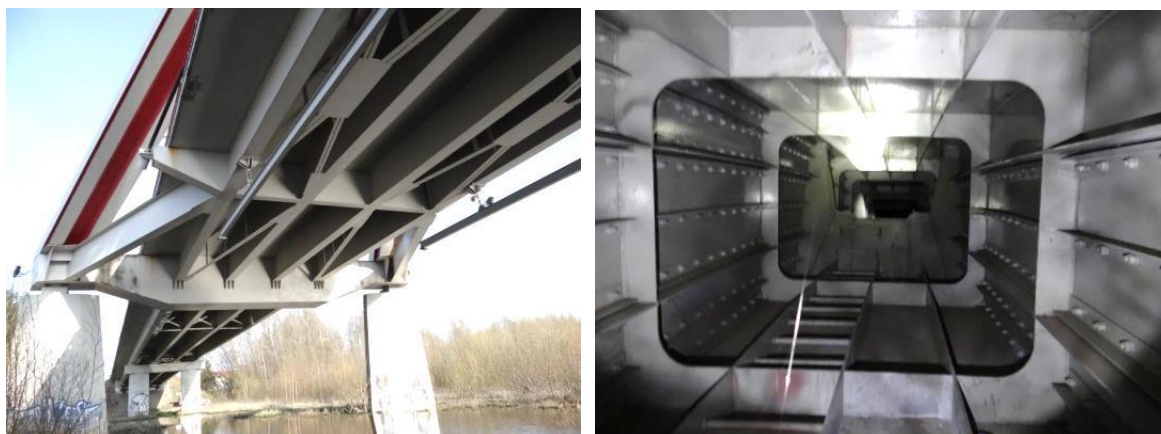
Figure 2. Investigated bridge: a) bottom view of superstructure, b) inside view of the arch.

In the course of special inspection some defects of the bridge superstructure were observed, however fatigue cracks were not detected. Anyway, considering the intensity of heavy traffic crossing the bridge during the period of excessive use, some uncertainties about the fatigue performance of the bridge critical members appeared. Thus, the main goal of the presented investigation was to estimate the level of fatigue damage accumulated in the critical bridge components during the approximate 9

months period of increased heavy traffic as well as to predict the remaining life time of the whole structural system.

2. General procedure of fatigue performance evaluation

The fatigue evaluation of the Antoni Madaliński Bridge was performed following the general rules of the European design codes [2]-[3] and recommendations [4]-[5]. The assessment is aimed at providing evidence that the bridge, after a period of excessive use in 2017, will function safely over a specified residual service life. The applied approach is based on a three-step procedure described below.

Phase I – a preliminary evaluation – aimed at identification of critical members in the structure for which a fatigue hazard exists. It was performed by means of the Finite Element Method (FEM) model of the bridge developed on the basis of data coming from the structural drawings supplemented by a site investigations. Within the initial calculations the Fatigue Load Model 3 (FLM3) of the Eurocode 1[2] was applied and its effects in bridge steel members were referred to the S-N curve corresponding to relevant detail fatigue categories according to [3]. The initial evaluation of the fatigue safety level was carried out according to the formula:

$$\mu_{\text{fat}} = \frac{\Delta\sigma_{\text{C}}}{\gamma_{\text{Mf}} \cdot \gamma_{\text{Ff}} \cdot \Delta\sigma_{\text{E},2}} \geq 1 \qquad (1)$$

where: μ_{fat} – fatigue safety level; $\Delta\sigma_{\text{C}}$ – reference value of the fatigue strength corresponding to $N_{\text{c}} = 2 \cdot 10^{6}$ cycles; $\Delta\sigma_{\text{E},2}$ – equivalent constant amplitude stress range related to $N_{\text{c}} = 2 \cdot 10^{6}$ cycles; γ_{Mf} – partial factor for fatigue strength $\Delta\sigma_{\text{C}}$; γ_{Ff} – partial factor for equivalent constant amplitude stress ranges $\Delta\sigma_{\text{E},2}$.

