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Fatigue assessment of existing riveted truss bridges: case study

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Abstract. Many steel riveted bridges have been built in Poland since 1950 and they have not reached their design working lives yet. Nevertheless, a number of fatigue damages are found, especially with structural joints. Moreover, in the past 25 years the traffic on the Polish road network has increased significantly leading to the increasing number of heavy vehicles in the traffic flows. This may affect the safety, serviceability and durability of existing bridges. The road administration is therefore interested in reliable and agreed methods to assess the safety and durability of existing bridges. The special procedure has been prepared to provide technical insight on the way in which existing steel structures could be assessed and the remaining fatigue life could be estimated. This approach follows the principles and application rules in the Eurocodes and provides a scheme with various levels of analysis: a basic level with general methods and further levels, more complex and sophisticated and requiring specific experience and knowledge. This procedure has been used for the fatigue assessment of the 60-year-old riveted truss bridge. The applied procedure as well as the main results of the assessment have been presented in the paper.

Key words: durability, fatigue, steel structure, bridge, riveted truss, Eurocodes.

1. Introduction

The European road network experienced rapid development twice, after the First as well as the Second World War. A large number of riveted steel bridges were built at that time and these bridges will soon celebrate their 100th or 50th birthdays respectively. For the first group this is usually regarded their design working life which had not been estimated in the calculation when the bridges were built. The second group of steel riveted bridges have been built since 1950 and they have reached their design working lives yet. Nevertheless, a number of fatigue damages, which cannot be ignored and neglected, are found, especially in structural connections, due to the inexperience and the lack of knowledge about fatigue strength in those days. Another reason is the high increase of traffic volume on these bridges. Many of these existing bridges have undergone repair or strengthening due to the changes in service requirements. However, very often there are no visible indications of fatigue.

Considering only steel structures, Oehme [1] made a study considering the cause of damage and the type of structures. Bridges (both railway and roadway structures) are among the steel structures most often damaged. In Table 1 existing steel bridges are divided into groups according to the causes which lead to certain damages. This table shows that fatigue ranks first in the frequency of causes of all recorded damages. Approximately 98% damages occurred in the period of 1955 to 1984 which means that most of these steel bridges were riveted structures. Further detailed case studies of failures in steel bridges were carried out by Fisher [2]. He also confirmed that fatigue is the most frequent cause of damages in steel riveted bridges.

Table 1
Main damage causes of existing steel bridges Ref. 2

Damage cause (multiple denomination possible)	Bridges	
	No.	%
Fatigue	49	38.3
Environment	41	32.0
Static strength	19	14.8
Stability (local or global)	11	8.6
Brittle fracture	5	3.9
Rigid body movement	2	1.6
Elastic deformation	1	0.8
Thermal loads	0	0
Others	0	0
Sum	128	100

In the past 25 years traffic volume on Polish road network has increased significantly leading to the increasing number of heavy vehicles in the traffic flows and greater exploitation of their carrying capacities. This may affect the safety, serviceability and durability of existing bridges. The road administration is therefore interested in reliable and agreed methods to assess the safety and durability of existing bridges and to make appropriate provisions for possible restriction of traffic, rehabilitation or replacement of existing bridges by new ones where necessary. The problem of existing bridges and their assessment has recently become more serious. The current low funding for maintenance forces the Polish road administration to postpone investments in existing road bridges and consequently stretch the service life of their old structures. Therefore, the owner of the infrastructure nowadays faces two main challenges: the need for a further continuing safe operation of the ageing bridges and the cost-effective maintenance. The

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methods must be provided that will enable engineers to offer safe and cost-effective assessment and maintenance methods to their clients.

In case steel bridges including the old riveted structures there are numerous foreign [3–7] as well as domestic approaches [8–11] to structural assessment including remaining fatigue life. Moreover, the significant UE founds have been spent recently for R&D projects dealing with this subject, f.e.: SAMCO – *Structural Assessment, Monitoring and Control* (www.samco.org), *Sustainable Bridges* (www.sustainablebridges.net), or *Long Life Bridges* (www.longlifebridges.com). Nevertheless, there are no standardised national codes or recommendations so far in this field. In the light of the development of the European single market for construction works and engineering services there is thus a need to harmonise procedures proposed so far with agreed European technical recommendations for the safety and durability assessment of existing structures. The first attempt has been made by *Joint Research Center of European Commission* (JRC) along with *European Convention for Constructional Steelwork* (ECCS). The recommendations [12] have been prepared to provide technical insight on the way existing steel structures could be assessed and the remaining fatigue life could be estimated. This procedure has been used for the fatigue assessment of an old riveted truss bridge, built in the early fifties of the 20th century and being a typical example of existing long span bridges over the Vistula and the Odra, the two biggest Polish rivers. The applied procedure as well as the main results of the assessment have been presented in the paper.

