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THE INFLUENCE OF HIGHER MODES VIBRATIONS ON LOCAL CRACKS IN NODE OF LATTICE GIRDERS BRIDGES

S. PRADELOK[1]

Several previous investigations on failure of a certain type lattice girders railway bridge (on so called BJD line) have not convincingly explained reasons nor have they described potential hazards. This paper attempts to provide an answer, employing static, dynamic, and fatigue analysis of the structure, focusing on previously not analyzed vibrations of elements constituting a lattice node. Detailed models of two types of such nodes – damaged and non- damaged were compared, inside carefully defined limits of applicability.

Key words: bridge, vibrations, cracking, detailed models, dynamic analysis, fatigue analysis.

1. Introduction

Standard dynamic analysis of bridges focuses on principle vibration modes. Indeed, for main bridge elements (girders, lattice girder members, main frames elements) dynamic influences affect the structure mainly at the lower frequencies. There are, however, documented failures that cannot be explained neither through static nor lower frequency dynamic effects.

The subject of this paper is a damage to a typical rail lattice girder bridge with lower bending in the lower chord. Several studies (Weseli, [5], Weseli, Pradelok [6÷8], Pradelok [9, 10]), as well as design projects [1, 2] attempted to explain the damage. Increasingly more precise and thorough conjectures regarding the reasons were formulated. They were, however, equally applicable to the second, "twin" structure right next to the subject one. Both were subject identical loads and conditions, yet no signs of failure were noticed in the second one. That obviously prompted questions on actual damage reasons.

Correct determination of the failure's reasons was of critical importance, as number of structures were built in Poland on basis of this standard 93 m span one track bridge design.

[1] PhD, Department of Roads and Bridges, Faculty of Civil Engineering, Silesian University of Technology, Gliwice, Poland, e-mail: stefan.pradelok@polsl.pl

It was therefore fundamental first to establish and analyse differences in the two apparently identical bays as-built status, and only then comparing that to theoretical results.

2. GENERAL CHARACTERISTIC OF THE STRUCTURE

The crossing is part of one track line Brzezinka-Jęzor-Dorota (BJD). It was built between 1982 and 1989 and consists of 6 simply supported bays, built to several different designs. 4 bays are plate girders, and are not considered in this paper. The main two bays are trusses (Fig. 1).

Fig. 1. Side view of the fault bay.
Rys. 1. Widok z boku na przęsło uszkodzone

The cracks in the node were spotted during a routine track walk – down. A more detailed examination followed, resulting in finding further, smaller cracks near other nodes of the same bay. No cracks, however, were found in the adjacent "twin" bay. The damages were described in details (WESELI, PRADELOK [6, 7]).

Further in this paper we will be caling the river bridge "non-fault bay" and the over-railway bridge "fault bay". All bays were designed and built of grades 18G2A and St3M steel.

The Maczki side truss is built over Biala Przemsza river, while Jęzor side over four railway tracks, two of which are for transportation of sand, and two others are regular railway tracks of Katowice – Krakow line. The track is laid in a reinforced concrete channel filled with rubble, as shown on cross section (Fig. 2).

