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DYNAMIC TESTS OF PONY TRUSS RAILWAY BRIDGE UNDER NORMAL OPERATION

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Abstract
The article describes the application of dynamic tests during normal operation in assessing the technical condition of the railway bridge on the Nysa Łużycka River. The investigated structure consists of six steel pony truss spans. Operational modal analysis was used to identify dynamic parameters. The results obtained were applied to verify the numerical model used in the assessment process of technical condition and in determining the current load capacity of the structure created in the 1950s. The experiment conducted allowed us to observe the dynamic response of the pony-truss bridges. The study demonstrated that dynamic tests during normal operation and the use of other available environmental excitations could complement other non-destructive diagnostic methods. This proves that we can obtain useful information about the dynamic properties of a structure without closing the traffic and without the application of the ordinary dynamic test load.

Keywords: **Operational modal analysis; Railway bridge; Pony truss.**

1. INTRODUCTION

Polish Railway Lines (PKP S.A.) operates around 25,600 engineering structures, including more than 3.3 thousand bridges and viaducts. The total length of bridges and viaducts in operation is approx. 133 km. About 45% of them are over 100 years old, and some fragments of the oldest bridge structures still in use date back to 1840 [1, 2]. Currently, a renewal program of railway lines is being implemented in Poland. Ageing railway infrastructure is a major problem for traffic safety. It specifically applies to bridge structures, especially those located outside the main communication corridors. Structures for which there is fear that their technical condition may affect traffic safety are subject to special inspections and various types of tests. The purpose of this research is to assess the possibility of further operation of the structure or the need to replace it.

The paper describes the application of dynamic tests during normal operation in the assessment process of the technical condition of a pony truss steel railway bridge (Fig. 1). The identification of modal parameters was performed using Operational Modal Analysis (OMA). The paper presents the results of the identification, with the emphasis placed on the specific dynamic response of the pony-truss structure. The applied layout of the vibration transducers allowed us to explicitly identify several important modes. The results were used to verify the calculation model and to perform additional numerical analyses.

2. DYNAMIC TESTS WITH ONGOING TRAFFIC

2.1. Typical dynamic tests

One of the methods used to assess the technical condition of a structure is the studies under test load [3], [4]. Such studies require the bridge to be closed. The load tests consist of two types of tests, static and dynamic [5].

Figure 1.
Bridge under study

The static test identifies the effects of static load, i.e. the most frequent subsidence and displacements arising from such loads.

Dynamic tests usually involve making a series of passages at different speeds of an arbitrarily selected train vehicle or the entire train. The observation involves the dynamic effects, displacements, and accelerations caused by the passage of the test rolling stock. Analysis of a dynamic test is limited to a simple analysis of signals [6]. This approach may prove effective in the case of simple bridges with well-separated forms of free vibration, but this involves two main limitations. The key limitation is the necessity of closing traffic on the examined object and on the adjacent section of the railway line, which allows the required speed of the test rolling stock to be reached. Another, not less significant limitation, involves the cost of renting and insuring the test railway vehicle.

2.2. Research with ongoing traffic using Operational Modal Analysis

As an alternative to typical dynamic test loads, we propose tests in normal traffic using all available, random, uncorrelated excitation, i.e., operational modal analysis [7–11]. The experiment described in the paper was carried out using the said OMA analysis.

In the research with the application of OMA technology, for the purposes of identification, we make use of vibrations caused by normal operation and vibrations induced by other, unknown excitation forces such as, e.g. wind or movement under the bridge span. Tests under ongoing traffic conditions allow us to make observations of the dynamic behaviour of the structure under real operating conditions. Identification encompasses the entire dynamic system, i.e., the span and its actual rather than idealised support system conditions.