2. Fatigue assessment procedure

The recommended process of any structural assessment can be divided into the four phases: preliminary evaluation, detailed investigation, expert investigation and remedial measures [12]. With each phase the effort in time and money is increased as well as the knowledge necessary to carry out the assessment. The aim of preliminary evaluation (phase I) is to remove existing doubts about the structure safety using fairly simple methods and identify critical parts or members of the structure. This is performed by gathering information on the structure from drawings and design computations, carrying out a site inspection, etc. Assessment is performed by using current codes and by making conservative assumptions where information is doubtful or none. Detailed investigation (phase II) is to update the information and to carry out refined assessment only for those members where safety is not provided. This is done by doing quantitative inspections (e.g. by means of easy to use, low-tech NDT methods) and the use of updated values for loads, resistance as well as more accurate models (static system, structural behaviour). In case of more serious problems concerning risks or costs related to the conclusions and proposals reached in phase II expert investigation (phase III) should be performed to verify a decision carefully. Further assessments using specific tools (high-tech NDT methods, fracture mechanics, probabilistic methods, etc.) can also be carried out to help make a decision.

Finally, remedial steps (phase IV) should be proposed to have a structure fit for service with sufficient safety. Various measures can be taken, among which there are: structural health monitoring (SHM), reduction of loads or change in use, strengthening, repair or rehabilitation and, in extreme cases, closing a bridge for traffic and replacement. The choice of the measures to be taken is a function of the structure assessed but in any case the proof of adequacy of the measures to provide safety must be shown [12].

3. Description of the existing bridge

The four-phase approach briefly described above has been applied for fatigue assessment of the riveted truss bridge built in 1953 as one of the several Vistula River bridges rebuilt after the Second World War. Its structural form – continuous steel Warren truss with upper deck – was very popular and typical for crossing big rivers in Poland in those days (Fig. 1). To the present more than 30 of such bridges (road and rail) have still been in service. In case of the discussed bridge the span lengths are as follows: 84.8 + 95.4 + 95.4 + 95.4 + 84.8 = 455.8 m. Two equal riveted truss girders have the

Fig. 1. The existing riveted truss bridge and its deck system

depth from 3.10 m (span) to 5.70 m (support). The girders are braced with the riveted cross-beams on which the deck grid is supported, made up of the longitudinal stringers and the secondary cross-beams. The rolled stringers are locally stiffened with welded ribs and there are the only welded joints in the structural system. The grid is covered with the 10-mm stay-in-place steel pan form connected to the grid with rivets and filled with asphalt concrete. The conventional two-course surface is placed on the deck. The total width of the deck is 6.45 m. The concrete bridge supports are founded on caissons (pillars) and driven wooden piles (abutments).

In 2013, after 60 years of service, the motivator appeared to assess the structure since the planned bridge replacement (due to the current traffic load requirements and bad state of repair) had been postponed and consequently the service life of an old structure was stretched for the next 10 years. Therefore an assessment was carried out to determine the remaining service life of the bridge and to establish further continuing safe operation conditions and the cost-effective remedial measures. One of the most important part of the bridge evaluation was the fatigue assessment of the 60-year-old steel riveted structure.

Site inspection is one of the main goals of preliminary evaluation (phase I). The most commonly used inspection method to detect bridge deterioration, even fatigue cracks, is the relatively elementary visual inspection (VT). The comprehensive visual inspection with the occasional use of dye penetrants (PT) was carried out on the steel structure. The aim of the inspection was to identify critical parts or members of the structure with regard to its state of repair. The inspection revealed a lot of corrosion damages due to insufficient maintenance during service life up to now. The most severe corrosion losses were discovered in the riveted structural joints of the upper chord of both truss girders as well as in the vicinity of two end expansion joints, partially damaged and leaky due to the lack of maintenance. Also the deck elements, such as the stringers and the steel pan forms, suffered from corrosion, especially in the vicinity of the surface drainage outlets. Anticorrosion protection layers on the steelwork were heavily deteriorated, moreover, the deck equipment did not protect truss girders and the deck grid from water and de-icing salt. However, no fatigue cracks were found in the steel structure during visual inspection.

4. Preliminary evaluation (phase I)

Preliminary evaluation usually involves finding information about the bridge (drawings, design computations, former inspection protocols, etc.), the use of the bridge management system (BMS) and a site visit with visual observation and qualitative inspection, the appraisal of the bridge. It was necessary to carry out an intensive study of the available documents, i.e. drawings and calculations. There were not any fatigue calculations done when the bridge was designed (1950). When performing the visual inspection, the differences in actual bridge structure were checked against the remaining drawings and any modifications and changes in the static system.