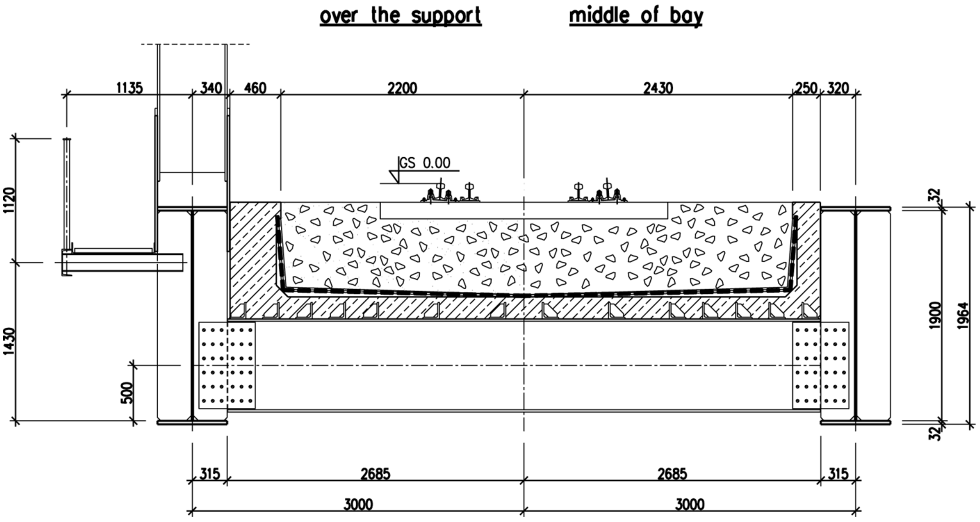

Fig. 2. Cross section.
Rys. 2. Przekrój poprzeczny

Mid bays are two simple supported trusses with track at the bottom chord. Their span is 93 m of a very similar design. Main trusses appear to be a standard Warren type truss with parallel chords. They are not, however, a typical truss, as their bottom cords are subject to bending as a result of cross bars configuration connected in nodes and in places between them. In addition to the above, the diagonal bars alignments cross at the top flange of the bottom chord (see Fig. 3). The girder therefore can be considered a beam subject to bending, reinforced with a truss.

The node structure (Fig. 3) is not typical either. The gusset plates are not, as usually, connected to the top flanges only, but extended to the chord ribs.

The main girders in each truss bay are braced horizontally in the plane of upper chords with K type bracings, and in the bottom chord – with lattice bracing and concrete channel supporting the track, which is supported on cross bars, at 3.1 m distances. The bridge reinforced concrete slab is banded together with the cross bars, and split twice with dilatations, where the cross bars spacing is 0.8 m as shown in Fig. 3.

3. DIFFERENCES IN CONSTRUCTION LATTICE SPANS

Number of bridges were built with truss bays constructed according to this typical design of a single track 93 m span bridge. Over the years it was observed that the structure is particularly sensitive to dynamic influences. Minor design faults spotted