Rules defining values of the above mentioned parameters are given in [2]-[3]. The equivalent constant amplitude stress range $\Delta\sigma_{\text{E},2}$ can be calculated as follows:

$$\gamma_{\text{Ff}} \cdot \Delta\sigma_{\text{E},2} = \lambda \cdot \Delta\sigma_{\text{p}}(\gamma_{\text{Ff}}, Q_{\text{k}}) \qquad (2)$$

where: $\Delta\sigma_{\text{p}}(\gamma_{\text{Ff}}, Q_{\text{k}})$ – the maximum stress range generated by fatigue loads in an analysed bridge member; λ – total damage equivalent factor determined according to [3]; Q_{k} – characteristic value of a single variable action generated by FLM3 [2].

When the calculated fatigue safety level $\mu_{\text{fat}} < 1$ for a structural component then such a member is classified as a critical element. Its safety needs to be further assessed in Phase II.

Phase II – a detailed fatigue performance investigation based on structure monitoring under increased live loads – is focused on estimation of the total fatigue damage $D_{\text{d}}^{\text{exc}}$ being accumulated during the 9 months period of excessive use in the critical bridge components. The main input data at this phase are results of stress/strain measurements carried out on site in bridge critical elements under increased traffic loads covering a representative (minimum 24-hours) period of the structure operation. The measured strain history in the selected steel components (supplemented, if relevant, by appropriate statistical and projected extrapolations) was used to create the real stress range spectrum representing bridge current, excessive operation conditions.

The measurement results were also used in the FEM model validation. In Phase II the accumulated fatigue damage in the selected structural members was calculated using the Palmgren-Miner damage summation rule, simply stated as follows:

$$D_{\text{d}}^{\text{exc}} = \sum \frac{n_{\text{E}i}}{N_{\text{R}i}} \leq 1 \qquad (3)$$

where: $D_{\text{d}}^{\text{exc}}$ – the total damage accumulated during the 9 months period of excessive use in an analysed spot of an evaluated steel member; $n_{\text{E}i}$ – number of cycles occurring at stress range $\gamma_{\text{Ff}}\Delta\sigma_i$ for band i in the factored spectrum; $N_{\text{R}i}$ – the endurance (expressed in cycles) obtained from the factored $\frac{\Delta\sigma_{\text{C}}}{\gamma_{\text{Mf}}}$ - N_{R} curve corresponding to a stress range of $\gamma_{\text{Ff}}\Delta\sigma_i$.

Phase III – a remaining fatigue life assessment after the period of excessive use – is aimed at estimation of remaining fatigue life T_{fat} considering all (past, present and expected future) periods of

the structure operation, leading to damage accumulation. The prognosis of the fatigue damage $D_{\mathrm{d}}(t)$ in the next t years may be determined using formula:

$$D_{\mathrm{d}}(t) = D_d^{\mathrm{past}} + D_{\mathrm{d}}^{\mathrm{exc}} + D_{\mathrm{d}}^{\mathrm{fut}} \cdot \beta(t) \cdot t \qquad (4)$$

where $D_{\mathrm{d}}^{\mathrm{past}}$ represents the estimated total damage accumulated during the bridge lifetime (excluding the period of increased traffic in 2017) considering natural (site specific) traffic conditions; $D_{\mathrm{d}}^{\mathrm{fut}}$ represents predicted damage accumulation for one year of bridge normal operation at future traffic conditions (admissible weight of vehicles is now again limited to 3.5 tonnes) and $\beta(t)$ is a time-dependent factor considering average changes in volume of freight vehicles crossing the bridge.

In presented case study both $D_{\mathrm{d}}^{\mathrm{past}}$ and $D_{\mathrm{d}}^{\mathrm{fut}}$ parameters were assumed to very small (close to zero) since for corresponding lifetime the bridge was subjected to lightweight traffic only which do not contribute to fatigue damages.