In order to identify critical members (fatigue critical construction details), apart from the visual inspection of the structure, the calculations were conducted as it is done when a new structure is designed using Eurocodes and conservative assumptions were made when the information was doubtful or none. The relevant Eurocode [13] is based on the classification method which employs S-N curves in conjunction with detail category tables. For the fatigue limit state, the safety level can be expressed by:

$$\mu_{fat} = \frac{\Delta\sigma_C}{\gamma_{Mf}\,\gamma_{Ff}\,\Delta\sigma_{E,2}} \geq 1.0, \tag{1}$$

where μ_{fat} – fatigue safety level; $\Delta\sigma_C$ – fatigue resistance at $N_C = 2 \cdot 10^6$ cycles (detail category); $\Delta\sigma_{E,2}$ – equivalent constant amplitude stress range at $2 \cdot 10^6$ cycles; γ_{Mf} – partial safety factor for fatigue strength $\Delta\sigma_C$; γ_{Ff} – partial safety factor for equivalent constant amplitude stress range $\Delta\sigma_{E,2}$.

Rules for the determination of γ_{Ff}, γ_{Mf}, $\Delta\sigma_E$, and $\Delta\sigma_C$ are given in Eurocode [13]. Equivalent constant amplitude stress range at $2 \cdot 10^6$ cycles $\Delta\sigma_{E,2}$ can be expressed by:

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

where Q_k – characteristic value of loads according to Eurocode [14]; $\Delta\sigma$ (γ_{Ff}, Q_k) – stress range for characteristic values of loads; λ_i – equivalent damage coefficients according to Eurocode [15].

The set of four damage coefficients ($\lambda_1 - \lambda_4$) takes into account the damaging effect of traffic depending on the length of the influence line or surface, the expected annual traffic volume, the design working life of the bridge and multi-lane effects respectively. For the assessment of the expected annual traffic volume (coefficient λ_2), indicative numbers of heavy vehicles expected per year and per slow lane are given in Eurocode [14].

At this stage of assessment, Q_k means a characteristic value of loads according to Fatigue Load Model No. 3 defined in Eurocode [14], which is intended for common verifications, without performing any damage calculation. It consists of four axles of 120 kN, each axle having two wheels with square contact areas of 0.40×0.40 m^2 (Fig. 2). Thus, the designer calculates the extreme stresses (maximum and minimum) resulting from the crossing of the bridge by FLM3 in order to evaluate a stress range:

$$\Delta\sigma_{FLM3} = \left|\Delta\sigma_{FLM3,\max} - \Delta\sigma_{FLM3,\min}\right|. \tag{3}$$

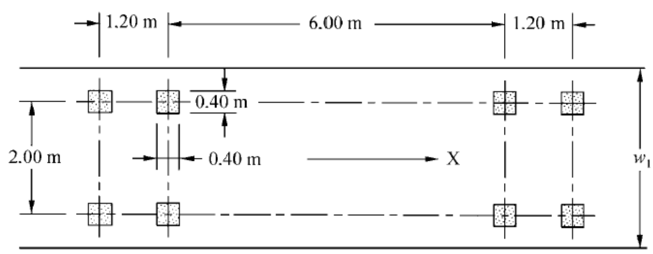

Fig. 2. Definition of FLM3 according to [16] (w_1: lane width, X: bridge longitudinal axis)

For $\mu_{fat} \geq 1$, the investigated element fulfils the fatigue safety requirements. For $\mu_{fat} < 1$, the fatigue safety needs to be further assessed in phase II. Analyzing the elements of the structure in this way we could derive a list of priorities for subsequent, more thorough investigations.

For preliminary evaluation of the bridge with the formula (1) the following values were applied: $\Delta\sigma_C = 71$ MPa – for riveted joints according to [16]; $\Delta\sigma_C = 80$ MPa – for welded joints in rolled stringers according to Eurocode [13]; $\gamma_{Ff} = 1.0$ – fatigue safety coefficient according to Eurocode [15]; $\gamma_{Mf} = 1.35$ – partial factor for high consequences of failure according to Eurocode [13].

Equivalent damage coefficients λ_2 and λ_3 for the bridge were determined with the following assumptions of the road administration: annual traffic volume according to general traffic record done in 2010 and the 10 year service life of the bridge. Using simple 2-D structural models of a truss girder and deck grid, the inner forces and corresponding maximum and minimum stresses in all members were calculated and followed by establishing stress ranges $\Delta\sigma_{FML3}$ applied for fatigue evaluation. This way were determined 11 members of each truss girder (8 diagonals and 3 bottom chord sections) with the fatigue safety level $\mu_{fat} < 1.0$. The most critical member of the deck was the stringer in sections with welded ribs, located in 1/3 and 2/3 of its span length. Table 2 shows the exemplary fatigue safety calculations for selected girder members. Once the critical construction details were known, the calculation of the remaining fatigue life could be made (phase II).