In research under normal traffic, the impact exerted by the presence of vehicles on the bridge on the estimated modal parameters is becoming more important. In the described example, the weight of the locomotive is almost twice the mass of the span. For this reason, the signals recorded when the locomotive is on the deck cannot be used to estimate the modal parameters of the structure. However, signals recorded after the load has left the deck contain useful information and the vibrations induced by the passage have amplitudes higher than vibrations typically induced by other excitations. In the analysed structure only part of the recorded response was used for modal parameters estimation, the part containing only free vibrations excited by the tran passage. In this way, possible variability of modal parameters due to changing total mass of the system was addressed.

It is well known that there is a significant impact of environmental conditions on the estimated modal parameters. A change in ambient temperature or other environmental parameters can cause a change in the observed dynamic response [12].

The identification of modal parameters (which most frequently consists of three quantities: frequency, the form of free vibrations, and the corresponding damping [13]) is understood in OMA as a process that correlates the dynamic characteristics of a mathematical model with the physical properties of the system obtained from the measurement data. In OMA, the measurement data obtained from the dynamic response under normal operation are used to estimate the parameters of the model that describe the observed behaviour of the structure. In the experiment presented in the paper, the stochastic subspace identification algorithm based on correlation functions (SSI-COR) described in the paper [14] was used to identify the modal parameters.

3. RAILWAY BORDER BRIDGE ON THE NYSA ŁUŻYCKA RIVER

3.1. Description of the structure

The supporting structure of the bridge, under the active railway track, is made of simple supported single-track truss spans with a bottom ride of the theoretical span of 30.0 m. The bridge across the Nysa Łużycka River belongs to the popular in the 20th century type of bridge structures used to cross medium-span obstacles – half-frame truss systems. The structure was built in the 1950s and has been used for more than 70 years. The specificity of this type of structure is that the upper flanges of the truss girders are not transversely braced (Fig. 2). Computational analyses of this type of structure usually demonstrate that the load-bearing capacity of the system is determined by the buckling of the compressed upper flange of the truss girder.

The sixth span, located on the territory of Germany, was selected for the study. The choice of span was dictated by its availability and by the fact that during the examination of the structure, no significant differences were observed between successive spans and bridge supports.

The bridge was built diagonally with respect to the obstacle and that is why the truss girders are shifted with respect to each other by a 3 m long field. Such a shift affects the shape of the observed vibration forms described in the following part of the paper.

3.2. MES model of the structure

Two models of the structure were prepared, a preliminary model for the planning and realisation needs of the experiment in the Autodesk Robot Structural Analysis (ARSA) programme and a model taking into account the research results in the Midas Civil programme. For the purposes of this work, only the results of the preliminary model were addressed.

The structure of the sixth span was modelled using finite elements (Fig. 4). Steel bridges with riveted joints are characterised by a high uncertainty in estimation of the stiffness of the joints. At the same time, riveted joints are a source of non-linearity, which is reflected in the estimated modal parameters [15]. The quoted model allowed for the articulated joint of the stringer supporting the track with cross beams, due to the susceptibility of the connection of these two members, which is difficult to assess. Boundary conditions represented theoretical boundary conditions derived from the visual inspection of the structure. The FE model lacks representation of some local stiffening elements found in the real structure. The toal mass of the structural and non-structural elements was also taken into

