Influence of Railway Train Speed on Span Vibrations

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**Abstract.** This paper considers the peculiarities of calculation of vibration and strength of reinforced concrete railway bridges under moving load with different speeds. The finite element method has been used to discretise the object and the Newmark method has been applied to integrate the system of equations of motion. Due to the symmetry of the problem, a single span girder is considered, which is modelled as a Timoshenko girder with prestressing. The point of contact between the beam and the support using eccentricity and the fixed and movable joints at the points of contact with the support are taken into account. Calculations were carried out for moving forces at velocities of 25 m/s, 40 m/s and 50 m/s with selection of appropriate time steps. The effects of a moving locomotive and a full train on a 23.6 m long span have been studied, considering the vertical vibrations of the bridge. The values of the maximum displacement in both of them are near to each other since the weight of the locomotive weighs more than the rest of the wagons. The deflection of the centre of the girder increases with increase in horizontal velocity of the vertical load. Shear forces and normal stress variation at the centre of the span is also taken into account. The dependency of the speed of the moving train in the vertical velocity of the midpoint of the span is illustrated.

**Keywords:** Railway bridge, span, locomotive, carriage, load, vibrations, stresses

# INTRODUCTION

Moving loads on railway bridges Characteristics of moving loads Moving trains passing through a bridge can cause dynamic behavior of a railway bridge, which has recently become an important issue in structural engineering, particularly in the high-speed rail development age. Because the trains now travel faster there is a very strong amplification of the inertia effect which is exerted on the bridge spans and they experience greater deflections, local stress concentrations and even vibration like phenomena. Therefore, a thorough consideration of the role of speed on the response of bridges is critical towards ensuring structural safety, serviceability and durability of rail infrastructures. To counter this challenge, this paper shall examine the vibrating behavior of a prestressed reinforced concrete bridge span at different speed of trains. Coupled dynamic problem between moving vehicle and a bridge is natural. It is a complicated, time dependent force that depends both on the mass and the speed of the vehicle and on the structural properties of the bridge, the span length, stiffness, damping and boundary conditions. Traditional modeling methods are the moving load and the more complicated moving mass models. The moving load model considers vehicles as some time-varying concentrated forces that do not have inertial interaction. It is only appropriate at low to moderate velocity and where the structural dynamics of the vehicle are not of high priority [1, 2, 3].

But with the higher-speed trains and more ductile bridge spans, the moving mass model would be necessary since it takes into account the dynamic coupling of a structure on the vehicle. Such effects as bouncing, pitching, and wheel-track contact forces that are variable in reaction to the bridge deformation are recorded in this model. Analysis on the two models (Yang et. al. [4, 5]) revealed that, when approaching resonance conditions, the results obtained are significantly different, especially on critical speed intervals when vehicle excitation frequency equaled the natural frequencies of the bridge. Possible resonance, when the dynamic amplification is a resonance between the loading frequency of the vehicle and the natural frequency of the span is also a major hazard in high-speed rail designing of bridges. Such effect may lead to large deflections, higher stress ranges and higher rate of fatigue damage. Kilikevičius et al. [6] stressed that in some cases of structural geometry and trains, it is possible to induce resonant conditions at speeds that are well below the design speeds. Thus, it is important to understand and forecast such dangerous speeds during the diagnostics of bridges and preventive maintenance calculations.