Table 2
The exemplary fatigue safety calculation for truss members in phase I of assessment

Truss girder zone	Middle	Support	Middle
Member number	105 (diagonal)	119 (diagonal)	125 (diagonal)
A_{netto} [cm^2]	102.4	161.1	71.3
$N_{\min}$ [kN]	604.8	2410.3	1.2
$N_{\max}$ [kN]	1389.0	3242.6	850.9
$\sigma_{p,\min}$ [MPa]	59.0	149.6	0.2
$\sigma_{p,\max}$ [MPa]	135.6	201.3	119.4
$\Delta\sigma_p$ [MPa]	76.6	51.7	119.2
λ	0.995	1.270	0.936
Φ_2	1.0	1.0	1.0
$\Delta\sigma_{E,2}$	76.2	65.6	111.6
$\Delta\sigma_c$	71.0	71.0	71.0
γ_{Ff}	1.0	1.0	1.0
γ_{Mf}	1.35	1.35	1.35
μ_{fat}	$0.69 < 1.0$	$0.80 < 1.0$	$0.47 < 1.0$

5. Detailed investigation (phase II)

The aim of the detailed investigation was to update the information obtained in phase I by carrying out refined assessments only for those elements for which adequate safety was not confirmed by the preliminary evaluation. Phase II usually comprises more accurate calculation with the use of updated values for loads, resistance, as well as more accurate models (static system, structural behaviour) and quantitative inspection using easy to use, low-tech NDT methods. Both calculation and inspection were applied in case of the discussed bridge.

In phase II a detailed investigation needs the use of updated values in different areas related to loads and actions (dead loads, permanent loads and variable actions), action effects and response of a structure to actions (e.g. secondary moments, dynamic behavior). All parameters have to be identified in case the obtained information has the largest influence on the assessment results. In addition, the variable actions (mainly traffic loads and its density) change during the life of the bridge and, in order to compute the remaining fatigue life, past, present and future traffic on the bridge has to be evaluated. Therefore it is crucial to get an accurate estimation of the load and load effect distributions on bridges for fatigue issues.

In order to determine correctly the action effects resulting from the actions, a proper modelling must be made. For fatigue assessment, the modelling of the structure must be improved by modelling more precisely the primary and secondary load carrying system. In cases of a special structural behaviour, a model in 3 D is advisable. The modelling of the joints is also a large source of difference between expected and real structural response of a structure. In a riveted truss the level of partial end fixity has a large influence on stresses in connections as well as in members attached to them. This is of particular importance when performing fatigue assessment.

The 3 D model of the whole superstructure was prepared for detailed calculation in phase II (Fig. 3). Beam and shell elements in FEM *Sofistik*-code environment were used for modeling. Using data from the site inspection (phase I) the actual cross-sections of members were assessed with regard to severe corrosion losses. Also due to corrosion of structural connections, full end fixity of riveted members was assumed for the truss girders and for the deck grid. Since the certain deck elements were most critical according to phase I investigations, the detailed deck grid model was built to include the secondary load carrying elements in the model (Fig. 4). The full composite action of stay-in-place steel pan forms with asphalt concrete was assumed using two-layers shell elements for deck modeling.