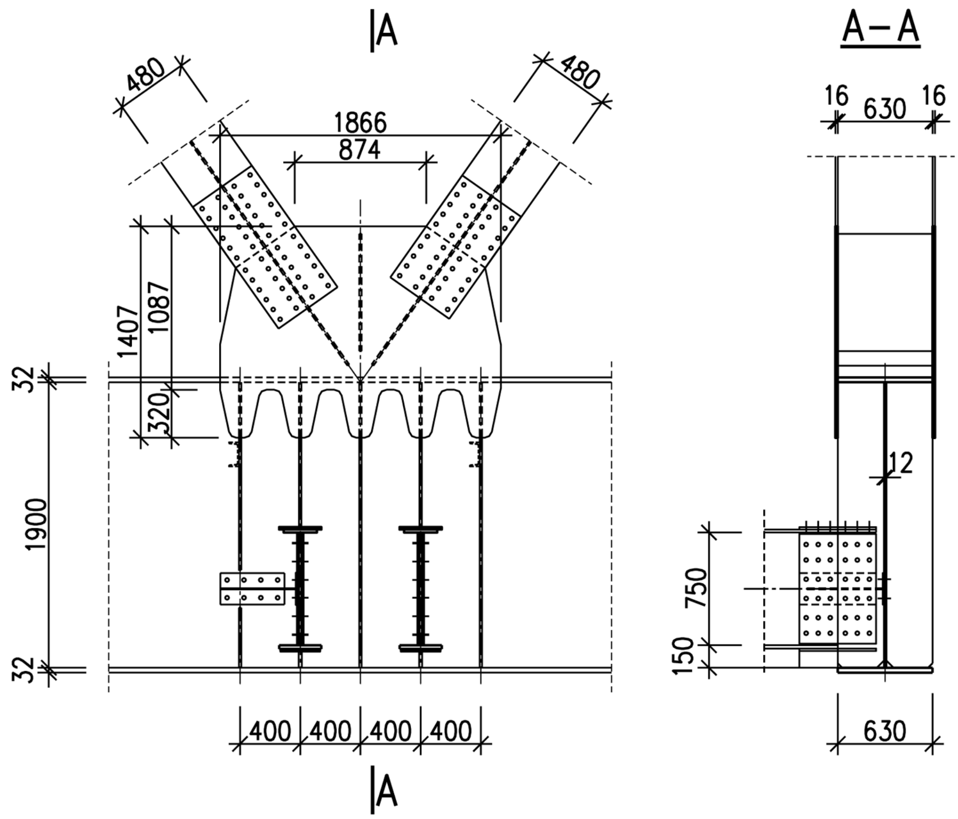

Fig. 3. Fault bay. Node construction near expansion joint in a reinforced concrete track channel.
Rys. 3. Przęsło uszkodzone. Konstrukcja węzła przy przerwie dylatacyjnej w żelbetowym korycie balastowym

were gradually eliminated from the design. As a result these apparently identical trusses of the subject crossing were substantially different in details.

The bay over the railway tracks is transversely braced stronger, with portal frames in plane of each diagonal bar. In the other bay the static stability is maintained only thanks to the edge portals.

Both trusses differ also in the way the members' connections are developed. In the fault bay there are extended nodes and the diagonal member webs are cut 80 mm short of the bottom chord. In the other bay, the diagonal members have trapezoidal perforations up to 800 mm away from the bottom chord, and the gusset plates are much smaller.

Cross members in the fault bay are connected with the bottom chord in a way that allows partial transfer of the bending moment. In the other bay connection transfers only shear. Moreover the cross members near the dilatations in the non – fault bay have ties in the middle of their spans, unlike the ones in the fault bay, which have no tie at all.

4. Models of analysed nodes

The described differences influence the structures only locally, and there was a need to build a model that would allow capturing their effect. That prompted development of theoretical models of the nodes of both bays, as finite element plates. Such models encompassed 6.2 m (=2.7+0.8+2.7 m) the length of bottom chord and half the lengths of four cross and two diagonal members (Fig. 5). Flanges, gusset plates, ribs, diagonal and cross members, were modelled as precisely as practically required, taking into account the differences between the bays.

Support conditions of such models are of great importance. To obtain most accurate results of the static and dynamic analyses, was decided not to support the node model on a dummy spring supports which would make it impossible to observe accurate shapes of natural frequency modes – the main reason to build the model.

Instead the node model was nested in a three – dimensional model of the truss, which was in turn supported on rigid supports that restrict the same number freedom dimensions as the actual bearings (Fig. 4). The connection of the node and truss models allows for correct freedom of movement.

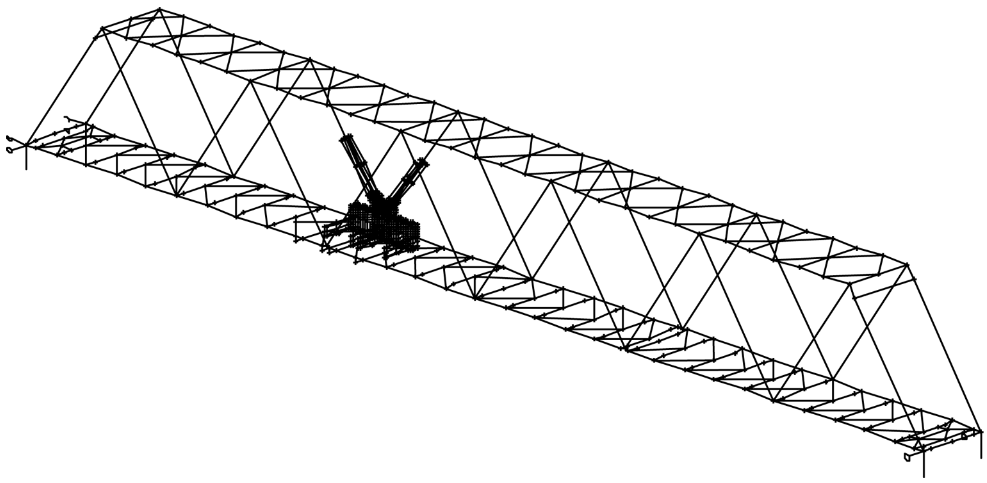

Fig. 4. Connection of membrane model of the node to the truss one.
Rys. 4. Zamocowanie powłokowego modelu węzła w prętowej kratownicy

Thin, weightless membrane elements were introduced to model properly rigidity of connections between the truss members and the membrane finite elements.