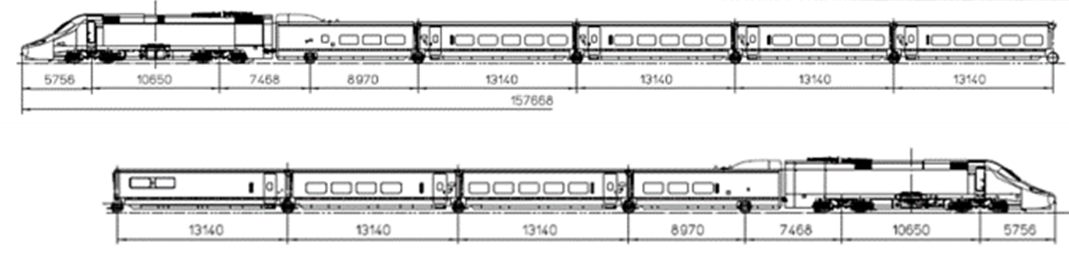
Prestressing of reinforced concrete spans is another aspect that affects responsibility on bridges. When bridge parts are prestressed, it not only increases the amount of loads that the part can carry but also alters its stiffness as well as their dynamic properties. Here, the prestressed Timoshenko model of the beam has been utilized in which both the bending and the shear deformation have been founded. This model has been indicated to offer improved dynamic predictions at short to medium span of the concrete bridges than Euler-Bernoulli beam assumption which neglects the shear deformation and the rotational inertia [7, 8]. Moreover, the dynamic response of railway bridge is considerably affected by the support conditions. Accurate simulation is critical to realistic modeling of the fixed and movable bearing systems, the eccentricity realized by position of the supports and the behavior of the interface between the girder and the abutment. It was shown by El Amrani et al. [9], Dyachenko & Benin [10] that poor assumptions on the support flexibility and damping is likely to make calculated deflections and natural frequencies to vary remarkably especially for prestressed span under repeated dynamic loading.

The example of the transport infrastructure of Uzbekistan can be considered an example where such trains as Talgo 250 they are called Afrosiyob locally finally made their way to the country and now the high-speed train system needs extremely serious structural checks on the bridge resources. These trains operate at speeds up to 250 km/h and exert significant dynamic influence on existing concrete bridge structures. Given the diverse geological and climatic conditions of Central Asia, a context-specific study is needed to evaluate whether the bridges can withstand high-speed train operations without compromising safety and service life. This study focuses on a 23.6-meter-long reinforced concrete bridge span, modeled using a finite element framework and subjected to moving loads representing both individual locomotives and full train sets. The use of the SHARK software, employing the Newmark integration method, allows for time-domain simulation of bridge response under transient dynamic loading. The structural parameters include prestressing forces, eccentric reinforcement placement, and realistic rail-support interaction. Previous investigations by Smirnov [11] and Shermukhamedov et al. [12] on seismic and thermal influences have also informed the boundary condition assumptions and damping parameters in the current model.

Key output variables analyzed in this study include midspan vertical displacement, shear forces, and normal stress distributions at various train velocities (25 m/s, 40 m/s, and 50 m/s). These outputs help determine the sensitivity of the structure to increased speed and evaluate whether performance thresholds, such as allowable deflection limits and stress envelopes, are breached. The study also considers the effect of moving trains on the velocity and acceleration profiles of the bridge span, with implications for fatigue life and passenger comfort. The overall goal of this research is to bridge the knowledge gap between theoretical dynamic models and practical implementation for high-speed rail systems in developing countries. By focusing on realistic structural configurations, loading scenarios, and regional train types, this work provides valuable insights for transportation authorities, engineers, and policymakers involved in railway bridge design, inspection, and retrofitting. Ultimately, this will contribute to enhancing the safety, efficiency, and resilience of the rail transport sector in Uzbekistan and similar contexts globally.

# OBJECTS AND METHODS OF RESEARCH

The adopted physical model for the problem of bridge-vehicle interaction in this study is as follows: the bridge is represented by a Timoshenko girder structure, and the loads transmitted through the wheels of a locomotive or train moving at speed *v* are modelled as concentrated forces. In this case, due to symmetry, 1/2 part of the locomotive mass is taken. The study was carried out on the example of rolling stock with Talgo locomotives (Fig. 1).



**FIGURE 1.** General view of the Talgo locomotives and the complete train

General characteristics of the Talgo 250 ‒ Afrosiyob: maximum speed at service: 250 km/h; weight at full train load: approximately 300 tonnes; full train length: approximately 158 m.