Finally, assuming in formula (4) $D_{\mathrm{d}}(t) = 1$ and $t = T_{\mathrm{fat}}$, the remaining fatigue life time T_{fat} can be obtained as:

$$T_{\mathrm{fat}} = \frac{1 - D_{\mathrm{d}}^{\mathrm{past}} - D_{\mathrm{d}}^{\mathrm{exc}}}{\beta(T_{\mathrm{fat}}) \cdot D_{\mathrm{d}}^{\mathrm{fut}}} \qquad (5)$$

3. Assessment Phase I

For the purpose of the fatigue evaluation of the structure included in Phase I of the presented procedure a 3 different numerical FEM models representing separately bridge superstructures (beam type spans and arch spans were evaluated separately) were created (figure 3). Each model is composed of one- and two-dimensional finite elements: one-dimensional beam type elements are utilized for representation of all steel components including bracings and stay cables, while two-dimensional shell type elements are used for modelling of the RC deck slab.
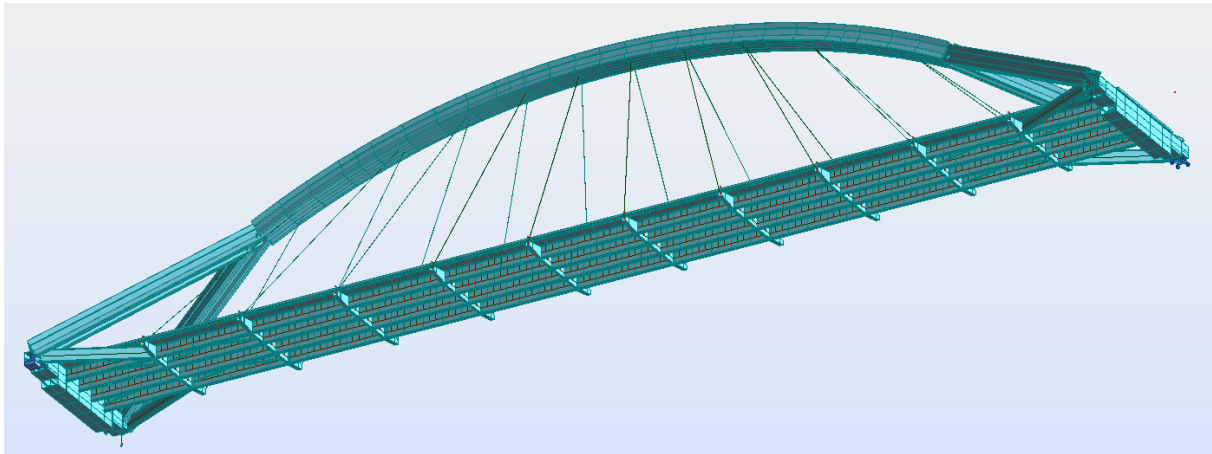
Figure 3. General view of FEM model of the main span of the Antoni Madalinski Bridge.

In Phase I of the bridge assessment presented FEM model was used for calculation of the stress ranges in steel members induced by the FLM3 vehicle of the Eurocode 1 [2] traveling along single traffic lane. The preliminary analysis pointed out critical elements of superstructure potentially exposed to fatigue phenomenon and made a detailed examination of these members necessary.

The identified critical elements (assuming $\gamma_{\mathrm{Mf}} = 1.35$) exposed to fatigue are:

- deck elements in span 3 in a place of connections of stringers with cross-beams – $\mu_{\mathrm{fat}} = 0.58$,
- main girders in span 4 in places where their cross-section changes – $\mu_{\mathrm{fat}} = 0.69$,
- in cable stays in span 3 – $\mu_{\mathrm{fat}} = 0.90$.

As it can be seen especially the deck elements in span 3 show very low fatigue safety level μ_{fat}. Therefore the detailed evaluation in Phase II was focused on estimation of damage accumulation in these elements.

