Experimental Vibration Tests in Fatigue Evaluation of a Riveted Truss Bridge

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Abstract. The paper presents a practical implementation of experimentally supported fatigue life prediction of an 80-year old riveted truss bridge. The bridge is an unique road structure crossing the Vistula River in Puławy in Poland. Its main part is composed of 5 large spans with Gerber beam static system. In the previous decades the bridge played an important role in the transportation system of the whole region and despite of its advanced age and historic value it was intensively exploited by heavy vehicles. Thus, taking into account the age of the structure and intensity of the traffic crossing the bridge in the past, some doubts about the remaining fatigue resistance of bridge critical members − defined by the fatigue safety level and damage indicator − were formulated.

Fatigue evaluation of the old riveted truss structure was performed according to general assumptions of the Eurocodes [1, 2] and the *European Recommendations for Estimation of Remaining Fatigue Life of Existing Steel Structures* [3]. 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 real live loads.

Keywords: Riveted road bridge · Experimental vibration test · Fatigue analysis

1 Bridge Structure and Objectives of the Study

The investigated structure called the Ignacy Mościcki Bridge is located over the Vistula River in Puławy in Poland. The bridge is a unique road structure in the form of a continuous beam with hinges – the so-called Gerber system – made of truss girders. The main part of the bridge is composed of 5 spans with the lengths: 85 + 88 + 110 + 88 + 85 meters. The bridge truss girders, bracings as well as deck grillage are riveted and the steel components of the bridge are made of mild steel. Top part of the deck is constructed in the form of an RC slab. Massive supports are made of concrete with stone facade. The structural form including basic dimensions of the investigated bridge is presented in Figs. 1 and 2.

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Fig. 1. The Ignacy Mościcki Bridge over the Vistula River in Puławy

During the World War II the structure was damaged twice: in 1939 and in 1944. After the war it has been rebuilt and opened for traffic in 1949. Since then, until the construction of the bypass of the Puławy city in 2008, the bridge was a part of a national road playing an very important role in the transportation system of the whole region. Despite of its advanced age and historic value it was intensively exploited neglecting compliance with any particular restrictions related to total weight of vehicles crossing the bridge. Since 2008 intensity of the traffic on the bridge has significantly decreased as the admissible weight of vehicles was limited to 12 tonnes.

In the course of special inspection [4] some defects of the bridge superstructure were observed, however no fatigue cracks were directly detected. Anyway, considering the age of the structure and intensity of heavy traffic crossing the bridge in the previous decades, some doubts about the remaining fatigue resistance of bridge critical members appeared. Thus, the main goal of the presented investigation was to estimate the level of the fatigue damage already accumulated in the critical bridge components as well as to predict the remaining life time of the whole structural system.

2 General Procedure of Fatigue Evaluation

The fatigue analysis and prediction of the remaining life of the Ignacy Mościcki Bridge is performed following the general assumptions of the European design codes [1, 2] and recommendations [3]. The assessment is aimed at providing evidence that the bridge will function safely over a specified residual service life. The approach is based on a step-by-step procedure of different precision levels and various input data defining considered loads. In the presented case study the method includes three phases 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 a Finite Element model of the bridge developed on the basis of data coming from the structural drawings supplemented by a site investigation. Within the initial calculations the Fatigue Load Model 3 (FLM3) of the Eurocode 1 [1] was applied and its effects in bridge riveted steel members were referred to the S-N curve corresponding to detail fatigue category of 71 MPa according to [2]. The initial evaluation of the fatigue safety level is carried out according to the formula:

$$\mu_{fat} = \frac{\Delta\sigma_C}{\gamma_{Mf} \cdot \gamma_{Ff} \cdot \Delta\sigma_{E,2}} \geq 1 \tag{1}$$

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

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

$$\gamma_{Ff} \cdot \Delta\sigma_{E,2} = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \cdot \Delta\sigma\left(\gamma_{Ff}, Q_k\right) \tag{2}$$

where: $\Delta\sigma(\gamma_{F,\ f}, Q_k)$ – the maximum stress range generated by fatigue loads in an analysed bridge member; λ_i – damage equivalent factors determined according to [2]; Q_k – characteristic value of a single variable action generated by FLM3 [1].

When the calculated fatigue safety level $\mu_{fat} < 1$ for any of structural members then the member safety needs to be further assessed following subsequent steps of higher precision of the procedure.