The model purposefully did not include the concrete channel (200 mm thick), as that would prevent movements of the cross beams. Connecting of the cross beams

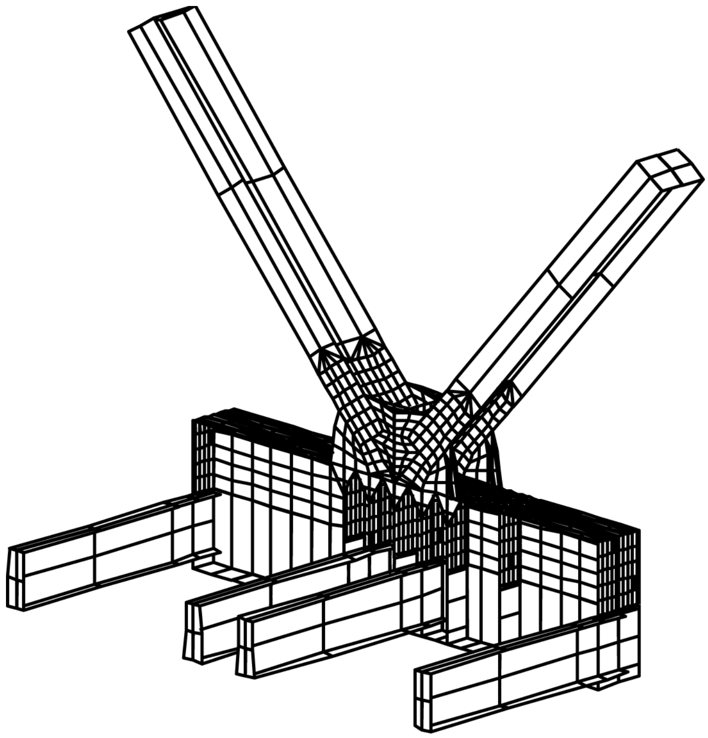

Fig. 5. Membrane model of the node.
Rys. 5. Powłokowy model węzła kratownicy

through steel angles with the concrete element is questionable in any case (Fig. 2). The influence of the omitted concrete elements was modelled with additional mass placed in the bridge nodes.

5. Static Analysis

Such developed models of fault and non-fault bays had loads applied in identical way: the dead load, weight of the concrete channel and the track with the rubble sub base, and live loads of the actual trains travelling over the bridge. Surveyed differences of supports displacements were also reflected. Normal stress σ_{xx}, in the characteristic combination at the fault location, are shown in Table 1.

As it can be observed, the differences in the level of strain between the identically loaded models of the "twin" bays are substantial. There are big differences of normal stress σ_{xx} in the top and middle layers which indicate principal influence of local bending of the top flange in the bottom truss chord.

Table 1

Normal stresses σ_{xx} at the fault location.
Naprężenia normalne σ_{xx} w miejscu uszkodzenia

	Stress σ_{xx} [MPa]	
	Faulty node	Non-Faulty Node
Top Layer	316.5	271.0
Mid layer	259.6	239.5
Difference	56.9	31.5

The calculated differences do not unambiguously indicate the reason for the cracks, as the stress levels observed were below the material capacity. A fatigue related failure was therefore suspected which called for further dynamic analysis.

6. Modal Analysis

The modal analysis showed appearance on a relatively low positions modes of natural frequencies of near dilatations cross beams. In the considered set of shapes they appeared in both models, however there were more of them present at the failing node. All had the period of 0.06 s (which is an equivalent to frequency of about 17 Hz). Fig. 6 shows one of the natural frequency shapes observed in the model of the failing node.