General characteristics of the head carriage (locomotive): length of the head carriage: 20748 mm; weight: 66400 kg (in calculations it is accepted 70000 kg); distance between the bogie rotation axes: 10650 mm; distance between the axes inside the bogies: 2800 mm. Number of cars in the train: 2 head cars (locomotives) + 9 cars, including 1 cafeteria. 1/2 part of the locomotive mass is 35000 kg (as a force of 350000 N), the load from each wheel is 87500 N. 1/2 part of the mass of the wagon is 8000 kg (80000 N), 1/2 part of the mass of the cafeteria is 16000 kg (160000 N).

Numerical calculations were carried out by the SHARK software package created by the authors. The discretisation of the bridge elements was performed by the finite element method, the resulting system of ordinary differential equations was solved by the implicit finite-difference Newmark method. The initial conditions are the solution of the static problem and zero velocities of the nodal points.

In bridge construction practice, the span girder is mounted on a supporting part, this must be correctly modelled in the investigations. Therefore, using the eccentricity with respect to the neutral axis of the girder and fixed or movable hinge connection by the support (footing), the interaction conditions of the girder ends with the footings are taken into account. The prestressing of the span is also taken into account.

The pre-stressing of the working reinforcement in the span is set by pre-deforming and attaching each finite element of the working reinforcement to the neutral axis of the span with eccentricities. In the practice of bridge construction, the span girder is placed on the abutment, this must be correctly modelled in the research. Therefore, using the eccentricity with respect to the neutral axis of the girder, the bottom surface of the span is mounted on a fixed or movable hinge connection of the support (bearing part). In this way, the conditions of interaction between the beam ends and the support parts are taken into account.

In the calculations performed, the influence of the movement of one locomotive (Fig. 2) and a full train (Fig. 3) on the vibrations of the spanning structure was studied.

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|  |
| **FIGURE 2.** Calculation scheme of mobile forces on the flyover during locomotive movement |
|  |
| **FIGURE 3.** Calculation scheme of mobile forces on the spanning structure during train movement |

Break the working reinforcement (reinforcement bundle) into beam-rod finite elements, the coordinates of the nodal points of which differ from the coordinates of the corresponding nodal points of the neutral axis of the span beam only in the vertical coordinate. Let us connect the nodal points of the reinforcement elements with the corresponding nodal points of the neutral axis using eccentricity. According to the similarity of prestressed beam fabrication, we set the reinforcement tension force *P0* and determine its initial deformation *ε0=P0/EF* (*E* is the elastic modulus of the reinforcement; *F* is the total cross-sectional area of the reinforcement).

In the track structure: R65 type rails are used with the weight of 64.72 kg/m, reinforced concrete sleepers with the weight of 270-300 kg, 1840 sleepers are laid per 1 km of track. The mass of ballast layer is 200-300 kg/m, the height of ballast layer is 0.35 m. The top structure of the track is taken as an additional distributed mass on the span.

Numerical calculations have been carried out for a typical 23.6 m long span girder (concrete class B35) with a T-beam cross-section. The mass of one girder is 49.2 t, prestressed reinforcement Bp-II with a specified strain of - 0.00504 was used. The value of elastic modulus of reinforcement elements is taken as E=2∙1011 Pa. The tension force of one bundle is equal to 30240 N.

# THE DISCUSSION OF THE RESULTS

For discretisation, the spanning structure was divided into 236 finite elements considering the performance of each type of finite element, the number of nodal points being 120. Types of finite element characteristics ‒ 5 different types were given through their respective ordinal numbers. The number of eccentricity connections is 120.

The discretisation of the problem in coordinate and time with a moving load while performing calculations at different speeds of motion leads to the appearance of “parasitic oscillations”, which is due to the sudden transition of the moving load to a neighbouring node. If the time step is chosen in such a way that in one step the moving load moves to the neighbouring node, then the “parasitic oscillations” disappear. Calculations were carried out using an implicit Newmark scheme considering viscous damping β=0.0005 s-1 in the beam for moving forces at a velocity of 25 m/s with a time step of 0.008 s; at a velocity of 40 m/s with a time step of 0.005 s; and at a velocity of 50 m/s with a time step of 0.004 s. For concentrated load velocities less than 50 m/s, the load can be approximated by a concentrated force. For load velocities greater than 50 m/s, an oscillator model should be used, since in this case the process is adequately described by the model.