Phase II – a detailed investigation – is a refined assessment carried out only for those members whose fatigue safety level determined in the Phase I was not ensured (i.e. when $\mu_{fat} < 1$). This can be done by application of a more complex load model as well as more precise numerical representation of the structure itself. Within step (a) of this phase FLM4 load model can be utilised consisting of 5 different vehicles given in [1]. If the obtained results are still not satisfactory step (b) of the Phase II is introduced by correction of the FLM4 model values of load on the basis of strain measurements carried out in bridge critical members under real traffic loads. In both steps (a) and (b) of the Phase II the already accumulated fatigue damage in the selected structural members is evaluated. It is calculated using the Palmgren-Miner damage summation rule, simply stated as follows:

$$D_d = \sum \frac{n_{Ei}}{N_{Ri}} \leq 1 \tag{3}$$

where: D_d – the total damage accumulated during the elapsed lifetime in an analysed spot of an evaluated riveted member, n_{Ei} – number of cycles occurring at stress range $\gamma_{Ff}\Delta\sigma_i$, for band i in the factored spectrum; N_{Ri} – the endurance (expressed in cycles) obtained from the factored $\Delta\sigma_c/\gamma_{Mf}$ N_R curve corresponding to a stress range of $\gamma_{Ff}\Delta\sigma_{i.}$

In case of the Phase IIb the values of loads defined in FLM4 are modified by factor χ, which is calculated individually for each structural component and is expressed by formula:

$$\chi = \gamma_f \cdot \eta \tag{4}$$

where: γ_f – partial factor for loads; η – ratio of extreme strain range generated by real traffic loads to extreme strain range generated by vehicles of FLM4.

Phase III – a remaining fatigue life estimation – is aimed at estimation of its remaining fatigue life considering present and future operation conditions of a structure. The main input data at this phase are results of stress/strain measurements carried out on site in bridge critical members under real traffic loads covering a longer (minimum one-day) period of the structure typical operation. The measured strain history in consecutive steel components is directly used to create a real stress range spectrum which may be used for representation of present and future operation conditions of a bridge. Additionally some expected external conditions may be included influencing future changes of the traffic on the bridge. Then, considering all periods of the structure operation including past and future traffic conditions leading to damage accumulation within structural members the remaining fatigue life time may be determined.

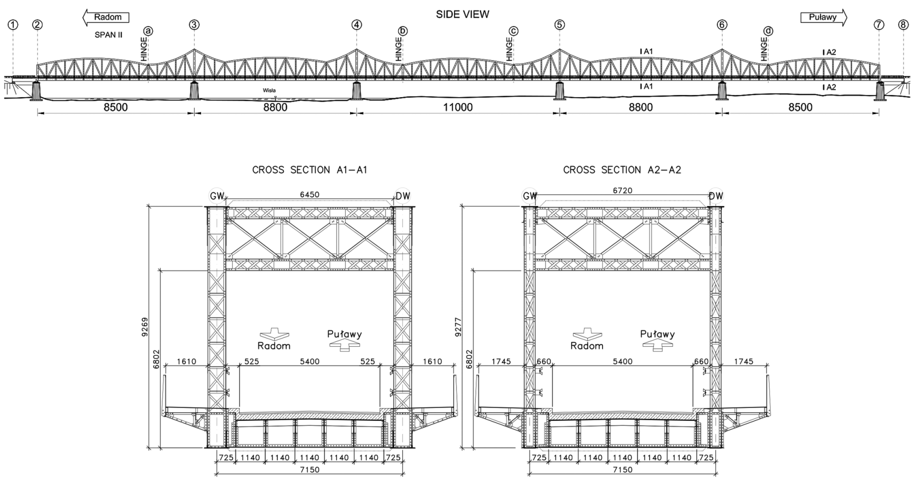

Fig. 2. Structural form of the investigated bridge: (a) side view, (b) cross-sections

For the purpose of theoretical fatigue evaluation of the analysed bridge in Phase I and IIa the truss superstructure was modelled by means of one- and two-dimensional finite elements. One-dimensional beam type elements were utilised for representation of all riveted components of spatial superstructure including bracings. Two-dimensional shell type elements were used for modelling of prefabricated RC deck slab and steel sidewalk decks. The finite element model of the truss spans (Fig. 3) was created and applied within ROBOT Structural Analysis system.

In Phase I the stress ranges in steel members caused by FLM3 load model of Eurocode 1 [1] were analysed. The preliminary theoretical analysis pointed out critical components of spans potentially exposed to fatigue phenomenon and made a detailed examination of those members necessary. The identified elements exposed to fatigue are some of the diagonals of main girders and deck components, especially stringers.