4. Assessment Phase II and Phase III

4.1. Methodology of experimental test

On site testing of the bridge in Ostrołęka provided information on current live loads and rate of damage accumulation in bridge critical spots during the period of increased live loads. The experimental fatigue performance investigation was based on measurements of structure response in selected points to various vehicles. Choice of the points' location was based on numerical analysis in Phase I using the described FEM models of the bridge spans which indicated the members with the lowest fatigue safety level μ_{fat}. Dynamic tests of the structure were executed according to recommendations and experience from previous similar tests, e.g. [5]-[8].
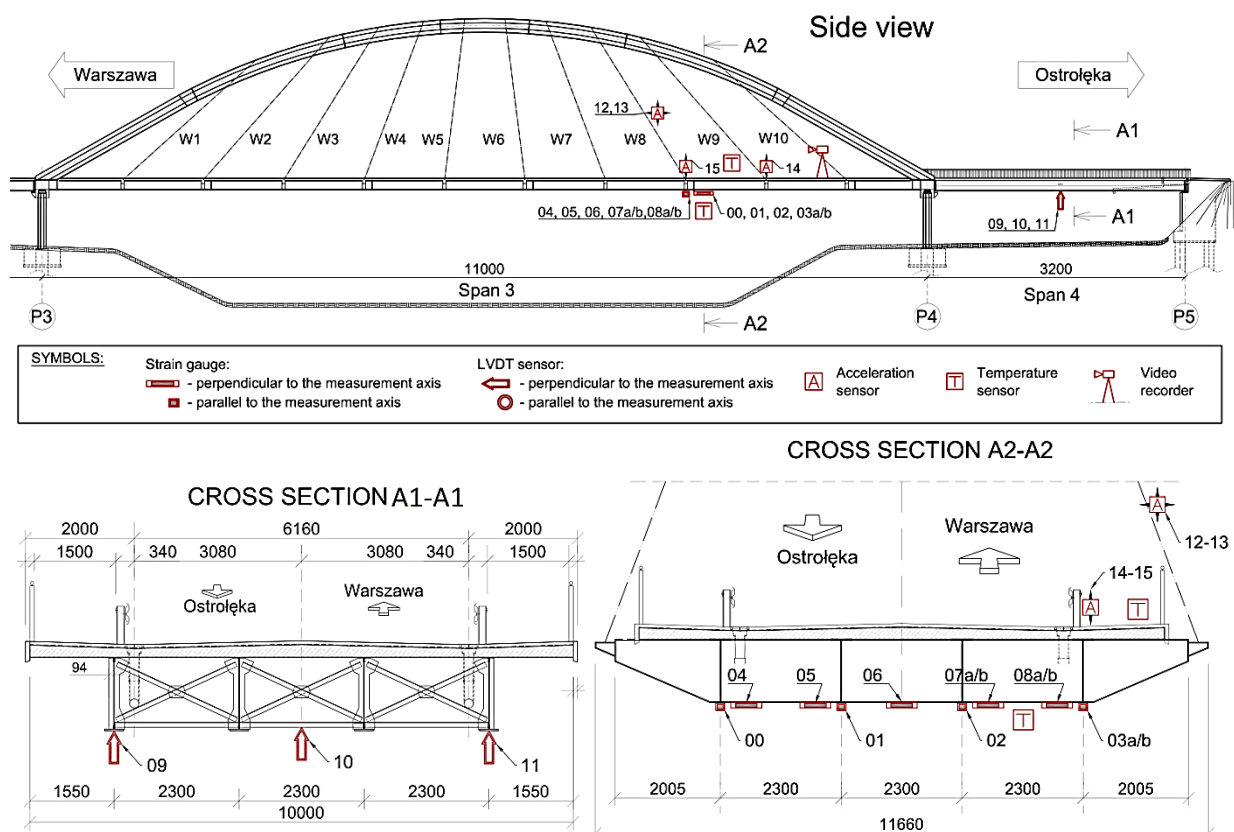
Figure 4. Arrangement of gauges during the tests: a) in side view, b) in cross sections.

