

DYNAMIC IDENTIFICATION OF A CABLE-STAYED BRIDGE INDUCED BY RANDOM TRAFFIC FLOW

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Received: 11.05.2025, Revised: 27.05.2025, Accepted: 24.08.2025

Abstract

The paper presents an experimental study of the dynamic response of a cable-stayed road bridge with a steel-UHPC (Ultra High Performance Concrete) composite deck slab, excited by daily random traffic flow along a three-lane highway with high traffic intensity. For this purpose, temporary *in-situ* measurements of vertical vibration accelerations of the box girder were performed along the entire length of the bridge. The measured data were then analyzed using three estimation techniques, i.e., the Peak Picking, Frequency Domain Decomposition, and Random Decrement technique. Their impact on the obtained results was determined. The results obtained using these approaches were compared. The first five modal frequencies and mode shapes of the bridge-vehicle system, as well as the corresponding damping ratios, were analyzed. It was found that the obtained vibration frequency values were variable in the range of 0.1-3.1%. In the case of the damping ratio, the variability in the range of 19-44% was observed, which may indicate a high sensitivity of this parameter to dynamic effects induced by traffic flow. Moreover, it was shown that the extracted modal shapes, corresponding to the natural frequencies of the bridge in the range up to 2.1 Hz, are related to the bending, torsion, and bending-torsion forms. The frequency components of bridge vibrations related to vehicle movement were in the range of 8-16 Hz. The obtained results provided essential data that can be helpful in monitoring the dynamic behavior and technical state of other bridges with a similar structural system.

Keywords: Cable-stayed bridge; In-situ measurement; Dynamic characteristics; Random traffic flow; Modal analysis techniques.

1. INTRODUCTION

The problem with the dynamic identification of long-span bridges in service is that it is often impossible to determine the dynamic characteristics of such bridges without considering the live load. This is usually due to the impossibility of temporarily closing such large bridges that perform important social and economic functions on a national scale. Referring to road bridges, Sohn [1] showed, based on the example of numerous steel bridges, that the accurate dynamic analysis of the technical state of the structure is strictly dependent on the variability of its modal parameters under random road traffic loading since relatively

large load variability during the day can cover small changes in the tested parameters caused by material degradation or local damage of the structure. These conclusions are similar to the findings by Yazhou and Yansong [2]. However, Zhang et al. [3] found that the experimentally obtained natural vibration frequencies on a cable-stayed structure with a steel-concrete composite deck changed by less than 1% within 24 h. Moreover, the structural damping ratio for each measurement showed significant fluctuations depending on the Root Mean Square (RMS) value of the bridge span accelerations, which corresponded to the experiences of Shao et al. [4], obtained on a cable-stayed steel box girder bridge.

According to Macdonald and Daniell [5], who also measured a cable-stayed steel box girder bridge, the influence of vehicle traffic on the dynamic behavior of the span may depend on wind speed, especially for wind speeds above 12 m/s. Nevertheless, the authors state that accurate determination of the natural vibration parameters of the bridge is possible during its normal use. Similar conclusions were reached by Wang et al. [6], who based on the investigation of a prefabricated steel box girder suspension bridge proved that at wind speeds up to 10 m/s, the main factor causing vibrations is the vehicle traffic load. Sheibani and Ghorbani-Tanha [7] proved that in the case of recording data with a length of 30 minutes, the influence of vehicle movement is insignificant and for the first vibration frequency it is less than 2%. Mao et al. [8] observed on the example of a cable-stayed steel bridge that the values of the bridge vibration frequencies on weekdays were higher than on weekends. They decreased particularly rapidly early Saturday morning and began to increase on Monday morning, which means that with the increase in the bridge vibration frequencies, a much more significant effect of vehicle traffic load became visible.

Green et al. [9] proved, after analyzing numerous cable-stayed highway bridges, that heavy goods vehicles induced dynamic forces in the range of 1.5–4.5 Hz, while higher frequency vibrations resulted mainly from the movement of vehicle wheels. Hence, for bridges with the first natural frequency in the range of 1–4.5 Hz, relatively high values of the dynamic amplification factor were assumed. Similar conclusions were drawn by Kohm et al. [10], who demonstrated that the natural vibration shapes on a prestressed concrete bridge show only slight differences in the amplitudes of vibrations excited by the operating load if the excitation frequency is different from the natural frequency of the bridge, which is the case for the vast majority of long cable-stayed bridges.

Dao et al. [11] based on a steel-concrete composite deck bridge and Lei et al. [12] on the example of a cable-stayed steel box girder bridge proved that the higher the speed of vehicles on a bridge, the smaller their influence on the characteristics of the natural bridge vibrations. Gentile and Martinez [13] found that the modal parameters, determined based on bridge vibrations measured on a cable-stayed concrete bridge in normal traffic conditions and those under impulsive excitation by a vertical force in the form of a ballast weighing several tons, were almost identical, and the differences amounted to only 1%.

The results recorded during the measurements by Chul-Young et al. [14], after measuring of 3 different concrete bridges allowed to conclude that the movement of the rolling stock had a negligible effect in the case of large bridge structures, while as regards the stiff structures with short spans, such effect was significant. Gara et al. [15] showed, based on the experiences of a steel-concrete composite deck bridge, that taking into account only the mass of the cars in the modal analysis was insufficient to assess the dynamic parameters of the bridge. Especially in the case of bridge frequencies, comparable to the excitation frequencies excited by heavy vehicles, the interaction between the truck and the bridge should be considered because omitting this factor may lead to an underestimation of the fundamental natural frequency of the entire structure.

The influence of moving vehicles on the dynamic behavior of a bridge can also be estimated using the so-called vehicle-bridge interaction analysis, which was demonstrated by Camara and Ruis-Teran [16]. In this analysis, different roadway profiles were considered depending on the roughness or unevenness of the road surface and the damage to the bridge expansion joint. Then, the frequency and amplitude changes of the bridge span vibrations were estimated.