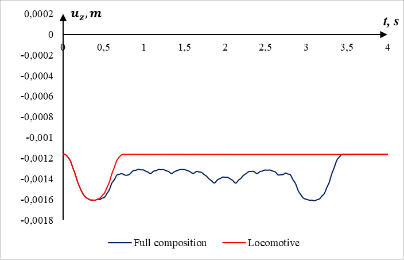
Firstly, the vertical vibrations of the spanning structure when the locomotive itself is moving and then the complete train is considered. Table 1 shows the maximum vertical displacement of the span when the load moves at different speeds.

**TABLE 1.** Maximum deflection of the centre of a 23.6 m long girder for different train speeds

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *Velocity* | | *25 m/s* | *40 m/s* | *50 m/s* |
| Locomotive | Max. deflection (m) | -0.00160 | -0.00161 | -0.00161 |
| Full line-up | -0.00160 | -0.00161 | -0.00161 |

According to the results, the maximum displacement values are close to each other because the weight of the locomotive is more than the individual wagons, as the weight of the locomotive leads to the maximum deflection of the spanning structure. As the horizontal speed of vertical load increases within the considered limits, the maximum deflection of the beam centre increases slightly.

Fig. 4 shows the variation in time of the middle of the spanning structure when locomotive and train are travelling at a speed of 50 m/s. Deflections can be seen when the locomotive, wagons and cafeteria pass. It is those parts of the diagrams corresponding to the head locomotive that in the cases of movement of only one locomotive and the whole train practically coincide.



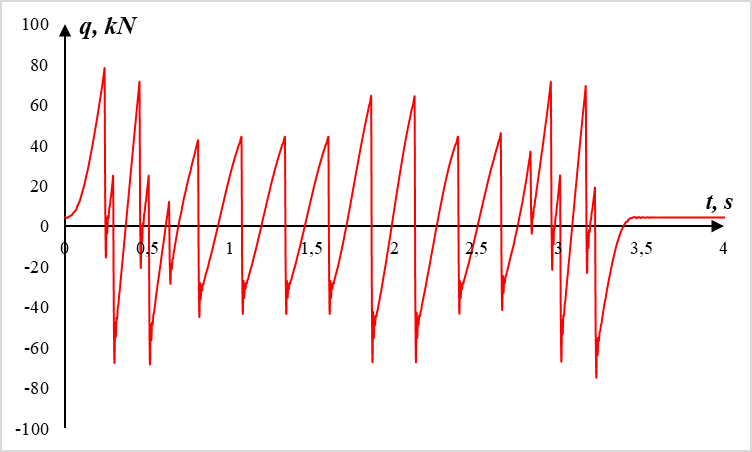
**FIGURE 4.** Vertical vibrations of the centre of the girder at the speed of an individual locomotive and a train of 50 m/s

The change of shear forces in the middle of the spanning structure during the train movement is also considered (Table 2). As the speed of the moving train increases, the shear force in the span slightly decreases. This is due to the shorter contact time between the wheels and the span of the railway bridge.