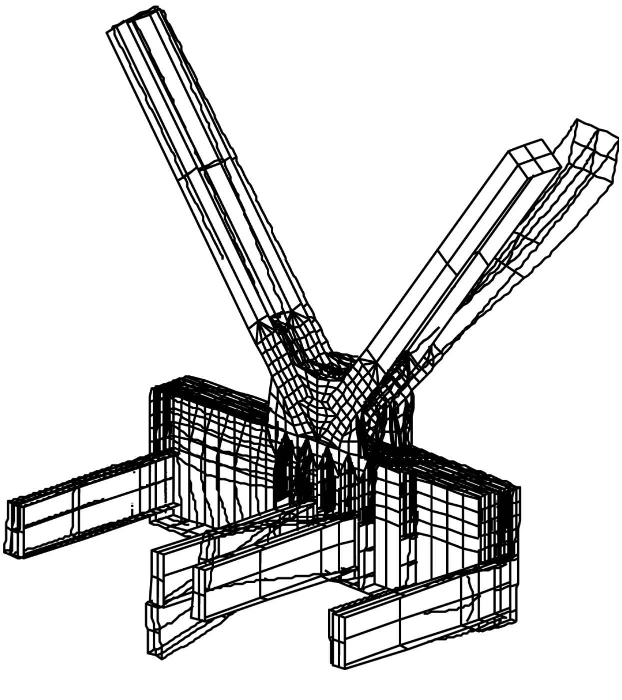

Fig. 6. Fault bay. Period 0.06 s natural frequency shape.
Rys. 6. Przęsło uszkodzone. Postać drgań poprzecznic o okresie 0,06 s

7. Time analysis

Further on, a time analysis was also conducted (integration of movement equations). Load was assumed to be exerted by travel of 4 consecutive axles of 165 tonnes ET41 locomotive. The axles are at 3.05 m distances which roughly coincides with the spacing of cross beams in the bridge (3.1 m). It was assumed further that the speed of the locomotive travel to be 0.23 s for this 3.1 m distance (equivalent of 48.5 km /h). Change in normal stress σ_{xx} of in the upper layer in the place of failure of the upper flange of the girder was shown in Fig. 7. Amplitude of stresses reaches 34 MPa. It can be also observed that there is a significant increase of normal stresses σ_{xx} at the time of the axle crossing of the cross beam, and next 4 cycles of natural frequency vibrations (with period of 0.06 s) which generate quickly fading vibrations. The approach of the next axle (after 0.46 s) initiates impulse to jump in the normal stresses, followed again by 4 cycles of fading natural frequency vibrations. All that is repeated 4 times. Approach of last, fourth axle happens after 0.92 s. Local vibrations of the cross beams of 0.06 s are damped after a few cycles and the graph shows stresses related second shape of natural vibrations of the entire structure with a period of 0.59 s.

Figure 8 shows a change in the normal stresses σ_{xx} in the upper layer of a location corresponding to that displaying failure in the fault bay. The stresses amplitude is about 16 MPa. The remaining characteristics are similar to those shown in Fig. 7.

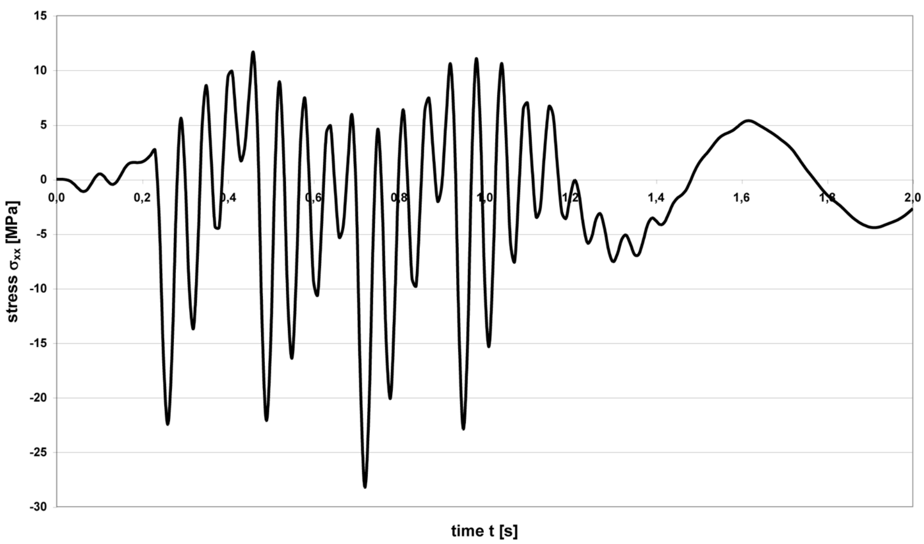

Fig. 7. Fault bay. Time analysis of normal stresses σ_{xx} at the location of failure.
Rys. 7. Przęsło uszkodzone. Analiza czasowa naprężeń normalnych σ_{xx} w miejscu uszkodzenia