Summarizing the above literature review, it should be emphasized that existing studies on the variability of the dynamic behavior of existing bridges under operating loads point to various important factors that should also be considered. These include traffic patterns, traffic intensity, and vehicle type, the range of natural bridge vibration frequencies and those forced by road traffic, the study period, wind velocity and direction, air temperature, road surface profile and the associated effects of vehicle-bridge interaction, as well as the span and structural system of the bridge. Considering the last issue, however, there are currently no results of studies on the dynamic behavior of long-span steel bridges with strengthened steel-UHPC deck slab. This seems to be particularly interesting, as this type of bridge structure is currently an innovative solution used on an increasing number of bridge structures and can significantly influence the dynamic behavior of the bridge.

This paper presents a study of the variability of modal parameters representing the dynamic behavior of a long-span cable-stayed bridge under random road traffic conditions. The specific structural system of the bridge consisted of a steel box girder connected to a composite steel-UHPC deck slab. The basis for the analyses was in-situ measurements of vertical vibration

accelerations on both sides of the load-bearing box girder along the entire length of the bridge. Such a measurement set-up allowed for determining bending, torsion, and bending-torsion forms of natural vibration modes of the bridge deck. Three estimation techniques were applied for dynamic identification, i.e., the Peak Picking (PP), Frequency Domain Decomposition (FDD), and Random Decrement (RDM) methods. The influence of the applied approaches on the resulting values of the primary five natural vibration frequencies of the bridge was determined. The following dynamic parameters were studied: (i) maximum vertical accelerations and standard deviations of vibration accelerations of the bridge induced by random traffic flow, (ii) modal character-

istics, i.e., natural frequencies, corresponding natural vibration modes, and structural damping ratios. A similar scope of research was already carried out on the examined bridge in 2016, and the results of the analyses were presented in the paper [17]. However, in 2020, the reconstruction of the bridge was completed, consisting of replacing the flexible bituminous road surface with a rigid UHPC plate bonded to the steel bridge deck slab. By assumption, this reconstruction leads to a change in the stiffness and weight of the bridge spans and, thus, changes the modal characteristics of the bridge. This study allows for assessing the change in the analyzed modal parameters of the bridge with a modified rigid deck plate under the influence of random traffic flow.



Figure 1.
The Rheinbrücke Maxau bridge

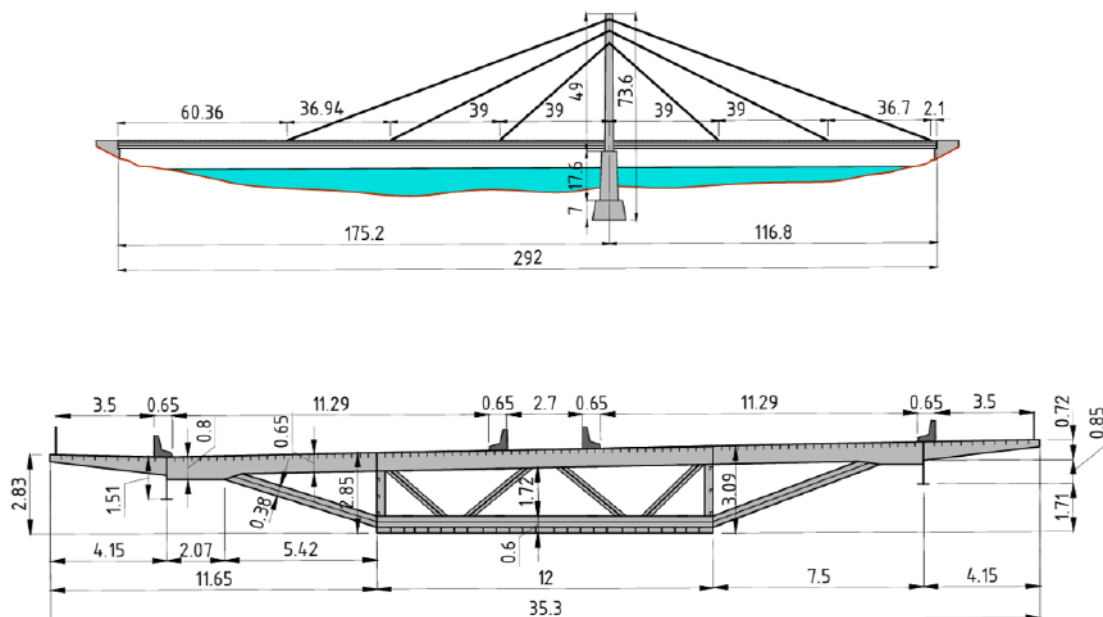


Figure 2.
Basic dimensions of the Rheinbrücke Maxau bridge

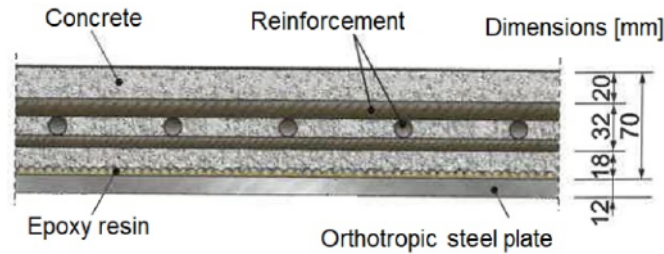


Figure 3.
Schematic cross-section of the bridge deck slab [18]

2. DESCRIPTION OF THE BRIDGE

The analyzed structure is a steel cable-stayed bridge located between Karlsruhe-Maxau and Wörth am Rhein (Germany) on the border of the Rhineland-Platinat and Baden-Württemberg states (Figure 1). Crossing the Rhine River, this bridge is located on the six-lane A10 highway and is one of the most important transport facilities, enabling transit to France.

The bridge structure consists of two spans with lengths of 175.2 m and 116.8 m, reaching a total bridge length of 292 m. The total width of the bridge deck slab is 35.3 m. The operational part consists of two roadways, each with three traffic lanes, with a total width of 11.29 m, and pedestrian and bicycle paths on both outer sides of the deck slab, 3.5 m wide. The basic dimensions of the structure are shown in Figure 2.