**TABLE 2.** Maximum shear force in the middle of the span structure

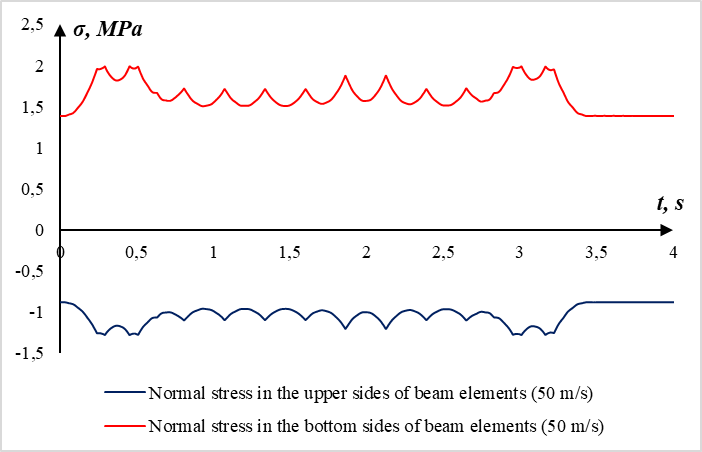
|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *Velocity* | | *25 m/s* | *40 m/s* | *50 m/s* |
| Full line-up | Max. shear force (kN) | 78.125 | 77.994 | 77.990 |

Fig. 5 shows the time variation of the shear force in the middle of the spanning structure when the train moves at a speed of 50 m/s. At the moments of time of passing locomotives, wagon wheels and cafeteria car wheels, the shear force sharply changes its sign and reaches its peak values when locomotives and wagon wheels pass the cafeteria car. Since the locomotives have eight wheels on four axles and the wheels are relatively close in pairs, this is reflected in the graph of the shear force change.



**FIGURE 5.** Shear force of the middle of the beam at a train speed of 50 m/s

Fig. 6 shows the time variation of normal stress in the upper and lower parts of the middle of the spanning structure when the train moves at a speed of 50 m/s. At the moments of time of passing the wheels of locomotives, wheels of wagons and cafeteria the normal stress reaches the maximum values. At the same time, compressive stresses are applied in the upper part of the span and tensile stresses are applied in the lower part. It should be noted that the stresses for B35 concrete are in the elastic zone.



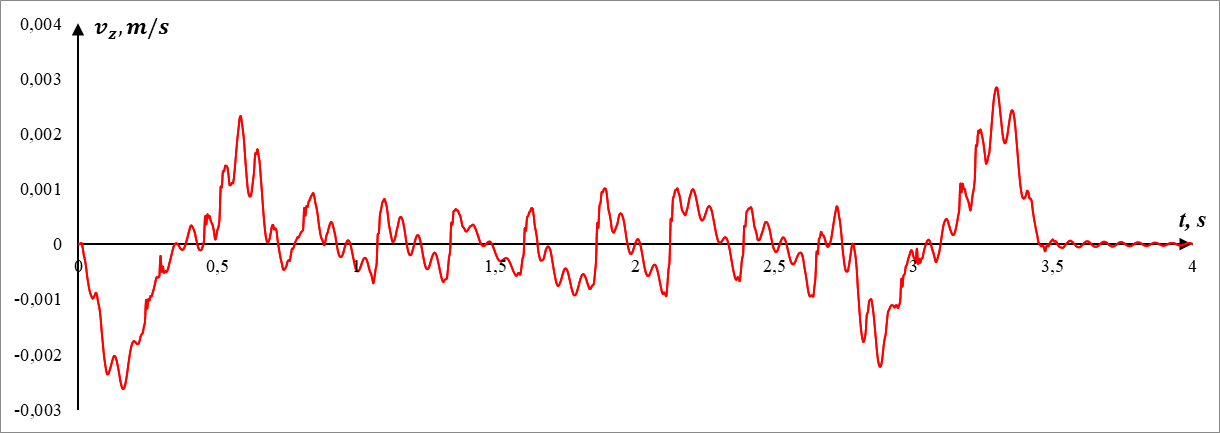
**FIGURE 6.** Variation of normal stress in the top and bottom sides of the middle of the span at a rolling stock speed of 50 m/s

The normal stress in the middle of the girder was studied when travelling at different speeds separately for locomotive and train (Table 3). The values of normal stresses slightly increase with increasing speed of the rolling stock.