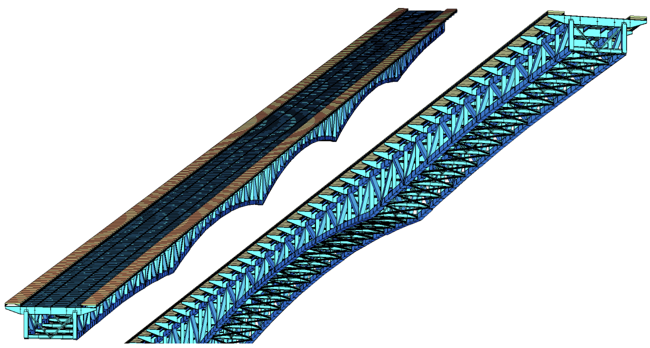

Fig. 3. The updated FEM model of the bridge superstructure

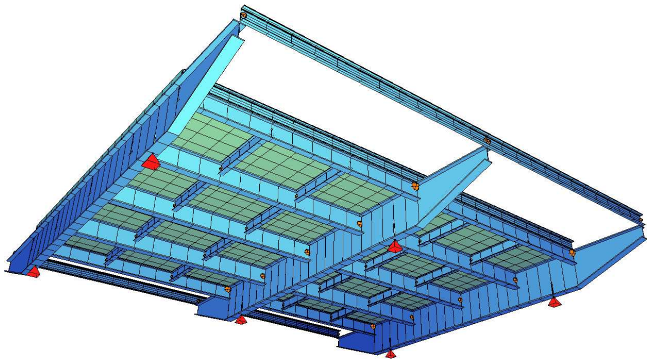

Fig. 4. The detailed FEM model for the deck assessment

For phase II updated values in areas related to loads and actions should take into account dead loads, permanent loads and variable actions. To update the values of dead loads and permanent actions used in the verification, updating the geometry values and the partial safety factors values can be performed. In case of steel bridges updating is not required as long as the members found in the structure are not different from those assumed at the design stage. After the design notes and the existing plans were studied and compared to the structure effectively built, more realistic values for dead loads were used in assessment. The measurement campaign was carried out to gain information on real geometry value affecting dead and permanent loads such as the depth of asphalt concrete deck or surfacing. The partial safety factor values used in the assessment calculations could be reduced by using the information from the measurements.

To update the values of variable actions in phase II a more refined load model is necessary. This load model shall be composed of the different types of trucks crossing the bridge. One example of such a load model for road bridges is given in Eurocode [14] – Fatigue Load Model No. 4 (FML 4). The set of 5 equivalent lorries for FLM 4 is described by the vehicle type (wheel type, axle spacing, equivalent axle loads) and traffic type (lorries percentage, long or medium distance traffic, local traffic). No other load models exist in Polish national codes or specific regulations. The traffic to be used with this load model is the function of the traffic type and volume and can be given in a code or by the road administration. In Eurocode [14] three types of traffic are defined, namely long or medium distance or local traffic, giving the percentage of each lorry. These are used with traffic volume and give the total number of lorries per year.

The FML 4 model is intended to be used for accurate verifications based on damage calculations and is applied for stress spectrum ($\Delta\sigma_i$) evaluation due to vehicles (lorries) crossing the bridge. To apply FML 4 model the number N_{obs} of heavy vehicles (maximum gross vehicle weight more than 100 kN) observed or estimated per year and per slow lane of the bridge is required. The exemplary indicative numbers of heavy vehicles expected per year and per slow lane N_{obs} are given in Eurocode [16] according to traffic category. It should be kept in mind, however, that measurements at a given point in time (N_{obs}) have to be extrapolated into the past as well as into the future.

To refine the FML 4 model, the recorded traffic was used to estimate number N_{obs}, i.e. the total number of heavy vehicles which have crossed the bridge since 1953 (the year of bridge completion). This number was estimated considering the following assumptions:

- the number of heavy vehicles which crossed the bridge in 2000, 2005 and 2010 was recorded by the road administration;
- the number of heavy vehicles which crossed the bridge in 1980–2000 was calculated using the traffic increase rates published in [11];
- the number of heavy vehicles which crossed the bridge in 1970–1980 was calculated on the basis of the trend line established for the previously recorded data (Fig. 5);
- the number of heavy vehicles which crossed the bridge before 1970 was assumed as for 1970 (conservative approach).

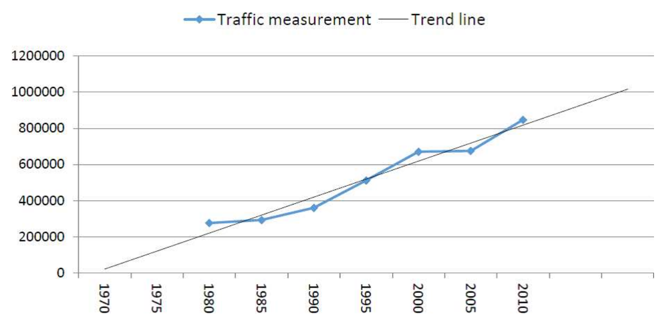

Fig. 5. The heavy vehicle traffic increase for the bridge between years 1970 and 2010

The indicative number of heavy vehicles which have crossed the bridge since 1953 is $N_{obs} = 17\,553\,283$. Using estimated N_{obs} and assuming medium distance traffic and lorries percentage according to Eurocode [14], the fatigue assessment has been performed.