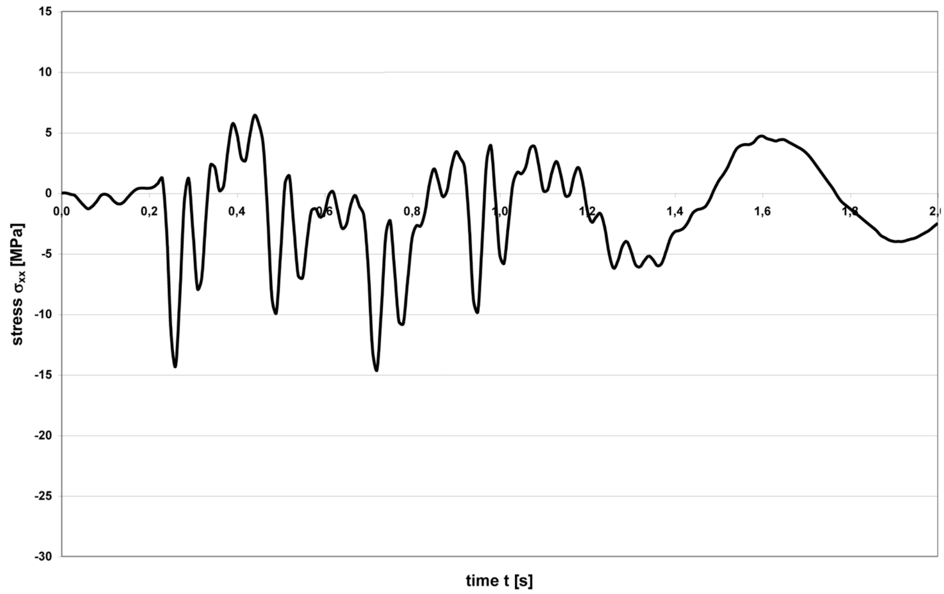

Fig. 8. Non- fault bay. Time analysis of normal stresses σ_{xx} at the corresponding location.
Rys. 8. Przęsło nieuszkodzone. Analiza czasowa naprężeń normalnych σ_{xx} w odpowiednim miejscu

8. FIELD TESTS

Several field tests were conducted. The results of one of them are presented herein showing the loading of the structure after it was rectified and reinforced. The conducted tests provide good basis to assess quality of theoretical calculation models. They also verify theoretical results of modal and time analyses.

Examples of change of deflection in time "t" registered with electro-resistance tensometer T1, were obtained during passing of a load at 50 km/h speed toward Jęzor, and the arrangement is shown in Fig. 9. Both drawings are divided into 4 parts, marked a through d.

The signal shown in part a of Fig. 9 is heavily distorted to a point where it is impossible to conduct analysis of vibrations using analysis graphs of power spectral density (PSD). After filtering of the measurement signal, it is possible to run some analysis. It is evident in part b of the graph where it is already possible to see the characteristic of the signal and range of changes during the load movement. In part c a heap of spectral density can be observed at frequency F_{max} = 4.58 Hz. However, only at the part d, after selection of 5 s part of registered measurement signal, obtained during the loading of the structure for analysis, and with the lower treshhold of the bandpass filter raised to F_d =10 Hz heaps at PSD graphs at frequencies between 16.4 Hz and 19.6 Hz were observed.