In Phase IIa, stress ranges induced by the theoretical live load model FLM4 defined in code [1] and recommended for fatigue analysis of existing road bridges in Europe [3] were evaluated. To determine the intensity of heavy traffic on the investigated structure in the previous years of its operation, data from the General Measurements of Traffic performed on national roads in Poland since 1975 were used. Estimated the total number of heavy vehicles N_{obs}, which crossed the bridge so far during its almost 80 years of operation is about 25.6 millions.

Calculated in Phase IIa values of fatigue damage indicator D_d occurred to be greater than 1 for most of the selected critical components of structure identified in Phase I. So high values of the damage level D_d indicates that remaining fatigue life of the bridge critical members have already expired and thus indicated the need of experimental confirmation of the bridge remaining fatigue life under real traffic loads.

3 Methodology of Experimental Vibration Tests

On site testing of the bridge in Puławy provided evidence that the structure may function safely over its specified service life. The experimental verifying of the bridge remaining fatigue life was based on measurements of structure response in selected points to various live loads. Choice of the points' location was based on numerical analysis in Phase I using the described FE model of the bridge which indicated the members with insufficient fatigue safety level (where $\mu_{fat} < 1$). Dynamic tests of the structure were executed according to recommendations specified in [5–8] and experience from previous similar tests, e.g. [9–12].

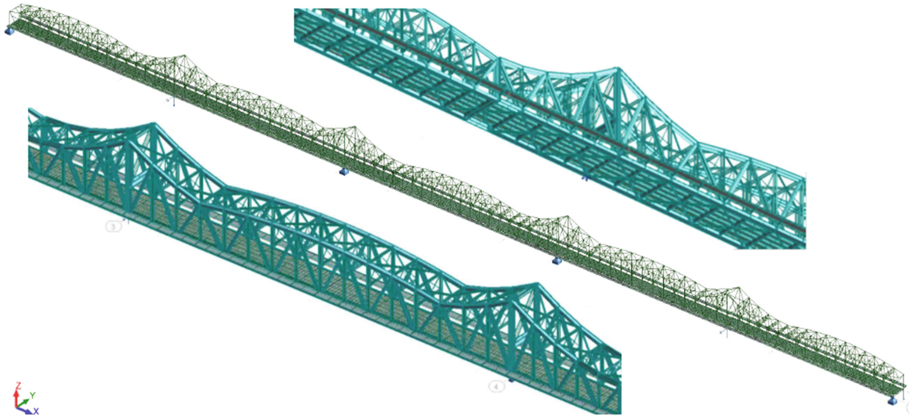

Fig. 3. General view of FE model of the Ignacy Mościcki Bridge

The whole-day special testing of the structure response to live loads included measurements of:

- strains in diagonal members of the main truss girder (points 00-03),
- strains in steel members of the deck within span no. VI, i.e. a cross-beam (point 04-05) and stringers (points 06-09),
- vertical displacements (points 10-12) close to cross-section A3-A3 (see Fig. 4).