The bridge structure is designed as a steel box girder supporting a composite deck slab consisting of an orthotropic steel slab bonded with bonding fasteners and epoxy resin to a 7 cm thick UHPC concrete layer. The cross-section of the bridge slab is shown in Figure 3.

3. EXPERIMENTAL SET-UP

The field measurements were carried out on June 7, 2022, between 9:00 and 16:15. The air temperature ranged from 16 to 21°C, with a relative air humidity from 57% to 82%. During the measurements, there was complete cloud cover, so the impact of sunlight on the object was also excluded this time. The average wind speed during the measurement period was below 5.5 m/s.

The dynamic response of the bridge deck was measured using accelerometers. The research work on the bridge presented in this publication, including measurements of vibrations of its structural elements and their analysis, was carried out in its entirety only by the co-authors of this publication. The measurement set-up consisted of four high-sensitivity, low-noise, single-axis accelerometers type PCB 3711E112G, manufactured using Micro-Electro-Mechanical System (MEMS) technology, connected by low-noise cables with a 4-channel dynamic signal analyzer type DT9837A. The vibration accelerations were recorded in such a way that the fixed sensor was placed at the reference point of girder "A" near the mid-span of the longer bridge span. The three remaining sensors were placed successively at each of the selected measurement points, i.e. from point 1' to

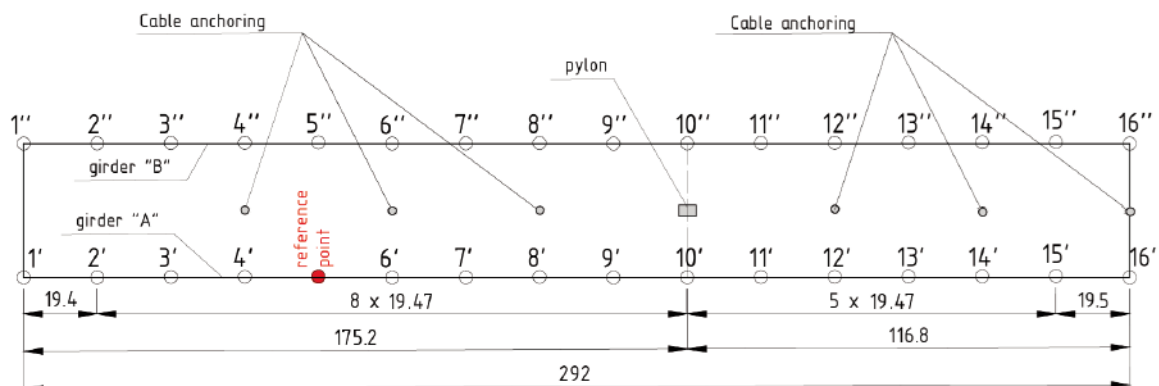


Figure 4.
Location of the measurement points on the bridge girders during the tests

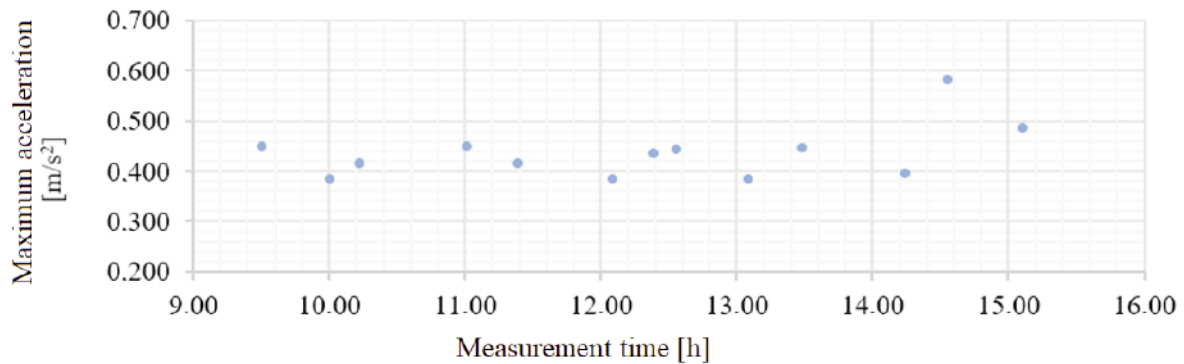


Figure 5.
Variability of 10-min averaged values of maximum vibration accelerations recorded at the reference point