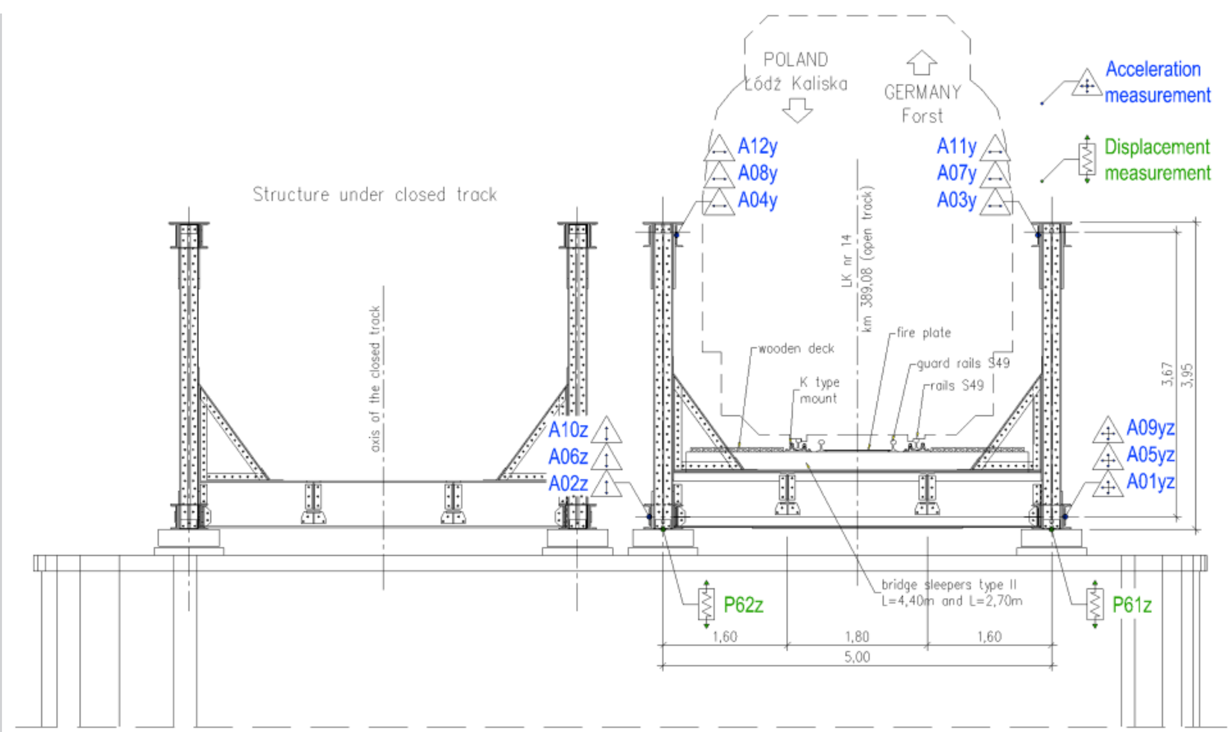

Figure 2.
Cross section of the the sixth span of discussed bridge with spatial transducer layout; suffix y denotes the transverse uniaxial sensor, z denotes the vertical uniaxial sensor and yz denotes the transverse and vertical biaxial sensor

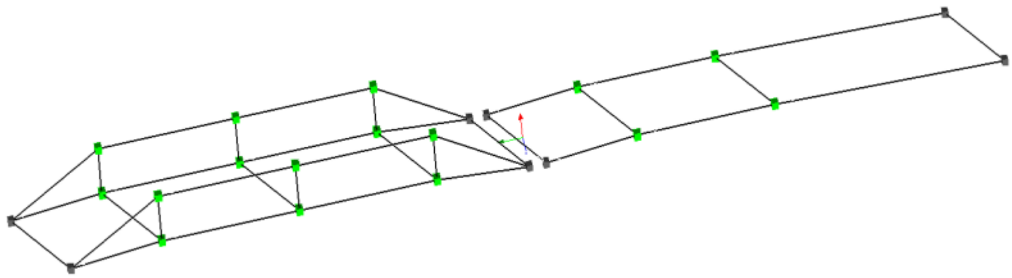

Figure 3.
Accelerometer layout in spans no. five (on the right) and six (on the left)

consideration, because of its importance when it comes to modal analysis results.

3.3. Research schedule and the course of the experiment

The distribution of the measurement points in the cross section of span no. six is presented in Fig. 2. Two inductive sensors of linear displacements were placed in the middle of the span (P61 and P62). In turn, the accelerometers were placed in three sections along the length of the span (15 single-axis transducers). During the experiment, the fifth span was observed, but the scope of the measurements was smaller. The measurement consisted only of vertical displacements in the middle of the span (P51 and P52) and vertical accelerations at four points on the deck. The observation of the fifth span was supposed to find possible differences in the dynamic response of the neighbouring spans. The presentation of a detailed sensor layout for span no. five was omitted. A general spatial layout of the measurement points for both spans (five and six) is presented in Fig. 3. The spatial arrangement of the sensors is clearly represented in this figure. The studies did not include observations of supports or points of the structure in the immediate vicinity of the support points. Only the highlighted nodes are those observed with accelerometers.