**TABLE 3.** Variation of normal stress in the top and bottom sides of span elements

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| *Velocity* | Locomotive | | Full line-up | |
| Top side (MPa) | Bottom side (MPa) | Top side (MPa) | Bottom side (MPa) |
| *25 m/s* | -1.2726 | 1.9881 | -1.2726 | 1.9881 |
| *40 m/s* | -1.2727 | 1.9883 | -1.2728 | 1.9887 |
| *50 m/s* | -1.2727 | 1.9884 | -1.2733 | 1.9896 |

Fig. 7 shows the graph of time variation of the vertical velocity of the middle of the span when the speed of the rolling stock is 50 m/s. Here, as in Fig. 5, the change in the vertical velocity of the centre of the span is related to the moments of passing the wheels of the rolling stock. After passing the train, starting from the time 3.5 s, the flyover performs free oscillations with the first eigenform. The damping of natural vibrations of the spanning structure is related to the viscous damping in the equations of motion.



**FIGURE 7.** Variation of vertical velocity of the middle of the span at the rolling stock speed of 50 m/s

For a train travelling at 50 m/s, the maximum velocity at the midspan was 0.0028 m/s and the acceleration was 0.9507 m/s2.

# CONCLUSIONS

This study presented a comprehensive analysis of how varying train speeds influence the vertical vibrations, shear forces, and stress distribution in a reinforced concrete railway bridge. Using a finite element-based simulation supported by the SHARK software and applying the Newmark method, the research effectively modeled a 23.6 m Timoshenko-type prestressed girder subjected to dynamic loads. The numerical approach incorporated critical structural features, including eccentric connections, prestressed reinforcement, and the practical load characteristics of Talgo 250 locomotives. The adopted model and methodologies provided a realistic and practical basis for evaluating dynamic responses under high-speed rail conditions. The findings clearly indicate that increasing train speed leads to a slight increase in the maximum vertical deflection of the bridge span. However, the difference in deflection between a moving locomotive and a full train remained minimal, primarily because the locomotive's weight dominates the dynamic loading response. Furthermore, the time-history plots of the vertical displacement showed distinct peaks corresponding to the passage of the locomotive and wagons, with the free vibrations diminishing due to incorporated viscous damping.

The study also observed that shear forces in the span reduce marginally with increased speed due to shorter wheel-span contact durations. Nevertheless, despite this reduction, the peak shear values still aligned with the passage of heavy rolling stock components. This indicates that the dynamic influence of load configurations, such as the cafeteria or tightly packed axle loads of the locomotive, plays a significant role in peak shear development. Regarding stress distribution, the bridge experienced compressive stresses on the top and tensile stresses on the bottom surface of the span, with values increasing marginally at higher speeds. Importantly, all observed stresses remained within the elastic limits of the concrete used (B35 grade), validating the structural integrity of the span under the studied dynamic scenarios. Moreover, the non-stationary nature of velocity and acceleration responses further emphasized the need to account for transient behavior in design and analysis. In conclusion, the research confirms that although high-speed train movement introduces increased dynamic effects, the prestressed reinforced concrete girder analyzed remains structurally safe within the observed speed range. These insights are particularly relevant for engineers and transport authorities involved in the development and maintenance of high-speed railway infrastructure. Future studies should consider even higher speeds, three-dimensional modeling, and combined seismic and thermal effects to enhance the robustness of dynamic railway bridge evaluations.