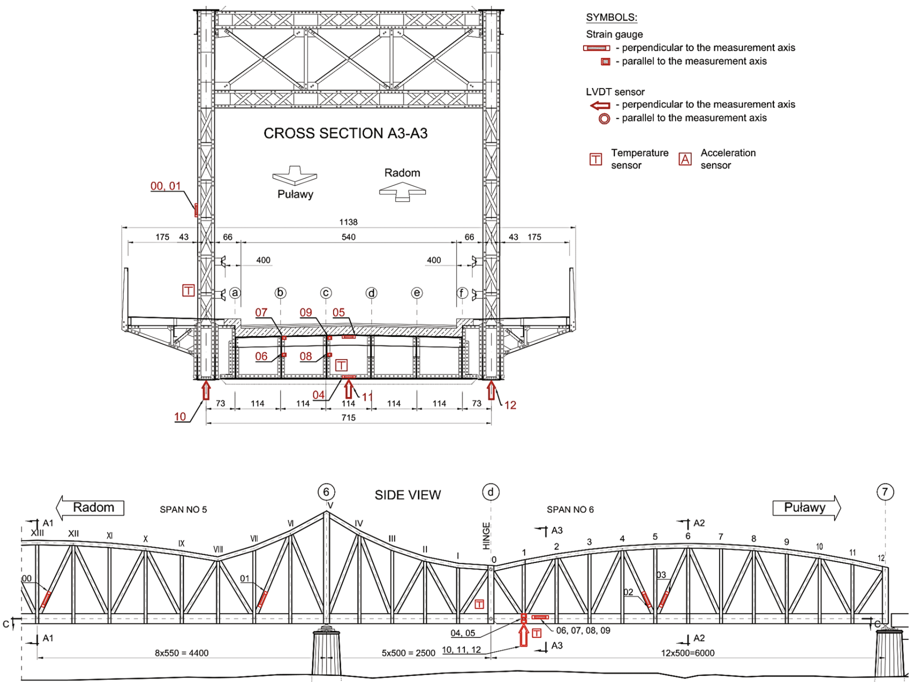

Fig. 4. Arrangement of gauges during the tests: (a) in cross section, (b) in lateral view

Location of the measurement points and types of gauges applied in tests is given in Fig. 4. The measurements were carried out during a one-day session under quasi-static and dynamic live loads including designed and controlled load scenarios as well as under natural operation conditions.

The obtained results were used twofold. Firstly, they enabled initial calibration of the numerical model applied in Phase I of the procedure and its enhancement introduced in Phase II related to improvement of numerical representation of a connection of the deck to the truss girder. Secondly, the measured response provided data on real stress ranges within the structural members and in this way allowed modification of the fatigue load models applied in Phase IIb as well as gave a direct input to remaining fatigue life estimation at real present and future operation conditions.

4 Results of the Vibration Tests

Measurements under live loads were carried out to get real-time information of the superstructure performance subjected to a live load of designed and controlled parameters (i.e. axle loads and spacing, vehicle velocity and location) as well as under natural traffic loads occurring at present time during regular operation of the bridge. Selected results of measurements of physical quantities caused by different types of vehicles in investigated bridge critical members are presented in Figs. 5 and 6.

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 Fig. 5a) have been selected in such a way to get a higher effect in the analysed bridge components than an effect induced by any truck meeting the regulatory requirements for admission to traffic in Poland. Eventually utilised 4-axle vehicle weighing 40 tonnes, due to its actual exceedance of the permissible axle loads, caused in the superstructure slightly larger effects than typical heavy vehicles being in service in Poland which meet regulatory requirements.

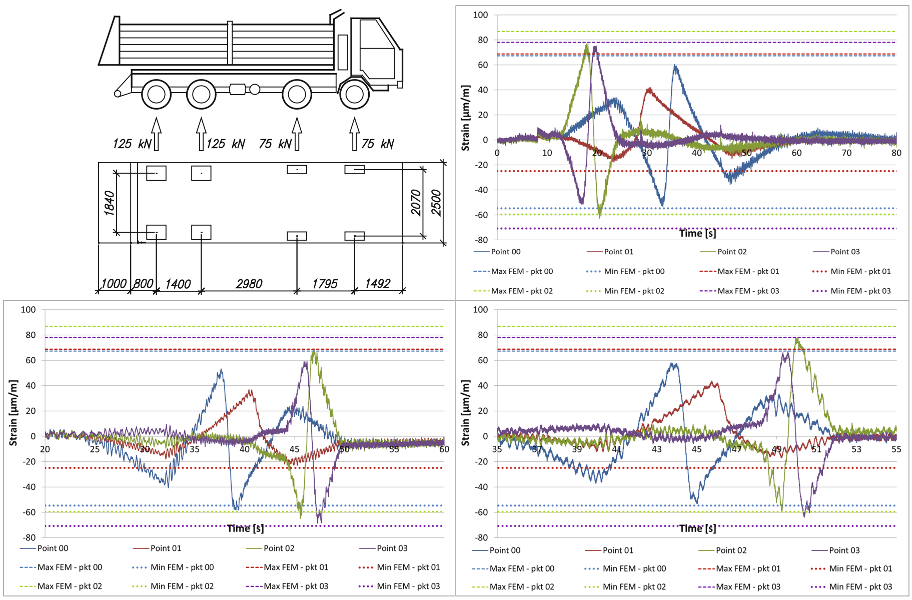

Fig. 5. History of strains induced in the main girders by a single 40-tonne 4-axle vehicle (a) crossing the bridge with various speed: (b) v = 20 km/h, (c) 40 km/h, (d) 60 km/h

Exemplary history of strains induced in diagonal members of the truss main girders by the applied vehicle (Fig. 5a) crossing the bridge with speed varying between 20–60 km/h is shown in Fig. 5b–d. In the diagrams the results of strain 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 FE analyses are very close to the experimentally identified values – maximum of measured strains was equal 78,26 µm/m (point 02) and corresponding calculated one amounts to 86,83 µm/m. The differences between the experimental and theoretical results are relatively small and do not exceed 5% 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 utilised in Phases I and II of the theoretical fatigue evaluation procedure. Besides the measurements showed low dependence of extreme internal effects occurring in the main truss members to the speed of a vehicle, while indicated larger fragility to dynamic excitation of short members of the deck. Thus phenomenon is predicted also in some design codes in the form of the dynamic factor being reduced to 1.0 for longer members.