The identification experiment was carried out on one day in stable weather conditions with low wind and without rain. Structure is located away from effective, environmental sources of excitation forces, namely a busy road, etc. Additionally, its size is relatively small, so it is not so prone to being excited by wind. As a consequence identification of less-excites, higher forms of free vibrations is proven to be difficult.

A limited number of available accelerometers enforced the measurements on the tested span to be carried out in two setups: In the first setup (U1), the sensors were placed at the measurement points A01: A08 (Fig. 3); in the second setup (U2), the sensors were placed at the measuring points A01: A04 and A09: A12. During the measurements, a total of almost 5h of signals were recorded.

Through the two application of the mentioned accelerometer setups, we could observe the vibrations of 15 degrees of freedom of the sixth span. It also afforded the possibility to observe higher-order vibration forms. We must remember that any change in the location of the sensors is time-consuming and it enforces a break in the recording process of measurement signals. In addition, the described tests had to be performed in one day, and the client expected only basic information about the behaviour of the structure loaded with real trains. The tests were carried out in consultation with the railway line operator. Thus, we had access to the parameters of the passing rolling stock and to the estimated time of their passage. The recorded passages are summarised in Tab. 1.

Tabela 1.
Summary of recorded real train passages

No.	Code name	Description	direction of passage	Accelerometer setup	Hour of pagage
1	T1	Freight train	Forst-Zasieki	U1	11:27
2	T2	Freight train	Forst-Zasieki	U1	13:11
3	T3	Freight train	Zasieki-Forst	U2	15:00
4	L1	Heavy locomotive	Forst-Zasieki	U2	15:16
5	O1	Railcar	Zasieki-Forst	U1	16:33
6	O2	Railcar	Forst-Zasieki	U1	16:41

Tabela 2.
FEM predicted and identified modes of vibration

FEM		n_{OMA}/n_t	OMA				Times identified	Mode shape description
No.	n_t [Hz]		n_{OMA} [Hz]	U_{nOMA} [Hz]	ξ [%]	U_ξ [%]		
1	**5.70**	0.81	**4.60**	0.02	0.66	0.28	9	I lateral bending
2	**7.29**	1,07	**7.83**	0.14	1.24	1.33	9	I vertical bending
3	**10.50**	0.97	**10.14**	0.11	0.38	0.13	8	I torsional along x axis
4	**10.94**	0.92	**10.06**	-	0.46	-	1	II upper flange in phase
5	**11.43**	0.96	**10.95**	0.10	0.29	0.09	9	I upper flange out of phase
6	**11.76**	1.03	**12.11**	0.03	0.28	0.16	9	II upper flange out of phase
7	**12.44**	1.08	**13.40**	-	0.41	-	2	III upper flange out of phase
8	**12.78**	1.07	**13.68**	-	1.38	-	1	III upper flange in phase
16	15.48	-	-	-	-	-	0	IV upper flange in phase
17	15.51	-	-	-	-	-	0	IV upper flange out of phase
19	**17.98**	0.95	**17.06**	0.12	0.79	0.46	9	I torsional
20	20.69	-	-	-	-	-	0	II lateral bending
21	21.82	-	-	-	-	-	0	V upper flange in phase
22	21.88	-	-	-	-	-	0	V upper flange out of phase
23	**22.47**	0.95	**21.27**	-	0.34	-	3	II lateral bending
24	29.52	-	-	-	-	-	0	III lateral bending
25	28.22	-	-	-	-	-	0	II torsional

The numerical model allows for obtaining many forms of vibration. Their number corresponds to the total number of degrees of freedom that all nodes have in the numerical model. Due to technical, time, and organisational limitations, the vibrations in the examined object were recorded only at a few points of the structure and along with the selected directions. This results in the observability being limited to several pre-selected forms of vibration. It also means that inappropriate placement of sensors can prevent the detection of significant forms of vibration in the tested structure.