**FUTURE SCOPE**

The findings of the proposed research leave a number of potential research/practical options in relation to the domain of railway bridge construction. When the effect of train speed on the span deflection, shear force and the pattern of stresses was determined, it became apparent that bridge design should be based on dynamic considerations, especially as a country such as Uzbekistan continues to build high-speed rail networks. Investigations on higher speed, i.e. more than 50 m/s, with variable axle car formation and train length should be undertaken, since the modern railway infrastructure is changing. Another important area of future research would be the elaboration of new more advanced structural models which would take into consideration the three dimensional response of the bridge systems under dynamic loading. These are incorporation of the non-linear material behavior consideration, track irregularities as well as bridge track structure coupling. Not only will these gains have positive effects on simulation accuracy, but they will also be utilized in the creation of more resilient bridging designs that will be more able to maintain its functionality under a greater amount of operational stresses and variable environmental conditions. Further, seismic effects combined with moving load dynamics is a crucial field where expansion should be made. In seismically active areas, coupled risk arises to bridges due to the vibrations caused by the passing trains and the earthquakes. Investigating these complicated responses through multi-axial loading and time-history seismic records will enable engineers to give improved estimates of safety factors and establish adaptive design regulations in high-speed corridors in earthquake susceptible regions. There is another crucial implication, which is that of full-scale experimental validation. Although finite element modeling will be profitable, lab-based approaches can assist in improving model-based parameters, substantiating assumptions, and aiding reforms of regulatory codes by examining field data gathered on instrumented live-traffic bridges. Such a testing in the real world is particularly important to discover the fatigue effects, long term structural health, and resonance phenomenon at certain operating speeds. Finally, this study underlines the importance of creating smart digital tools that will enable monitoring bridges and predictive maintenance in real-time. Programmed simulation presentations coupled with sensor feedback responses may assist in continually tracking dynamic response movements alerting the engineers of possible failure threats. They would be valuable instruments in providing long-term security and efficiency of the high-speed railway structural situation, particularly under the influence of higher numbers of climate-related stresses and higher traffic loads.

**ACKNOWLEDGEMENT**

The work was carried out under grant AL-8924063439 of the Agency for Innovative Development of the Ministry of Higher Education, Science and Innovations of the Republic of Uzbekistan.

# REFERENCES

1. Y. B. Yang, J. D. Yau, Z. Yao, and Y. S. Wu, *Vehicle-Bridge Interaction Dynamics: With Applications to High-Speed Railways* (World Scientific, Singapore, 2004). ISBN 981-238-847-8.
2. D. Provornaya and S. Glushkov, *Vehicle-bridge interaction system*, MATEC Web of Conferences **239**, 05004 (2018).
3. A. K. Chopra, *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, 4th ed. (University of California, Berkeley, 2012), 944 p.
4. Y. B. Yang and C. W. Lin, *Vehicle–bridge interaction dynamics and potential applications*, Journal of Sound and Vibration **284**(1–2), 205–226 (2005).
5. Q. S. Yan and C. Cole, *Vertical dynamic interaction of trains and rail steel bridges*, Electronic Journal of Structural Engineering **13**(1), 88–97 (2013).
6. A. Kilikevičius, D. Bačinskas, J. Selech, J. Matijošius, K. Kilikevičienė, D. Vainorius, et al., *The influence of different loads on the footbridge dynamic parameters*, Symmetry **12**(4), 657 (2020).
7. S. Mao, T. Gao, B. Wang, and Q. Chen, *Effect of indenter type on GaN single crystals in nanoindentation from the atomic perspective*, Materials Today Communications **44**, 112115 (2025).
8. S. Sadeghi Eshkevari, T. J. Matarazzo, and S. N. Pakzad, *Simplified vehicle–bridge interaction for medium to long-span bridges subject to random traffic load*, Journal of Civil Structural Health Monitoring **10**(4), 693–707 (2020).
9. A. El Amrani, H. Mataich, and B. El Amrani, *Stability of beam bridges under bridge-vehicle interaction*, WSEAS Transactions on Applied and Theoretical Mechanics (2024).
10. L. K. Dyachenko and A. V. Benin, *An assessment of the dynamic interaction of the rolling stock and the long-span bridges on high-speed railways*, MATEC Web of Conferences **107**, 00014 (2017).
11. V. N. Smirnov, A. V. Lang, and N. A. Labutin, *Determination of dynamic effects of a rolling stock on bridges during high-speed traffic*, Proceedings of Petersburg Transport University **19**(1), 90–96 (2022) (in Russian).
12. U. Shermukhamedov, I. Mirzaev, A. Karimova, and D. Askarova, *Calculation of the stress-strain state of monolithic bridges on the action of real seismic impacts*, E3S Web of Conferences **401**, 05080 (2023).