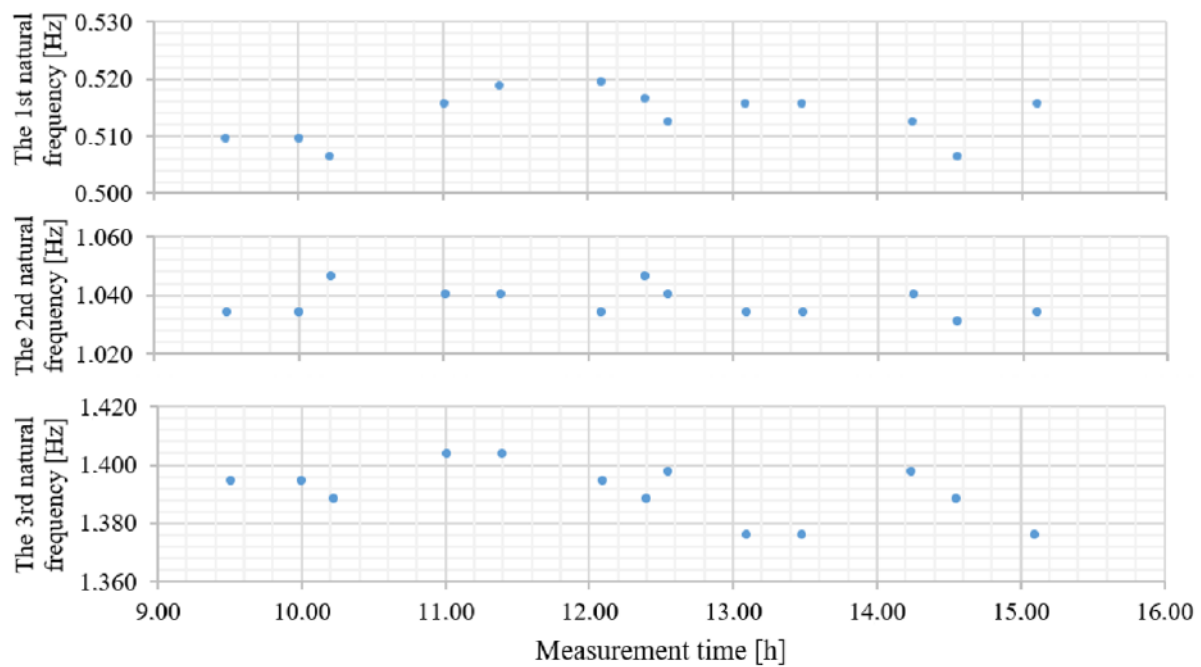


Figure 6.
Variability of the 10-min averaged values of the first three natural frequencies of the bridge-vehicle system recorded at the reference point

point 10' along girder "A", and from point 1" to point 10" along girder "B". These points were located mainly at a distance of 19.47 m, which corresponds to the length of one repeatable structural segment of the bridge span. The reference point was selected in the place where the largest number of dominant frequencies in the dynamic response spectrum was initially identified. Such a sensor configuration allowed for determining the natural vibration mode shapes. A draft of the measurement points distribution on the bridge girders is shown in Figure 4.

During the experimental measurements, the bridge vibrations were excited by continuous random vehicle traffic flow. At each measuring point of the bridge

spans, vibration accelerations were recorded with a sampling frequency of 200 Hz during a measurement period ongoing at least 10 min.

4. EFFECT OF RANDOM TRAFFIC FLOW ON THE BRIDGE BEHAVIOR DURING THE RESEARCH PERIOD

During the tests, there was relatively stable and free traffic, with a travel speed close to the permitted, and in particular, there were no traffic jams, slowdowns, or situations without vehicles on the bridge or even with single vehicles. This was due to the very good capacity of the bridge and simultaneously

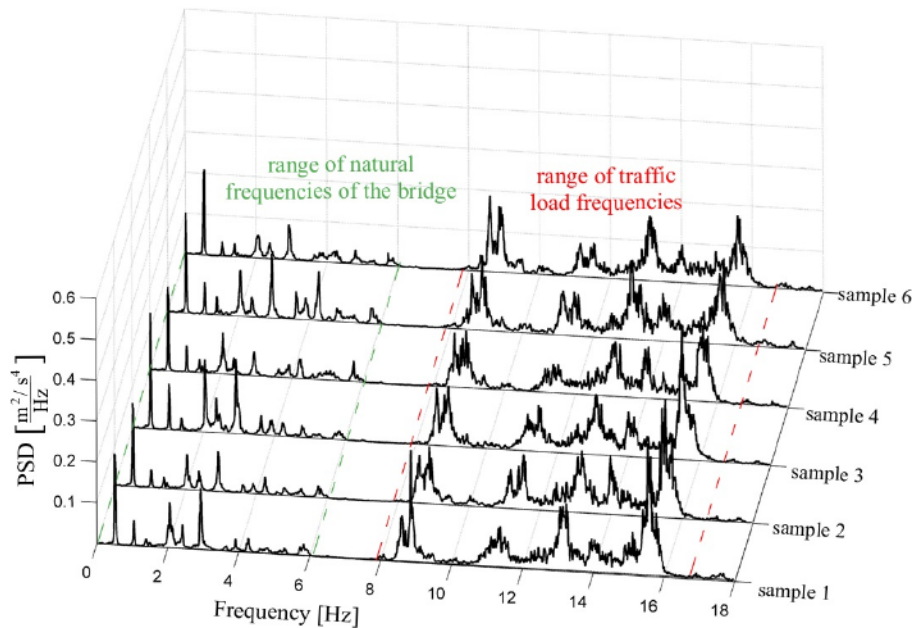


Figure 7.
10 min-averaged PSDs of daily traffic-induced vertical accelerations of the bridge

intensive road traffic on the six-lane transit highway near the German-French international border. Therefore, the bridge vibrations occurred continuously, with similar intensity. The permitted speed of vehicles on the bridge was 80 km/h. Hence, the passage of a single vehicle over the 292 m-long bridge takes about 13.2 s on average. To verify the speed of vehicles and control the traffic flow, the traffic intensity was periodically monitored, and the number and type of passing vehicles were recorded using a digital camera at selected periods of the day. The average number of vehicles simultaneously moving on the bridge was 3-4 trucks and 15 passenger cars. To control traffic conditions, the vehicle speeds were experimentally determined by recording their travel time from the moment they drove over the expansion joint before the bridge to the moment they crossed the expansion joint after the bridge. The travel time of these vehicles ranged from 11 s to 15 s, with an average value of about 13 s. On this basis, a 10-min averaging period of the analyzed vibration parameters was considered sufficient to consider the influence of an adequate number of vehicles passing over the bridge and simultaneously limit the influence of road traffic randomness.

To estimate the maximum variability of the bridge vibration accelerations during the measurement period, data recorded at the reference point of the longer span between 9:27 and 15:05 from thirteen 10-min

measurement cycles were analyzed using the PP technique. The change in vibration accelerations and the values of the first three natural frequencies of the bridge-vehicle system, resulting from the random nature of road traffic on the bridge, are shown in Figures 5 and 6.

As shown in Figures 5 and 6, there is no clear relationship between the natural frequencies of the bridge and the maximum values of vibration accelerations. It was found that during the measurement period, the natural frequencies of the bridge showed a variability of less than 3.5%. Such a small range of variability of this parameter indicates a relatively small impact of traffic intensity on its values throughout the measurement period.

Figure 7 presents the Power Spectral Density (PSD) function of the bridge vibration accelerations, recorded in the vertical direction at the reference point due to daily road traffic flow in 10-min intervals at different times.