3.4. Research results

The applied distribution of vibration transducers allowed us to explicitly identify several basic forms of free vibrations. The identification results are summarised in Table 2. The modes related to local vibrations were not included in the above summary (thus some of the local vibration modes are missing in Table 2). The identified frequency of free vibrations of a given form (n_{OMA}) was determined as the arithmetic mean of several estimations based on various available signals. The same was done with the fraction of critical damping (ξ). The expanded experimental uncertainty (n_{OMA} and $U\xi$) of both of these quantities was also determined to the be equal to double standard deviation of the estimates used to determine the mean value. Some modes were not detected in all estimates. This indicates a weak excitation of these forms of vibration and may highlight a lack of certainty of estimation. In most cases, the uncertainty of the damping estimate is much higher than the uncertainty of the estimate of the frequency of free vibration. Table 2 shows quite close resemblence of OMA vs. FEM mode frequencies. The model can be further improved, but its further improvement is of limited value from an engineering standpoint. Differences in frequency for global vertical modes are of order of not more than 8%. For fundamental horizontal mode, 19% difference is of limited interest for us and can in our opinion be accepted.

Fig. 4 presents two examples of the correctly detected vibration mode shapes. One basic and one higher-order mode of vibration associated with the upper flange of the truss girder. The visualisation of the shape of the form was prepared on a single setup of accelerometers. This is due to the fact that the signal-to-noise ratio in most of the recorded signals was low. Merging the signals from two sensor setups did not give satisfactory results. Propper assemblage of mode shape pair, OMA and FEM, was performed only on visual inspection of mode shapes. No formal way of mode shape comparision, e.g. Modal Assurance Criterion, was implemented.

Fig. 5 presents the displacements of the fifth and sixth spans caused by the passage of a single heavy locomotive (Tab. 1). The signals were prefiltered with a low-pass digital filter of the cutoff frequency of 32 Hz. The