The phase II calculation normally takes the form of damage accumulation calculation (Fig. 6). The most commonly

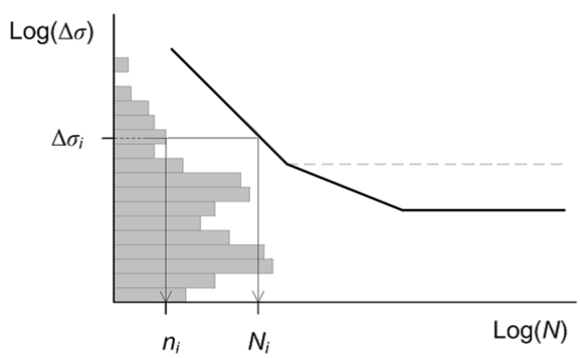

Fig. 6. Damage accumulation calculation

used method is the linear *Palmgren-Miner* damage rule [17], which can be simply stated as follows:

$$D_d = \sum \frac{n_{Ei}}{N_{Ri}} \leq 1.0, \tag{4}$$

where D_d – damage level; n_{Ei} – number of cycles occurring at stress range magnitude $\gamma_{Ff}\Delta\sigma_i$ of a stress spectrum; N_{Ri} – number of cycles corresponding to a particular fatigue strength at stress range magnitude $\gamma_{Ff}\Delta\sigma_i$; $\gamma_{Ff} = 1.0$ – fatigue safety coefficient according to Eurocode [15].

For critical construction details (determined in phase I) the stress ranges $\Delta\sigma_i$ resulting from crossing the bridge by each lorry of FLM 4 were calculated, using the refined 3 D model of the superstructure (the girder as well as the deck). The number of cycles occurring at each stress range magnitude $\gamma_{Ff}\Delta\sigma_i$ of a stress spectrum was calculated on the basis of estimated N_{obs}. Thus the stress history for the selected bridge members was established. N_{Ri} was determined using the S-N curve-based classification method, i.e. for fatigue resistance at $N_C = 2 \cdot 10^6$ cycles (detail category $\Delta\sigma_C$). No updated material resistance information was used in phase II. As is case of phase I, fatigue resistance $\Delta\sigma_C = 71$ MPa for riveted joints according to [16] and $\Delta\sigma_C = 80$ MPa for welded joints in rolled stringers according to Eurocode [13] were assumed in damage accumulation calculation.

The final results for the assumed 10-year service life of the bridge are shown in Table 3. The damage accumulation calculation revealed that most critical members of the bridge had finished their service life due to fatigue ($D_d > 1.0$). It means that in these members fatigue cracks could be initiated and propagated. The most fatigue-endangered elements are the deck stringers and diagonals No. 125, situated in the middle of the second and fourth spans of the bridge. In case of the stringers the hot spots are located in the welds between the vertical rib and the bottom flange. As for the diagonals, their riveted connections with both girder chords are likely to crack. The cracks in these locations are in general invisible (under gusset plates) and therefore very dangerous. In addition, diagonals No. 125 are non-redundant truss members and their cracking may be of high risk for a break down or collapse of the bridge and for the safety of users. A fatigue crack, initiated in a secondary member, such as the deck stringer, is usually not of major importance for a hazard scenario of the bridge.

The detailed investigation (phase II), based mainly on damage accumulation calculation, is usually complimented with the quantitative NDT inspection, and this was the case of the discussed bridge. Each NDT method has limits with regard to its application and level of accuracy under different testing conditions. Testing and acceptability of flaws in the welds are well investigated and standardised [20]. There is no such a kind of standard for riveted connections. Table 4 describes NDT methods, which can be applied to riveted steel bridges by the specialists who have enough experience in evaluation of measured signals and possibly necessary precautions. The recommendations [14] give a more detailed description and some hints for the use of the available NTD methods.

The main goal of the quantitative NDT inspection was to identify "active" cracks propagating under service load and posing a risk to the bridge. Therefore the acoustic emission technique (AE) was chosen as the best method for "active" as well as subsurface cracks (Fig. 7). Since the probability of crack detection increases by loading the examined members with a high but still admissible load on the bridge which results in opening possible cracks, the testing has been carried out under normal traffic on the bridge. The applied method relies on the analysis of acoustic waves generated by active destructive processes that develop in a bridge under service loads [19, 20]. The signals received by the acoustic sensors located on the structure are compared with reference signal database complied beforehand for specific destructive processes. Thus identified destructive processes are located due to the analysis of differences in time-of-arrival of signals at individual sensors. Identification and location of active destructive processes provides a basis for monitoring which makes it possible to evaluate the technical condition of a bridge. The advantage of the method lies in the fact that it is possible to space sensors in such a way that their measurement ranges cover the whole of the examined bridge structure.