As can be seen, the vibration frequency characteristics have very similar patterns with resonance peaks corresponding to the natural frequencies of the bridge-vehicle system in the range of up to about 6 Hz. On the other hand, the vibration frequencies in the range of about 8 to 16 Hz change slightly. This range corresponds to the frequencies of the vibration excitation source caused by passing vehicles [9]. Hence, it can be concluded that during the 10-min

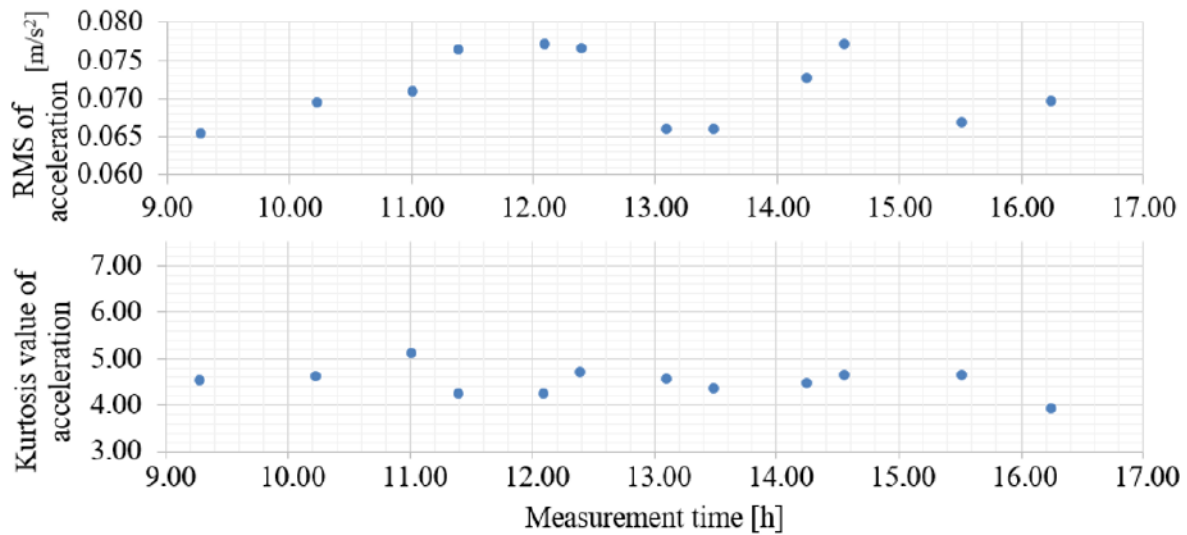


Figure 8.
10-min averaged RMS levels and kurtosis values of bridge accelerations recorded at the reference point

measurement period, the vehicle traffic showed slightly variable dynamic excitation energy, and the frequency characteristics of the bridge showed repeatability.

4. STATIONARITY CLASSIFICATION OF THE RECORDED STOCHASTIC PROCESS

To classify the stationarity of the collected stochastic process, representing the recorded vibrations of the bridge within a 10-min averaging period, a criterion based on the kurtosis value k , proposed by some researchers, e.g. Guo [19], Kim et al. [20], Sharma et al. [21] and Tao et al. [22], was used. The kurtosis value is defined as the normalized fourth-order central moment of a discrete random signal lasting in a certain fixed time interval. Therefore, the kurtosis is a measure of the concentration of N values of the analyzed stochastic process, the realization of which is described by the values x_j , around the mean value $\bar{x}$, and can be determined according to the formula:

$$k = \frac{N \sum_{j=1}^N (x_j - \bar{x})^4}{\left[\sum_{j=1}^N (x_j - \bar{x})^2 \right]^2} . \quad (1)$$

If, due to the non-stationary nature of the analyzed signal representing the dynamic response of the structure, this signal contains peak values of vibration amplitudes significantly deviating from the mean value, the kurtosis value increases quickly with the increasing number of these peak values and their more significant deviation from the mean

value. According to the experience of other researchers in [19,20], the kurtosis value exceeding the number of 10 classifies the analyzed stochastic process as non-stationary. Below this value, the process can be treated as a process with features similar to a stationary process. The kurtosis of a random signal, defined in equation (1), which characterizes the probability density function with a Gaussian distribution, is equal to 3.

In our study, the kurtosis and RMS values were determined for each 10-min measurement cycle of vibration accelerations representing the response of the analyzed bridge induced by vehicle traffic recorded at the reference point. The calculation results for one-day cycles are presented in Figure 8. As can be seen, the kurtosis values ranged from 3.9 to 5.1, with an average value of 4.5. Hence, it was assumed that the recorded bridge vibrations induced by vehicles passing the bridge could be treated as a stochastic quasi-stationary process in the analyzed measurement period.

5. MODAL PARAMETER IDENTIFICATION OF THE BRIDGE-VEHICLE SYSTEM

The computational part was carried out using selected Operational Modal Analysis (OMA) techniques, which enabled the identification of modal parameters of the studied structure based on its measured dynamic response excited by an unknown operational load. Currently, there are many

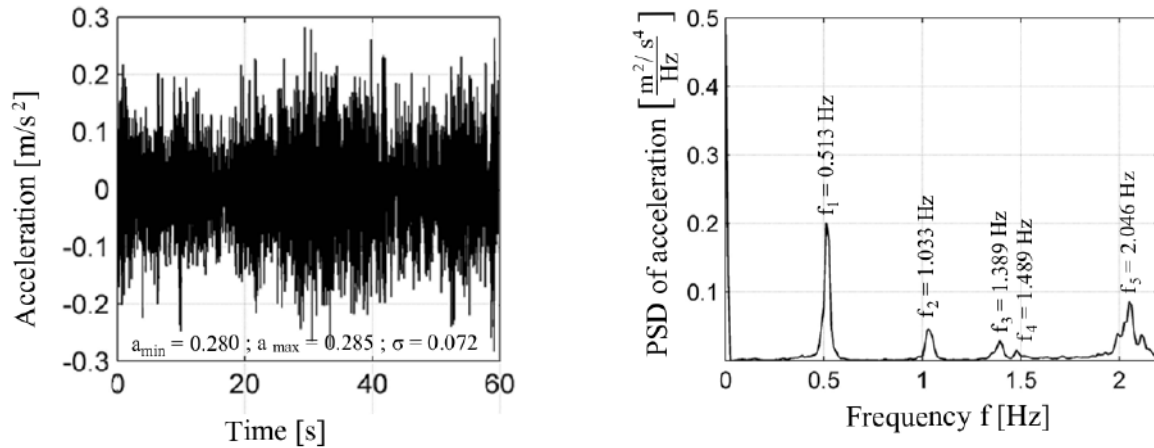


Figure 9.