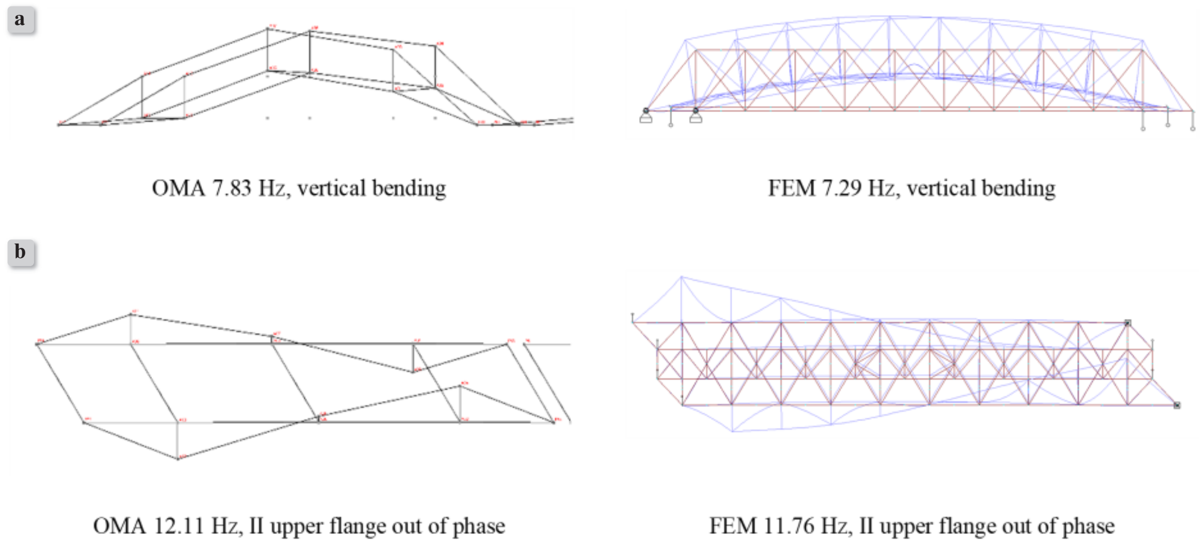

Figure 4.
Comparison of mode shapes estimated from experimental data and calculated using the FE model; a) low order mode shape; b) higher order mode shape

comparable displacements of the two observed spans are visible. Fig. 6 shows the vertical displacements from the beginning of the passage of a freight train pulled by the same heavy locomotive as in the previous graph. This time, the maximum displacements under the locomotive are different in the two spans. Similarly, wagons cause displacements of different amplitudes. No explanation was found for the observed differences. Additional smoother time waveforms were obtained after applying the 0.5 Hz filter. They were used to estimate the dynamic amplification factor (DAF) as the ratio of the signal filtered out with a filter of 32 Hz cutoff frequency and that filtered out with a filter of 0.5 Hz cutoff frequency. In none of the registered passages, the DAF determined in this way does not exceed the value of 1.04.

The presented results (Tab. 2) speak of the difficulty in unambiguously identifying the forms of vibration associated with the transverse movement of the upper truss flanges. We can see small number of the parameter which in turn shows how easy or hard the given mod was to detect. This is due to the insufficient number of measuring points on the top flange. We can also observe that, in the case of small, weakly excited structures, not all expected forms reveal themselves to the extent that their detection.

The results presented involve a short-term measurement campaign. Long-term measurements using a vibration monitoring system could provide more information on the variability of the dynamic response of the structure as environmental conditions change. The weather conditions on the day of the study were recorded and are part of the research records report. This should allow one to relate the information obtained in effect of the described experiment to future results obtained under similar conditions. Significant differences may be a premise that indicates a change in structure.

The scope of the experiment was limited to observing one of the six spans. This may significantly undermine the legitimacy of transferring the conclusions from the research of a single span onto the remaining structures of the same type, even though all spans had been examined and no significant differences had been detected. The potential variability of the results of OMA identification experiments for individual spans could be an indicator of significant differences between individual trusses not detected during the examination.

4. CONCLUSIONS

This article describes the application of operational modal analysis in the dynamic testing of pony truss railway bridge with a relatively small span length. In the described case, the use of OMA enabled the unambiguous identification of several basic modes of vibrations. The scope of identification was sufficient in terms of the verification of the numerical model. The identified dynamic properties of the structure matched quite well with those of the initial FEM model. The results of dynamic tests showed that even