Table 3
The results of damage accumulation calculation for the critical members

Member type	Diagonals								Bottom chord			Deck	
Member No.	105	108	119	123	125	128	137	141	315	321	322	P1	P2
Damage D_d	1.77	1.84	0.63	0.61	5.64	0.74	0.29	1.02	2.08	1.98	0.78	5.40	8.03

Table 4
Available NDT methods applicable to riveted steel bridges

No.	Shortcut	Method	Application
1	MT	Magnetic particle inspection	Surface cracks
2	PT	Colour penetration test	Surface cracks
3	RT	Radiographic inspection	Surface and subsurface cracks also in sandwiched elements (connections)
4	UT	Ultrasonic inspection	Material thickness, in some special cases crack detection possible in riveted sections and in rolled sections
5	ET	Eddy current technique	Crack detection in rivet holes after rivets were removed, cracks in thin plates
6	AE	Acoustic emission techniques	Surface cracks, subsurface cracks, identification of "active" cracks only
7	FOS	Fibre optical sensors	Monitoring during the crack propagation

Fig. 7. Acoustic sensors mounted on the gusset plate of the critical diagonal

The intensive AE testing campaign revealed, that a few fatigue microckacks could be initiated in the vicinity of the gusset plates in the truss members [21]. The corrosion destructive processes in riveted connections were also identified as the potential source of microcracks. The signals received by the acoustic sensors indicated that cracks were active only under heavy traffic (lorries). Despite the small number of recorded signals, it is highly probable that fatigue microcracks will propagate in the near future. The most severe destruction was discovered in the members located close to the bridge expansion joints, which are in very bad state of repair. It may constitute a threat when no maintenance measures are implemented on the bridge in next 10 years of required service life.

6. Expert investigation (phase III)

Since several members of the superstructure failed phase II verification, further action was therefore clearly required and justified. An expert investigation is usually carried out for problems with high consequences in terms of risks or of costs related to a decision. The further assessments using specific tools (high-tech NDT methods, fracture mechanics, probabilistic methods, etc.) can be carried out to help take final decisions.

The idea of using fracture mechanics approach for the calculation of a service life interval is based on the theory that the structure contains small defects. Since the application of fracture mechanics is based on the assumption of an initial crack size, high-tech NDT methods are the most appropriate to help characterise the value of the initial fatigue crack size. Therefore NDT campaign was extended in phase III with the application of more accurate methods.

The qualitative NDT inspection of the critical members and cross-sections was carried out by means of magnetic particle inspection (MT) as well as ultrasonic testing (US). The first method was used to detect any surface cracks of questionable cross-sections. This method is of high accuracy so very small surface cracks can be detected if the surface has been prepared thoroughly. All surfaces under consideration were cleaned with a wire brush. Since the method is not applicable for subsurface damage, the testing extension by means of ultrasonic inspection was required. It is the only method to detect the crack depth and size needed in the prospective fracture mechanics calculation. The commercial ultrasonic equipment with adapted ultrasonic sensors and the oscilloscope was used for inspection (Fig. 8). All hot spots in the rolled sections (stringers) and gusset plates in riveted connections of diagonals were accurately checked against cracks. However, no cracks were found in this comprehensive inspection campaign.

Fig. 8. The qualitative NDT inspection of the critical members and cross-sections

The use of fracture mechanics methods can be useful when the information about a crack size is either known or needed for safety evaluation. This may include situations where a fatigue crack has been detected and information about the remaining fatigue life is required. Since no fatigue cracks were discovered in the bridge superstructure, fracture mechanics calculation was not performed for fatigue assessment of the bridge.

The variations in some parameters required for calculations based on the classification method used for phase I and II can be shown to have a significant effect on the calculated fatigue life. Normally, when this method is applied, the various "input parameters" (e.g. the detail category) are considered deterministic values. But there are many uncertainties in the applied procedure for fatigue assessment, both on "load" and "resistance" site, for example:

- the determined stress spectra in bridge elements are based on FLM4 and corresponding traffic data according to Eurocode [14], not on real heavy traffic measurements on the bridge;
- the extrapolated traffic data into the past as well as into the future is very rough (trend line);
- the fatigue resistance $\Delta\sigma_C$ (detail category) for riveted members according to [16], used in fatigue calculation, is sometimes questionable (e.g. [22]);
- the real condition of the superstructure was not accurately taken into account in FEM model used for fatigue calculation (e.g. corrosion losses of critical members).

One way in which these uncertainties may be clarified explicitly is through the use of probabilistic methods, which can be employed in conjunction with either the classification method (used herein) or the fracture mechanics method. When probabilistic analysis is employed, these deterministic "input" values are replaced with statistical distributions. The probability of failure is then determined for a predefined limit state function.

Probabilistic methods in the current context are primarily for determining the "probability of failure" P_f or "reliability index" β of a given structure or structural component, where:

$$\beta = -\Phi^{-1}(P_f) \tag{5}$$

and Φ is the standard normal cumulative distribution. In case of fatigue, which is a deterioration process, P_f and/or β are usually presented as a function of some measure of time. Various codes and standards prescribe maximum P_f (minimum β) targets for new and existing structures [23]. Comparing the calculated P_f and/or β versus time curves for a given structure or structural component with these target values, an estimate of the remaining fatigue life can be made.