Measured strains caused by the single 40-tonne truck provided data on extreme real stress ranges occurring in critical components of the bridge superstructure induced by heavy traffic crossing the bridge in the previous decades, until 2008. The obtained maximum values of strain ranges caused by the utilised single vehicle was equal to $\Delta\varepsilon_{max}$ = 139.68 µm/m (point 02) which confirmed a risk of fatigue failure in the studied structural members.

Dynamic testing of the bridge truss superstructure under regular operating conditions aimed also at identification of the contemporary loads which may affect the fatigue durability of bridge critical components. The measured history of strains caused by the actual traffic constituted essential input for the prediction of the bridge remaining fatigue life.

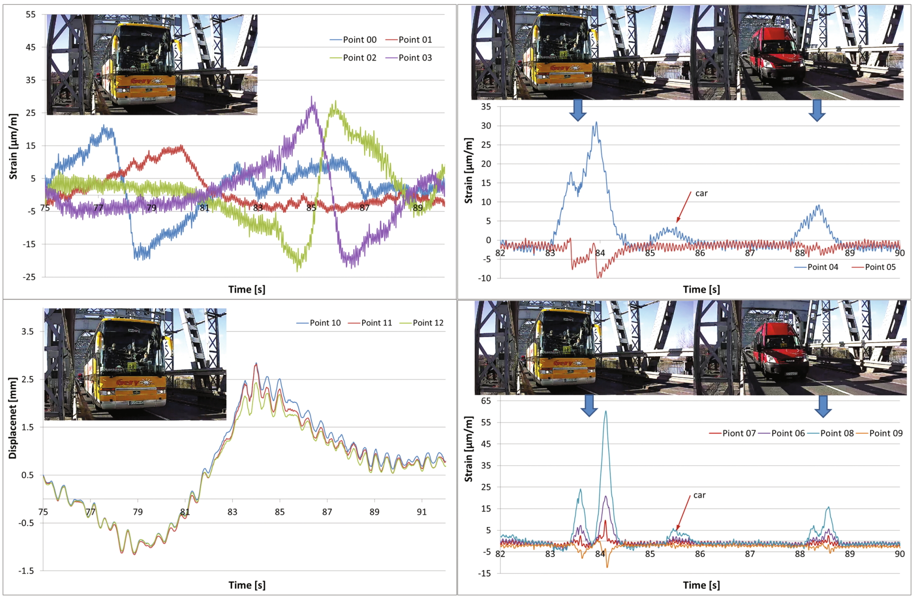

Fig. 6. History of physical quantities induced by selected actual traffic loads during regular operation of the bridge in analysed bridge members: (a) strains in main girders, (b) strains in cross-beam, (c) strains in stringers, (d) displacements of span

It was found that the highest values of measured physical quantities at present time of bridge operation are essentially related to the passing of 2-axle buses with total weight usually varying between 14–16 tonnes. Exemplary history of physical quantities induced by such vehicles are presented in Fig. 6. However it should be underlined that during preparation to the load test, in particular on the day preceding the measurements, a few 5-axle trucks of permissible weight of 40 tonnes were also identified.

The presented data in Fig. 6 show that at the current operating conditions (i.e. since 2008 when bypass of city was build) the highest strains occurring in bridge critical members caused by the actual traffic are relatively small. Maximum value of measured strains in diagonals of the truss girders and in deck components are equal 37,53 μm/m (7,69 MPa) and 60,35 μm/m (12,37 MPa), respectively.

5 Fatigue Damage Accumulation Assessment and Remaining Fatigue Life Estimation

Experimental tests performed using a single 40-tonne vehicle revealed that maximum values of strain ranges caused by such vehicles, which intensively exploited the bridge over the previous decades until 2008, in the light of current knowledge (e.g. according to [2, 3]) can affect fatigue life of some diagonals of truss girders and stringers of the deck system. Assuming detail category of 71 MPa (defined in [2] and suggested in [3]) for bridge riveted components the corresponding fatigue cut-off limit below which micro-damage does not accumulate in material is 103.8 μm/m (21.28 MPa) considering safety factor $\gamma_M = 1.35$. Meanwhile the maximum strain range taken from the measured strain history (see Fig. 7a) for selected elements equals $\Delta\varepsilon_{max} = 139.68$ m/m (28.63 MPa).