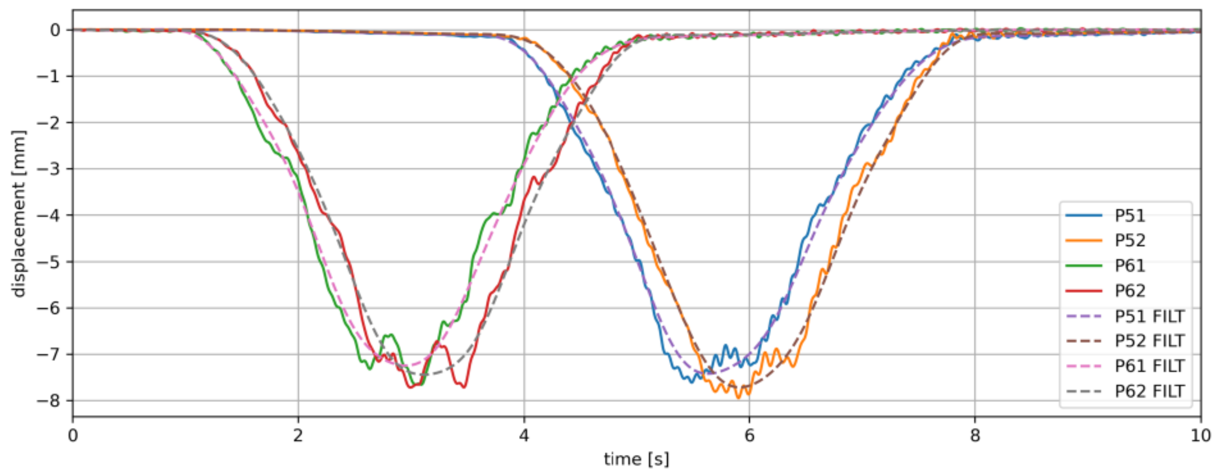

Figure 5.
Vertical displacements induced by a heavy locomotive (L1); span 6 – on the left, span 5 – on the right

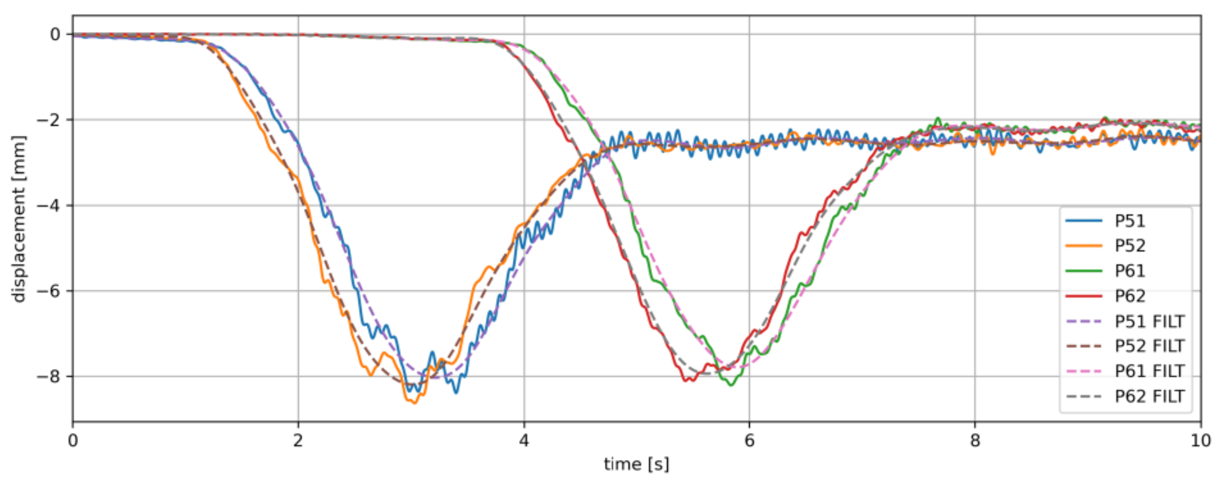

Figure 6.
Vertical displacements induced by a locomotive of a freight train (T3); span 6 – on the left, span 5 – on the right

preliminary FEM model of decent quality can be used, together with other inspection methods, in the process of determining the current load capacity of the structure.

The available environmental excitations were insufficient to effectively excite higher modes of free vibration, preventing their identification. However, the scope of the identification was sufficient and useful results were obtained without closing the bridge traffic and without renting expensive test rolling stock.

The tests under normal traffic do not allow carrying out the estimation of the dynamic amplification factor (DAF), understood as the ratio of displacements caused by the rail vehicle passage with a number of higher speeds to displacements caused by the same rolling stock but moving along the structure at low speed (quasi-static passage). The determination of this parameter is still one of the expected effects of the dynamic test, but its usefulness and legitimacy of referring to it to the standard-specific dynamic coefficient remain debatable. The DAF estimated on the basis of few passages of a arbitrarily chosen one type of rail vehicle lacks statistical significance, necessary to meaningfully describe the dynamic response of the bridge under different load types.

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