The 24 hours special testing of the structure response to live loads included measurements of:
- strains in steel members of the deck within span no. 3, i.e. a stringers (points 00-03) and cross-beam (point 04-08),
- vertical displacements (points 9-11) in middle section of span no 3,
- vertical accelerations (points 12-15) of deck and stay cable in span 3.

During 24-hours measurements all vehicles crossing the bridge were registered using the camera and digital video recorder (DVR), for matching type of vehicles with the corresponding measured physical values. The temperature was controlled continuously in two specific areas of the structure.

Location of the measurement points and types of gauges applied in tests is given in figure 4. The measurements were carried out:

- under controlled, quasi-static and dynamic load scenarios executed by means of utilization of special vehicle of designed and controlled parameters (i.e. axle loads and spacing, vehicle velocity and location),
- during a 24-hours session under increased operation conditions.

The obtained results were used threefold. Firstly, they enabled initial calibration of the numerical model applied in Phase I of the assessment procedure. Secondly, the measured response provided information on real stress ranges within the structural members and in this way gave a direct input to calculation of fatigue damage being accumulated during the period of structure exploitation under increased live loads. Thirdly, it was also possible to identify level and distribution of approximate total vehicles weights corresponding to fatigue progress in critical bridge elements.

4.2. Results of experimental test under controlled load scenarios

Tests under dynamic loads of designed and controlled parameters consisted of a series of passages of a single 40-tonne truck along the bridge at the following speeds: 5, 20, 40, 60 km/h. Parameters of the applied truck (see figure 5a) have been selected in such a way to determine the highest level of effects in the analyzed bridge components induced by the heavy trucks meeting the regulatory requirements for admission to traffic in Poland.
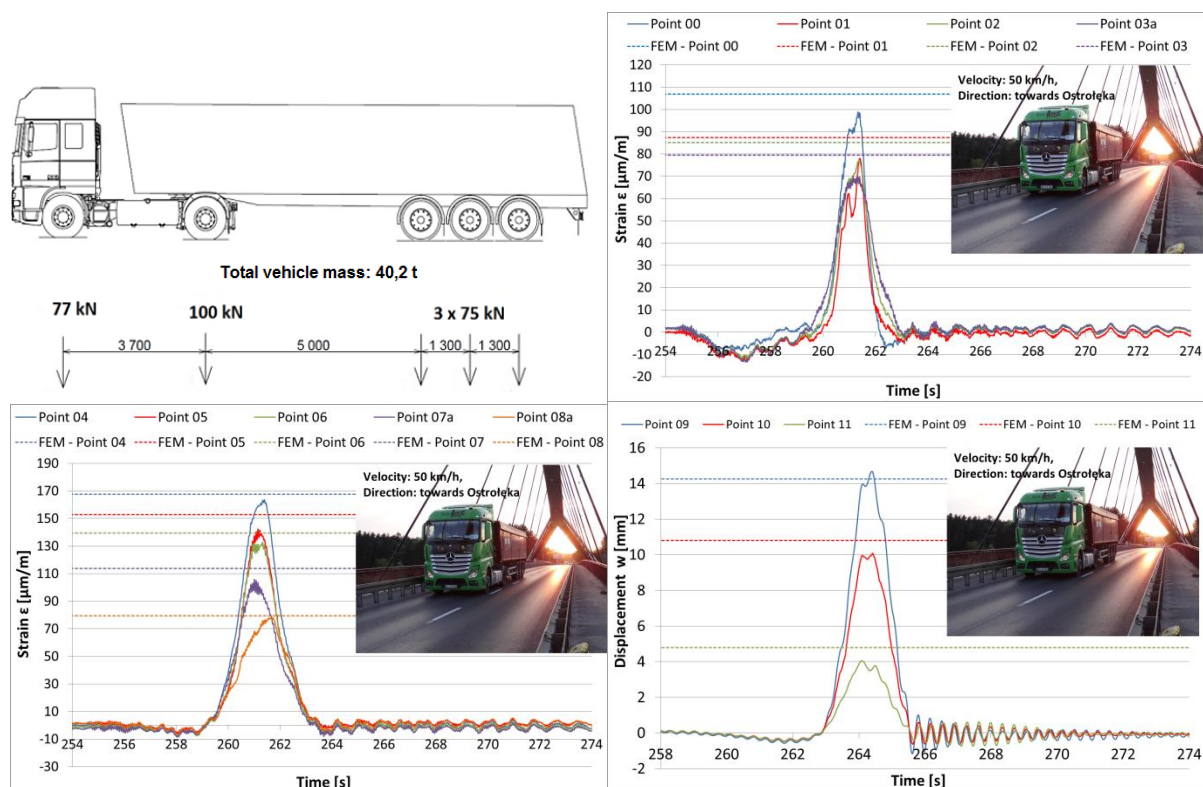
Figure 5. History of strains and displacements induced in selected points by a single 40-tonne, 5-axle vehicle traveling at a speed of 50 km/h towards Ostrołęka: (a) vehicle parameters, b) strains in stringers of span 3, c) strains in cross-beams of span 3, d) deflections of span 4.