Vertical acceleration time history and their 10-min averaged PSD at the reference point

techniques for such analysis, including Frequency Domain Decomposition (FDD), Enhanced Frequency Domain Decomposition (EFDD), Random Decrement (RDM), Eigensystem Realization Algorithm (ERA), Stochastic Subspace Identification (SSI), Natural Excitation Technique (NExT), Natural Excitation Technique Eigensystem Realization Algorithm (NExT-ERA), or Bayesian OMA (BAYOMA). In this study, FDD and RDM techniques were used because, as reported by Zahid [23], unlike other methods, these are characterized by a relatively simple algorithm and are still eagerly used and developed by other researchers, e.g. Nguyen et al. [24], Sakai et al. [25] and Chen et al. [26]. The selected methods allow for modal analysis of structures with closely spaced vibration modes excited by stochastic quasi-steady-state loading. Such a situation concerns the analyzed bridge.

5.1. Preliminary determination of frequencies using Peak Picking technique

Before the target estimation of modal parameters, the analytical part started with a spectral analysis of the recorded accelerations at the reference point using the PP technique. Figure 9 shows a selected fragment of the recorded vibration accelerations of the bridge induced by vehicle traffic flow at the reference point and their PSD with the dominant bridge vibration frequencies. The forced vibration frequencies of the bridge identified in this way constitute reference values for verifying the accuracy of the results obtained with the other two estimation methods due to certain non-stationarity features of the analyzed measurement signal. It should be emphasized, however, that the PP technique allowed for direct analysis of forced bridge vibrations, while the FDD and RDM methods, by their specific algorithms, lead to the analysis of a modified signal representing the natural vibrations of the structure as a result of the

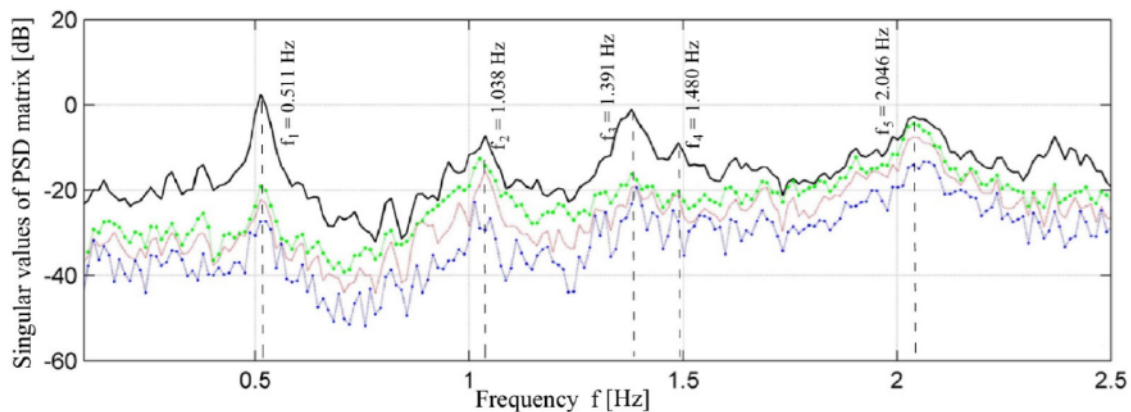


Figure 10.

Charts of singular values of decomposed PSD matrices of traffic-induced bridge vibrations recorded at the reference point and points No. 3', 4', and 3''

Table 1.
Statistical data on modal frequencies of the bridge-vehicle system determined by the FDD method

Mode No.	Mean modal frequency f_{FDD} [Hz]	Range of modal frequencies $f_{min} - f_{max}$ [Hz]	Dominant [Hz]	Standard deviation σ [Hz]	Max. relative variation $\frac{f_{max}-f_{min}}{f_{FDD}}$ [%]	Variability factor $\frac{\sigma}{f_{FDD}}$ [%]
1	0.511	0.507-0.517	0.507	0.004	2.11	0.8048
2	1.038	1.032-1.047	1.035	0.013	1.46	1.2733
3	1.391	1.376-1.404	1.389	0.017	1.98	1.2172
4	1.480	1.471-1.489	1.489	0.046	1.24	3.0779
5	2.046	2.023-2.069	2.023	0.022	2.24	1.0929

reduction of the dynamic effect of the operation load. Hence, slight differences in the results obtained with the applied methods should be expected. Nevertheless, each of these methods analyzed the bridge vibrations, taking into account the influence of the mass of the vehicles passing over the bridge at the same time. Therefore, the presented analysis results in each case concerned the bridge-vehicle system.

5.2. Frequencies and mode shapes identification by Frequency Domain Decomposition technique

The frequencies and mode shapes of the bridge-vehicle system vibrations were determined using the FDD technique. Each of the obtained singular values set was analyzed graphically in the form of independent graphs, corresponding to the number of measurement signals recorded simultaneously at three different measurement points and the reference point. Each collected and analyzed 10-min measurement cycle contained 120,000 acceleration data, which allowed obtaining PSD functions and, thus, singular values at a frequency resolution of 0.00167 Hz. Example charts of singular values, determined based on vertical vibration recorded at four points simulta-

neously, i.e., the reference point and points No. 3', 4', and 3", are shown in Figure 10.

Using the singular value charts for all the analyzed measurement cycles, the natural frequencies of the bridge-vehicle system f_{FDD} in the vertical direction were referred and summarized in Table 1.