It was also concluded that effects caused by purely theoretical load model FLM4 [1] substantially differ from those induced by the real live loads crossing the bridge, both at present as well as also during intensive operation of bridge in the past. In particular, axle loads and total weights of FLM4 trucks, which were adopted in [1] with a considerable safety margin, do not meet the regulatory requirements relating to the technical conditions of vehicles operating in Poland. Therefore extreme stress ranges generated by fatigue load model FLM4 was modified by factor χ calculated individually for each structural component according to formula (4). Obtained values of factor χ, are varying between 0.63-0.75 for different members.

In Phase IIb, based on modified FLM4 axle loads, the calculated fatigue damage D_d was less than 1 for all investigated members. The estimated level of fatigue damage D_d for the considered critical members of the riveted bridge superstructure is diverse and ranges from 0.13 to 0.90. The greatest fatigue damage (D_d = 0,87–0,9) have already accumulated in diagonals of the suspended span (points 02-03), while the relatively low level of damage occurred in deck members. This means that the fatigue life of critical components of the bridge has not been completely exhausted so far.

Maximum strain ranges registered in analysed bridge members induced by current regular traffic loads are presented in Fig. 7b.The maximum value of the calculated strain range $\Delta\varepsilon$ from among all performed measurements is 66,10 μm/m. Obtained value of $\Delta\varepsilon$ is significantly less than the fatigue threshold value accounting for

103,8 µm/m ($\gamma_{Ff}\Delta\sigma_L$ = 21.28 MPa) below which the occurring strain ranges potentially do not contribute to fatigue durability of bridge riveted members.

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.

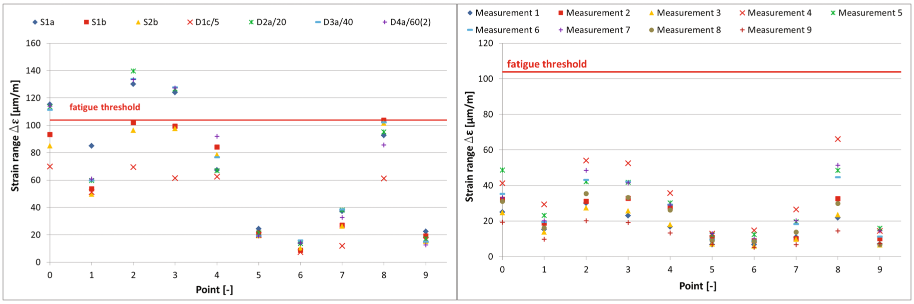

Fig. 7. Maximum strain ranges [µm/m] registered in analysed bridge members by means of experimental tests, caused by: (a) the single 40-tonne vehicle representing the loading acting on the structure until 2008, (b) current regular traffic loads

6 Summary

Fatigue evaluation procedure of the old riveted bridge in Puławy in Poland was described in this paper. The applied procedure of structure evaluation was performed following the general assumptions of the European design codes [1, 2] and recommendations [3]. The collecting of basic input data used for bridge assessment was based on experimental vibration 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 riveted superstructure of the Ignacy Mościcki Bridge enabled precise representation of strain/stresses in all structural members taking into consideration both, global and local effects of loads.
– Fatigue evaluation supported by results of the experimental vibration tests provided information on actual loads acting on the structure and enabled refinement of theoretical load models of the European design codes [1] suggested also in [3], allowing a more detailed assessment of the level of fatigue damage of the bridge critical members.

– The estimated maximum level of fatigue damage is quite high ($D_{d,\ \max} = 0.9$) but indicates that the remaining life of the critical structural components has not yet been fully exhausted.
– Measured strain ranges induced by actual traffic, with the total weight of vehicles limited to 12 tonnes, are relatively low. This allows to conclude that according to the present knowledge in the analysed members fatigue damage does not accumulate and there is no risk of superstructure failure associated with it.
– Determined low dependence of extreme internal effects occurring in the main truss members to the speed of a vehicle (contrary to the effects occurring in short members of the deck) confirmed such a phenomenon expressed by the dynamic factor being reduced to 1.0 for longer members given by some design codes.
– 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. All that is of great importance in condition evaluation of existing old structures.

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