Exemplary history of strains and displacements induced in analyzed points of the bridge by the applied vehicle (figure 5a) crossing the bridge at speed of 50 km/h is shown in figure 5b-d. In the diagrams the results of strain and displacement measurements were also compared with extreme static

effects of loading with a moving theoretical vehicle of the same type determined by means of the applied numerical model. As it can be seen the experimentally determined peak values of strains related to real vehicle load proved to be consistent with the results of the theoretical analysis. The theoretical strains based on FEM analyses are very close to the experimentally identified values. The differences between the experimental and theoretical results are relatively small and do not exceed 10% on average. Therefore it can be concluded that the obtained results confirm high precision of applied numerical model as well as a proper modelling approach utilized in Phases I and II of the theoretical fatigue evaluation procedure.

During the model validation process significant (due to possibility of fatigue phenomenon) horizontal bending of steel deck elements of span 3 was revealed. Therefore, for capturing more realistic peak values of stresses in the vicinity of potential cracks at connections of stringers with cross beams an additional strain measurement points (03b, 07b and 08b) were installed before executing of 24-hours session under operational conditions. This was of great importance for determination of more precise strain ranges induced in critical spots by real live loads as the extreme strains recorded at "b" points were significantly greater than recorded at the corresponding "a" ones.

4.3. Results of experimental test under operational loads

Measured quantities caused by the real traffic provided data on extreme real strain/stress ranges occurring in critical components of the bridge superstructure induced by heavy traffic crossing the bridge during 9 months period of excessive use.

Maximum values of strains registered during measurements were relatively high and amounted to $\varepsilon_{max} = 148.41$ μm/m (point 03b) in stringers and $\varepsilon_{max} = 283.74$ μm/m (point 07b) in cross beams (figure 6). It was concluded that recorded peak values of strains, significantly larger than obtained by means of special truck use for model validation, were induced by passage of about 50 tonne truck.