The obtained results are closely similar to those obtained using the PP method, which indicates the good accuracy of the FDD method in determining bridge vibration frequencies. Each sequent natural frequency corresponds to the next natural vibration mode shape. Figure 11 shows five normalized natural vibration mode shapes in the vertical direction of the deck slab of the bridge-vehicle system. Modes No. 1, 3, and 5 were related to the bending type of mode. Mode No. 4 resulted from the bridge span torsional movement, while mode No. 2 was the effect of the bending-torsional movement.

5.3. Damping variability identification using Random Decrement method

One of the most important modal parameters characterizing the ability of a complex structural system to dissipate vibration energy is the structural

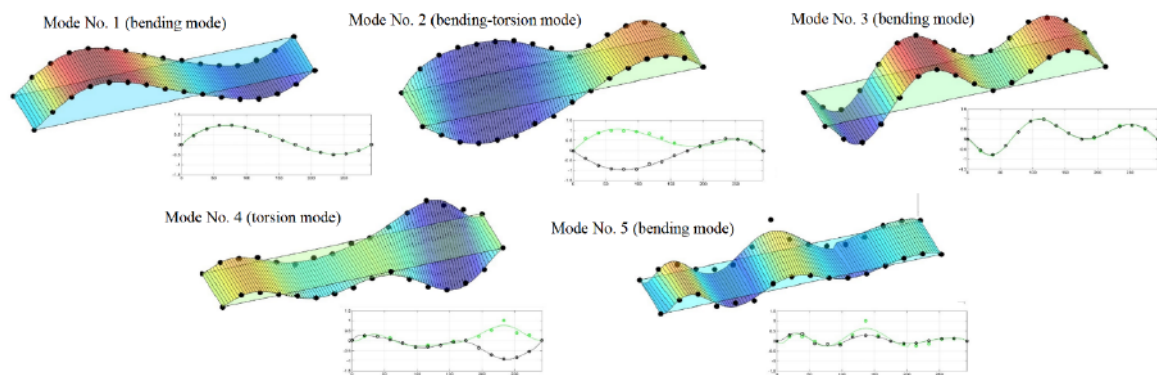


Figure 11.
Normalized mode shapes, their theoretical approximation, and corresponding modal frequencies identified by the FDD technique

Table 2.
Natural frequencies of the bridge-vehicles system determined by the RDM method and statistical data of the modal damping ratios

Mode No.	Mean modal frequency f_{RDM} [Hz]	Mean damping ratio $\bar{\xi}$ [%]	Range of damping ratio $\xi_{min}-\xi_{max}$ [%]	Standard deviation Σ	Max. relative variation $\frac{\xi_{max}-\xi_{min}}{\bar{\xi}}$ [%]	Variability factor $\frac{\sigma}{\bar{\xi}}$ [%]
1	0.514	2.35	1.84-2.69	0.32	36.17	13.62
2	1.032	1.62	1.44-1.94	0.17	30.78	10.49
3	1.377	1.42	1.29-1.55	0.10	18.35	7.04
4	1.494	3.61	2.90-4.13	0.42	34.12	11.63
5	2.046	1.68	1.25-1.98	0.32	43.45	19.13

damping ratio. The exact determination of this parameter is very difficult because it cannot be described by strict mathematical or physical relationships. However, there are empirical equations that allow for an approximate determination of the damping ratio, which were formulated based on the analysis of experimental test results conducted by various researchers, e.g. Bachmann et al. [27], Hwang et al. [28], Kim et al. [29] and Ni et al. [30]. In this study, the RDM method was used to experimentally estimate the vibration damping of the bridge based on the recorded forced vibrations. This technique allows for the transformation of structural random vibrations, constantly excited by the dynamic load, into vibrations representing free vibrations of this structure at a given modal frequency, which are defined as a random decrement signature (RD signature).

Based on the authors' experience, e.g. [31] and [32], it can be stated that the RDM technique is more accurate than the EFDD method in determining the structural damping ratios because the measurement signal is analyzed in the time domain based on a relatively simple calculation algorithm. In turn, the EFDD method is a frequency domain technique and requires spectral transformations of the recorded time history of the structural response. This is associated with certain uncertainties depending on, among others, the frequency resolution of the spectral functions and the possibility of the adverse effect of the spectral leakage phenomenon on the obtained results. Therefore, in this study, the damping ratios were determined exclusively by the RDM method.

To apply the RDM method, which requires the analysis of the measurement signal representing the response of the bridge-vehicle system with one degree of freedom, it is necessary to perform the bandpass filtration process of the recorded vibrations and to extract the vibration components associated with one selected modal frequency of the structure, which corresponds to the tested modal vibration damping ratio. For the analyzed signal representing the bridge slab accelerations, a type I Chebyshev fil-

ter with a ripple ratio of 1 dB was used. After a thorough analysis of the bridge response spectral structure, the following filtration passbands were adopted: for mode No. 1 – 0.45-0.60 Hz, for mode No. 2 – 0.95-1.10 Hz, for mode No. 3 – 1.30-1.42 Hz, for mode No. 4 – 1.45-1.55 Hz, and for mode No. 5 – 1.95-2.10 Hz. Data recorded during six measurement cycles, each lasting 60 min, were analyzed. Based on the RDM algorithm, RD signatures corresponding to the subsequent natural frequencies of the bridge-vehicle system were obtained. These functions were approximated by the Root Mean Square, and the obtained results are presented in Table 2.