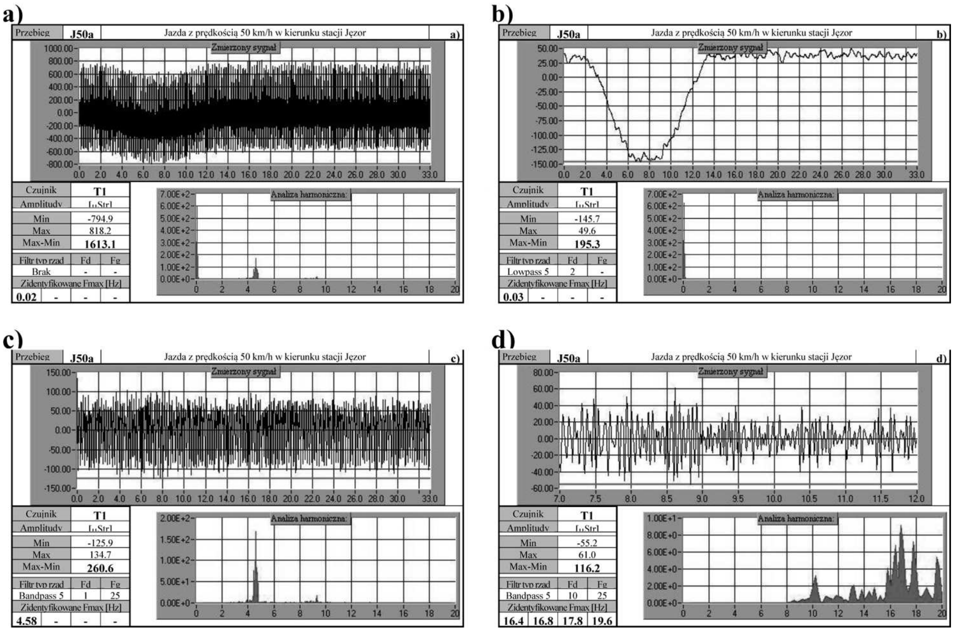

Fig. 9. Tensometer T1. Drive at 50 km/h speed toward Jęzor.
Rys. 9. Tensometr T1. Jazda z prędkością 50km/h w kierunku stacji Jęzor

This confirms correctness of the calculation model developed. The dynamic tests proved occurrence of vibrations of frequencies affecting the strain of the structures near the nodes. The way the electro resistance tensometers (T) were deployed enabled registration of local shapes of vibrations in the analysed node.

9. Fatigue analysis

The fatigue analysis was performed using [11] software package employing the Wöhler function. Excess of the limit was established from linear approximation of the Smith Curve to be σ_g =333.3 MPa and Z_{rc} =142.9 MPa, as the first falls in the plastic range of the structure's steel, and both values are connected through empirically established code η value with theorethical model. Due to the presence of cross weld at the place of crack's beginning, the notch factor was assued to be β =3.4.

With established levels of stresses due to dead (see 5) and live (see 7) loads, a coefficient of accumulation of failures during the train movement was developed, in accordance to Palmgren-Miner theory. In the unfailing node D=0.0005 <<1. That proved that the bridge was in the range of permanent fatigue resistance. In the failing node D=23.5 >>1. Such significant excess of value 1, and huge difference between the values for failing and not – failing nodes proves that there is a qualitative difference between these two nodes work conditions. It proves lack of fatigue resistance and indicates possibility of soon development of cracks in the failing node.

10. CONCLUSIONS

The presented calculation analysis, confirmed through field tests, made possible an unambiguous establishment of the failure reasons in one of the bays and lack of thereof in the other.

For the analysis, it was necessary to analyse local static and dynamic influences, which could only be captured with precise modelling of the nodes. Such local influences with vibrations of higher frequencies, added to the global stress levels explain precisely reasons for failures.

Despite the fact that this paper discusses a particular structure, the established results are of a general nature. Similarity of analysed bays provided an opportunity to verify the design solutions employed in similar structures. The presented results identify potential risks and indicate preferred solutions. Established differences between the failing and non- failing nodes arrangements allow to develop guidelines preventing future failures of similar structures.