A necessary step in formulating the probabilistic model is to define a limit state function, $G(z_i)$, wherein z_i are the i-probabilistic parameters characterizing either the "load" or the "resistance" of the structural member or detail. The function $G(z_i)$ is defined such that: $G(z_i) > 0$ means the limit state is satisfied, whereas $G(z_i) < 0$ signifies "failure". $G(z_i) = 0$ represents the failure surface (Fig. 9). Several criteria may be considered to define "failure". For the classification method, this may include surpassing a certain damage level, D_d or fatigue safety ratio μ_{fat}.

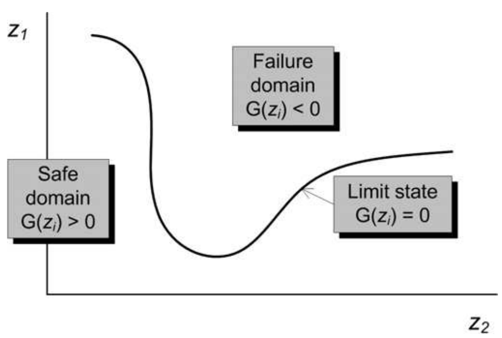

Fig. 9. Limit state function $G(z_i)$

The generic formulation for the limit state function $G(z_i)$ of a structural problem wherein the "load" or "load effects" on the structure and its "resistance" are fully independent takes the following form:

$$G(z_i) = R(z_i)_{i=1\ldots j} - S(z_i)_{i=j+1\ldots k}. \tag{6}$$

In expression (6), the first term on the right side can be roughly equated to a measure of the "resistance" of the structural detail (R), and the second term as a measure of the "load" or "load effects" (S). With the limit state function formulated, the next step is to replace the more important deterministic variables that occur in the function with appropriate probabilistic distributions. The "importance" of the various input parameters is determined by the extent to which their variation affects the result of the calculation. A sensitivity study is recommended to ensure that the most important parameters are treated in a probabilistic manner.

Many researchers have proposed probabilistic models that allow to see the effect that various inspection strategies will have on the probability of failure of a given structure over time. The minimum acceptable safety level should be specified by a competent road authority (some indications can also be found in different codes). As noted in [23], only remedial actions will have an effect on the true probability of failure of the structure. The inspection is only a means to reduce the level of ignorance of the actual state of the bridge or, in other words, to modify the probabilistic distributions for the various input parameters based on the new information. Therefore, instead of extension the fatigue assessment with specific sophisticated tools, the remedial measures were considered for the bridge.

Because none of the assessments provide sufficient justification for leaving bridge in service "as it is", then suitable remedial measures had to be implemented. The possibilities include repair, strengthening, structural health monitoring, reduction of traffic loads or volume, and at worst: closing and dismantling the structure. Since no evident fatigue failures has occurred so far on the bridge, neither repair nor strengthening were taken into account. However, the fatigue assessment revealed with high probability that initiation and propagation of fatigue cracks were likely to take place in some non-redundant members within the actual structural system. It constitutes high risk of a collapse of the bridge and for the safety of users. Therefore the reduction of traffic loads was recommended and the structural health monitoring system (SHM) was designed to monitor the structure. The implementation of such remedial measures would ensure safety of the bridge and its users for next 10 years required by the road administration. Despite the thorough justification of these recommendations, the administration decided to close the bridge and move the traffic on the new bridge built parallel to the existing one.

7. Conclusions

The applied approach of fatigue assessment of existing riveted steel bridges follows the principles and application rules of the Eurocodes and provides a scheme with various levels of analysis: a basic level with general methods and further levels, more complex and sophisticated and requiring specific experience and knowledge. All important parameters, on the "load" and "resistance" site, having the influence on the reliability and safety of the structure, are taken into account in this approach. The stepwise procedure is arranged in such a way that with each step the input for the assessment fits more accurately to the structure assessed and the assessment itself becomes more realistic. By means of the similar approach the remaining fatigue life can be calculated for all steel bridges, especially for old riveted structures.

In the case presented in the paper, calculation was performed for the non-redundant diagonal members of the main truss which were estimated to be the most critical for fatigue according to phase I preliminary evaluation. Phase II assessment showed that the studied members had no remaining fatigue life. Due to the fact that these members could no longer be considered safe, further measures were necessary. Since phase III investigation using NDT revealed that no cracks had been found, the adapted final solution could have resulted in the two following possible further measures: the reduction of traffic loads along with the implementation of monitoring system. However, the owner has decided to close the bridge instead.

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