For a perfectly stationary input signal, the resulting RD signature should be exactly proportional to the free vibration decay of the damped single-degree-of-freedom system, and the linear regression of its extreme values as a function of time should follow a sloped straight line. However, in most practical cases, the accuracy of the RD signature is limited primarily due to the incomplete fulfillment of the RDM assumptions regarding the analysis of a fully stationary random process. Therefore, to ensure the accuracy of the damping ratio estimation, in this study, a linear regression analysis of the extreme values of the RD signatures obtained from vibration measurements of the tested bridge was performed. This limited the analysis of the resulting RD signatures to time ranges in which this function precisely matches the free vibration response. Thus, for each modal frequency, the longest duration of the analyzed RD modal signature range was determined individually, ranging from 7 to 20 s. In this case, increasing the length of the RD signature time window did not affect the accuracy of the modal analysis results. To control the stability of the RD signature within the analyzed ranges, a stability ratio was introduced, expressed as follows:

$$\eta = \sum_{j=1}^{\hat{M}} \frac{|\hat{x}_j - A_j|}{\hat{M}} \quad (2)$$

where $\hat{x}_j$ is the j -th extreme value, while $\hat{M}$ is the total number of all extreme values of the RD signature within the analyzed range, and A_j is the j -th value of the linear regression of the RD signature, calculated from the formula:

$$A_j = 2 \cdot \ln \left(\frac{x_0}{|\hat{x}_j|} \right) \quad (3)$$

where x_0 is the initial value of the RD signature, which for the normalized function is equal to 1.

When the accuracy of the RD signature is high, the value of the stability ratio η is very low, i.e., close to 0, and vice versa. Therefore, due to the limited accuracy of the obtained RD signatures, the estimation of the damping ratios was limited to selected ranges of these functions, which were characterized by high stability confirmed by relatively low values of the η ratio. Based on the calibration of this ratio discussed in the paper [17], its limit value of 0.08 was adopted as the acceptable threshold value. Figure 12 presents normalized charts of selected RD signatures and linear regressions of extreme values of these functions, along with the marked analyzed range of RD signa-

tures, and the resulting bridge vibration damping ratio values for one selected 60-min measurement cycle.

Table 3 summarizes the results and differences between the average modal frequencies of the bridge-vehicle system obtained by the PP, FDD, and RDM techniques. This table shows that the differences in the frequencies of natural and forced vibrations obtained depending on the applied technique did not exceed 1%. Therefore, the results obtained using different methods can be considered similar.

6. CONCLUSIONS

This study presents the analysis of the vibration accelerations of the cable-stayed steel bridge with a composite steel-UHPC deck, recorded at 32 measurement points of its spans. Based on the recorded measurement data, the analysis of the frequencies and modal vibration shapes of the bridge-vehicle system, as well as the vibration damping ratio, were performed using three estimation methods.

Based on the kurtosis criterion, the analyzed bridge vibrations induced by random traffic flow were classified as a stochastic quasi-stationary process.

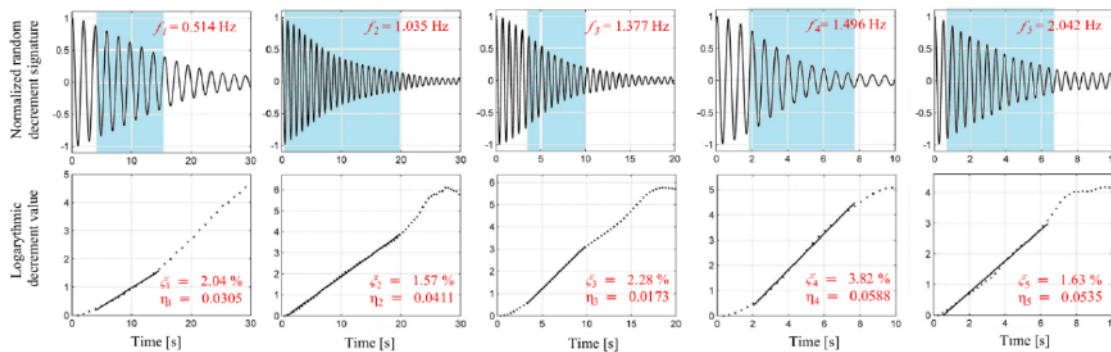


Figure 12. Normalized RD signatures of vertical bridge accelerations collected during a 60-min measurement cycle (first row) and linear regression on the extremes of these signatures (second row)

Table 3. Comparison of the average natural (f_{FDD} , f_{RDM}) and forced (f_{PP}) vibration frequency values of the bridge-vehicle system determined using different techniques

Mode No.	Mean vibration frequency f_{PP} [Hz]	Mean modal frequency f_{FDD} [Hz]	Mean modal frequency f_{RDM} [Hz]	Relative variation $\frac{f_{FDD}-f_{PP}}{f_{PP}} \cdot 100$ [%]	Relative variation $\frac{f_{RDM}-f_{PP}}{f_{PP}} \cdot 100$ [%]
1	0.513	0.511	0.514	-0.39	0.19
2	1.033	1.038	1.032	0.48	-0.10
3	1.389	1.391	1.377	0.14	-0.86
4	1.489	1.480	1.494	-0.60	0.34
5	2.046	2.046	2.046	0.00	0.00

Five modal shapes and corresponding modal vibration frequencies of the bridge in the range of up to about 2.1 Hz were distinguished. This was possible due to the clear separation of the frequency components related to the dynamic load, which were in the range of 8-16 Hz, and the modal natural frequency components of the bridge, which were identified in the range up to approximately 6 Hz.

Based on the analysis of 13 samples of recorded 10-min bridge vibration accelerations, it was found that the variability of the first five natural frequencies ranged from 0.8% to slightly over 3%, which indicates a small effect of road traffic on this parameter. The average value of the fundamental frequency of the bridge vibrations was 0.511 Hz, with a variability of 0.8%. Both bending (No. 1, 3, and 5), torsional (No. 4), and bending-torsional (No. 2) modes of natural vibrations were identified.

To determine the variability of the vibration damping ratio, six acceleration records of vibrations, each lasting 60 min, were analyzed. For this purpose, the vibration components related to the analyzed vibration frequency were separated from the entire measurement signal using a type I Chebyshev pass-band filter. It was found that the determined values of the vertical vibration damping ratio of the bridge spans showed relatively large variability in the range from about 19% to 44%. In relation to the first mode of natural vibrations, the variability of the damping ratio was about 36%. Such a significant variability of the structural damping parameter indicates its sensitivity to time-varying dynamic effects caused by road traffic.

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