Dynamic testing of the bridge superstructure under real operating conditions aimed also at identification of the contemporary loads which contribute the most to fatigue durability of bridge critical components. It was found that the highest values of measured physical quantities at present time of bridge operation are essentially related to the passing of 4-axle or 5-axle trucks transporting goods with total weight usually varying between 30-40 tonnes. What is more, in many cases estimated total weight of vehicles was varying between 41-50 tonne. Crossings of such vehicles of excessive total mass, in comparison with bridge load capacity, significantly reduce fatigue life.
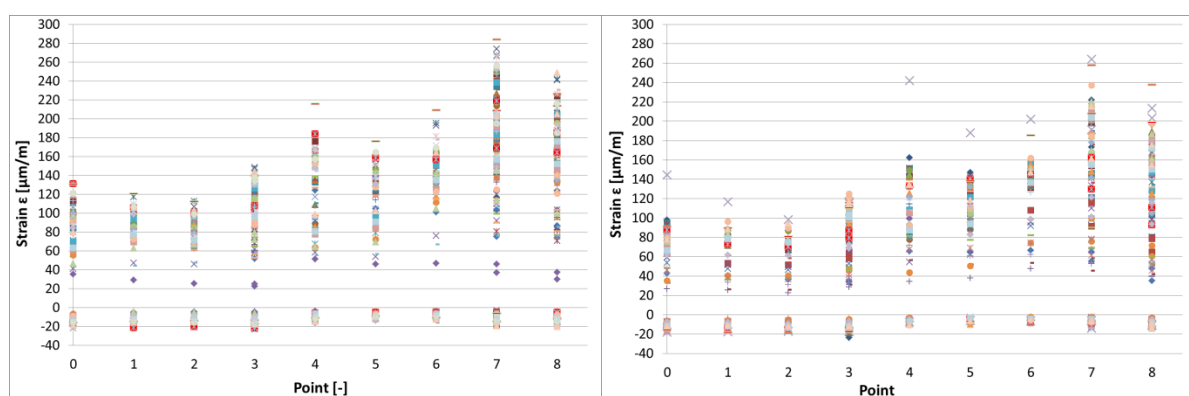

Figure 6. Extreme values of strains induced by live loads during consecutive measurements (10 minutes each): a) in the daytime: 7 a.m.–7 p.m., b) at night: 7 p.m.–7 a.m.

4.4. Fatigue performance assessment

Measurements during a 24-hours session were carried out to get real-time information on the amount of damage accumulated during the period of increased operating loads.

The measured history of strains caused by the actual traffic constituted essential input for the prediction of the bridge remaining fatigue life. The rainflow counting algorithm was used in order to reduce a spectrum of varying strains into a set of simple stress reversals. Histograms presented in figures 7 show number of cycles for 50 selected threshold values of stress ranges occurring in measurement points during 24-hours of bridge operations conditions under increased live loads. Assuming detail category of 50 MPa (defined in [3]) for span 3 welded grillage system components the corresponding fatigue cut-off limit (indicated also in figure 7) below which micro-damage does not accumulate in material is: 15.20 MPa considering safety factor $\gamma_M = 1.35$ and 17.84 MPa assuming safety factor $\gamma_M = 1.15$. Meanwhile the experimentally obtained maximum values of stress ranges caused by the real traffic was equal to $\Delta\sigma_{max}^{EXP} = 61.17$ MPa (point 07b) which confirmed, according to the current knowledge (e.g. [4]), a risk of fatigue failure in the studied structural members.
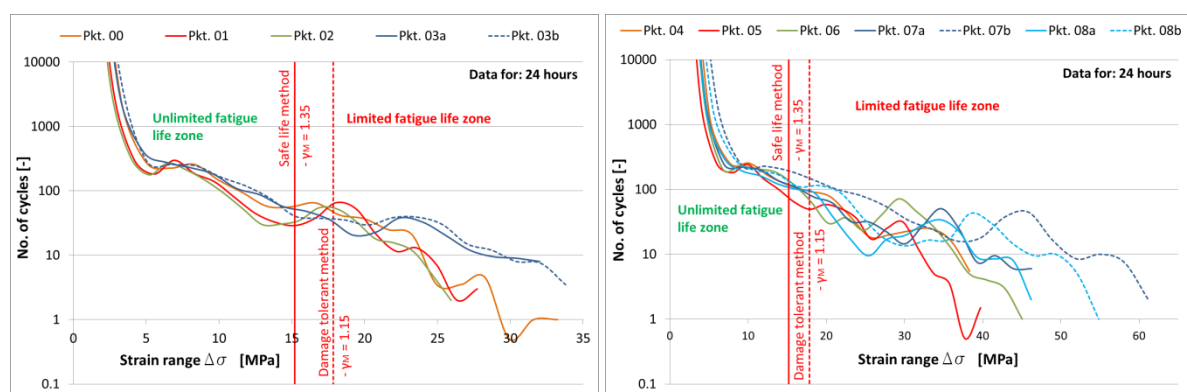
Figure 7. Stress range [MPa] spectrum calculated for 24- hours of bridge operation on basis of experimental tests results: a) points at stringers, b) points at cross beams