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EXTENDED ABSTRACT

The research project was aimed at finding out the reasons behind lattice girders nodes cracking of railway bridge. The study outlines history of several previous investigations, that did not convincingly explained reasons of failure. The lattice girders that are subject of the study are a part of a railway bridge, which consists of six independent, freely supported steel spans. Four of them are mill rolled and two are apparently identical lattice girders. In one of lattice girders some cracking has been noticed. In second, twin lattice girder, supporting the same rail, such cracking does not occur. The research project focused on specific, particular circumstances for comparative investigation. Detailed models of two types of such nodes (damaged and non-damaged), as well as detailed models before and after repair were analyzed. The study suggests that the main reasons of damage are local static and dynamic influences and all real level strains.

Modern numeric analysis of bridges provides very accurate results, dependant largely on accuracy of the calculation model. It is therefore necessary to verify theoretical calculation results empirically. Field tests provide good basis for verification and analysis of quality of theoretical models. The paper describes how a specific placement of deflection sensors made possible recording of main natural vibration frequencies in the subject bridge. The empirically detected frequencies confirmed the ones identified by the theoretical analysis. Specific placement of sensors (accelerometers and tensometers) allowed further detection of local vibration patterns in two nodes being subject to the tests. Prevailing part of the power spectrum density was identified near the theoretically obtained natural frequencies of transverse girders near to the dilatation.

The analysis, confirmed by the empirical tests, allowed unambiguous explanation of the damage cause, and the reason it appeared only in one of two twin bridge spans.

WPŁYW WYŻSZYCH POSTACI DRGAŃ NA ZAISTNIENIE LOKALNYCH SPĘKAŃ W WĘŹLE MOSTU KRATOWEGO

S t r e s z c z e n i e

W kolejowym moście kratowym zdarzyły się trudne do wytłumaczenia awarie. Mimo licznych wcześniejszych ekspertyz i badań naukowych nie udało się w sposób jednoznaczny ustalić przyczyn tych awarii. Ustrój nośny, który uległ awarii, jest częścią większego obiektu mostowego złożonego z sześciu niezależnych, swobodnie podpartych stalowych przęseł. Cztery z nich to przęsła blachownicowe, a dwa pozostałe to bliźniacze przęsła kratowe. W blachownicowych przęsłach nie stwierdzono żadnych uszkodzeń. Natomiast w jednym z przęseł kratowych zauważono pęknięcia w dolnym pasie kratownicy. Pojawiły się one w węzłach położonych w okolicy przerwy w betonowym korycie pomostu. W drugim, bliźniaczym przęśle kratowym, położonym w tym samym torze, nie stwierdzono żadnych uszkodzeń pomimo wielokrotnych i dokładnych oględzin. Wykorzystano te szczególne okoliczności sprzyjające przeprowadzeniu badań porównawczych.

W celu wyjaśnienia przyczyn pękania zbudowano szczegółowe modele obliczeniowe węzłów kratowego ustroju nośnego mostu kolejowego. Przedmiotem analiz były dwa węzły kratownic. Pierwszy to ten węzeł kratownicy, który uległ awarii, a drugi to jego odpowiednik w nieuszkodzonej kratownicy. W wyniku przeprowadzonych analiz obliczeniowych wykazano, że przyczyną uszkodzeń są lokalne wpływy statyczne i dynamiczne działające na tle rzeczywistego poziomu naprężeń.

Współczesne metody numerycznej analizy mostów pozwalają na uzyskanie bardzo dokładnych wyników. Nie jest to jednak cała prawda, gdyż wyniki te są zależne od dokładności danych wprowadzonych do modelu obliczeniowego. Z tych względów należy otrzymane teoretycznie wyniki zweryfikować

w badaniach terenowych. Badania terenowe dają dobrą podstawę do przeprowadzenia analizy jakościowej teoretycznych modeli obliczeniowych. Umożliwiają one weryfikację otrzymanych wyników z tych analiz.

Przeprowadzona analiza obliczeniowa, potwierdzona badaniami terenowymi, pozwoliła jednoznacznie wyjaśnić przyczyny awarii. Zdołano obliczeniowo wykazać, dlaczego doszło do awarii w jednym przęśle kratowym, a w drugim, bliźniaczym, identycznie obciążonym przęśle kratowym takie uszkodzenia nie powstały.

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