In Phase II, based on results of 24-hours measurements, the calculated total fatigue damage D_d^{exc} being accumulated during the 9 months period of excessive use in the critical bridge components was less than 1 for all investigated members. The maximum estimated level of fatigue damage D_d^{exc} of the deck elements of the span 3 is diverse and ranges from 0.012 (stringers, point 03b) to 0.062 (cross beams, point 07b).

In Phase III, considering all stages of bridge exploitation, the maximum estimated total damage accumulated during the elapsed 22 years bridge lifetime in analysed spot of the evaluated critical members was assumed to be around 0.132 (point 07b). This value was assessed assuming D_d^{past} at level 0.069 corresponding to the traffic volume during 22 years bridge lifetime equal to 5% of traffic measured during the tests. This means that the fatigue life of critical components of the bridge has not been completely exhausted so far.

Similar assumptions were made for representing predicted damage accumulation for one year of bridge normal operation at future traffic D_d^{fut}. Finally assuming in formula (5) $D_d^{past} = 0.069$, $D_d^{exc} = 0.062$, $D_d^{fut} = 0.003$ and $\beta = 1.5$ the remaining fatigue life time T_{fat} was assessed to be equal 180 years.

It was concluded that when the limitation related to the total weight of a single vehicle crossing the bridge is respected by bridge users than the fatigue damage does not accumulate in critical bridge members and in that case there is no risk of superstructure failure associated with it.

5. Summary
The case study on fatigue evaluation performance of 22 years old, steel-concrete, through arch bridge in Ostrołęka in Poland was described in this paper. The applied procedure of structure evaluation was performed following the general assumptions of the European design codes [2]-[3] and recommendations [4]-[5].

The basic input data used for bridge assessment were based on 24-hours, experimental dynamic tests carried out using real live loads of designed and controlled parameters as well as under natural (random) operation conditions.

Taking into account the results of the theoretical analysis and experimental load tests the following general conclusions may be formulated:

- Applied numerical modelling and analysis approach of composite, steel-concrete superstructure of the Antoni Madaliński Bridge enabled precise representation of strain/stresses in all structural members taking into consideration both, global and local effects of live loads.

- Fatigue evaluation supported by results of the experimental vibration tests provided information on actual loads acting on the structure and enabled refinement of FEM model of bridge superstructure, thereby allowing a more detailed assessment of the level of fatigue damage of the bridge critical members.

- The estimated, current maximum level of fatigue damage is $D_{d,\ max}(t=0) = 0.132$ and actual total weight of vehicles crossing the bridge is now limited to 3,5 tonnes. This indicates that the remaining life of the critical structural components has not yet been fully exhausted and bridge fatigue performance should be satisfactory throughout its design life.

- Overall maintenance of the considered bridge is satisfactory. However, localised corrosion pits in any members need a special attention because simultaneous acting of corrosion and fatigue will dangerously accelerate degradation mechanism.

The applied method of testing planning and execution as well as measuring system confirmed practicability of the solutions in identification of bridge characteristics as well as in determination of real load spectra acting on a structure. The proposed methodology can be efficient in condition evaluation of other bridge structures.

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