OPTIMIZATION OF BRIDGE TRANSITION ZONES ON HIGH-SPEED RAILWAYS

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ABSTRACT

The author studies issues of track service life and rail profile stability on bridges and bridge approaches, as well as safety issues on these sections at train speeds of up to 400 km/h. The optimal control theory of dynamic processes in the bridge-track-vehicle system makes it possible to synthesize structures that exert comparatively uniform impact on the sleepers (or other rail supports) in the bridge zone while maintaining the loads on the ballast closest to their design values. Optimal designs ensure a sufficient force of the wheel-rail contact thus substantially improving traffic safety.

Keywords: railway bridge, train-bridge interaction, HSR, traffic safety, track stability.

Background. *Bridge transition zones, where bridges interface with the railway bedding are known to be problem zones. The approach to a bridge, and the bridge egress ramp are different zones in terms of impact conditions. Pic. 1 shows the results of high-accuracy height measurements of the track profile at a zone where the ballastless bridge deck track interfaces with the railway embankment at one of the railway bridges of the Moscow Railway's ring branch. Sleepers numbered with «0» are located on the backwalls of bridge abutments; sleepers marked with positive numbers are located on the approach; those with negative numbers, on egresses from the bridge deck.*

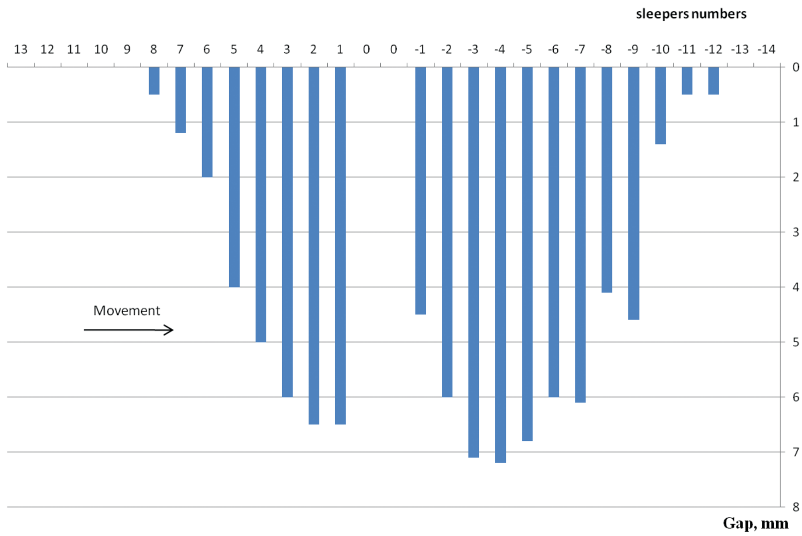
Methods*. The author uses engineering methods, mathematical apparatus. The measurement methodology is described in [1].*

Objectives*. The author's task is to synthesize design of structures that exert comparatively uniform impact on the sleepers (or other rail supports) in the bridge zone while maintaining the loads on the ballast closest to their design values.*

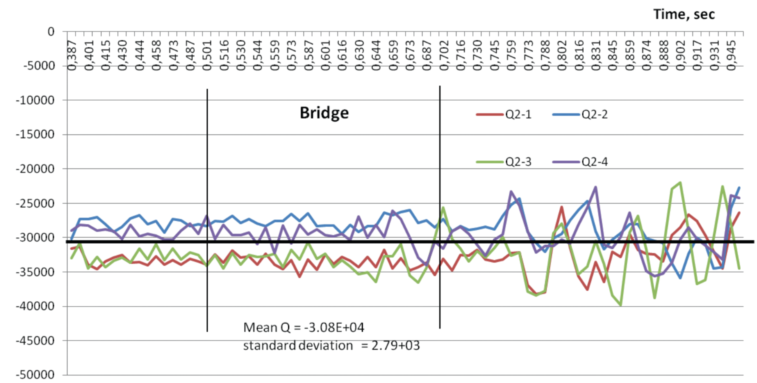
1.

Settlement of the conventional track on the approaches had led to the formation of significant geometric irregularity of the profile and hidden deflections (the levels of sleeper bases). These irregularities and deflections were caused by two factors: the practically settlement-free rail supports on the bridge with ballastless track, and oscillations of vehicles caused by deformations of the bridge superstructure under the load. The operation of the first factor had resulted in a situation when in the absence of loading the rail simply hanged in the air without resting on the ballast over several meters of the approach; there were gaps under the sleepers. The sizes of the gaps are shown in Pic. 1. The wheel of a railcar thus moves on a hidden irregularity that is significantly deeper than it appears.

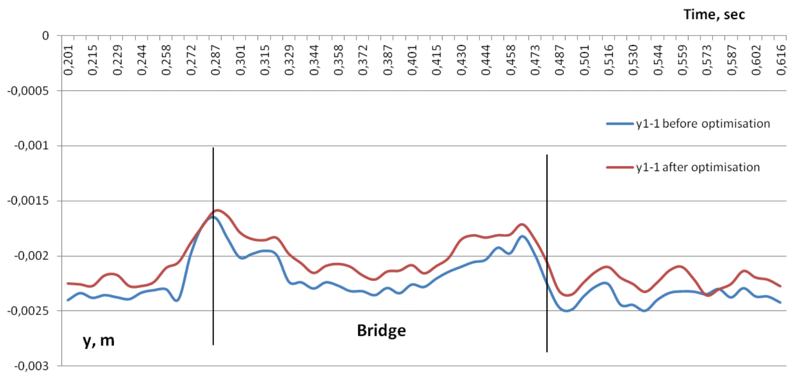
The other factor determines the differences between the irregularities on the approach to the bridge and the egress from the bridge. On the approach, the irregularity has a comparatively short length, and the depths of the gaps grow monotonously as the bridge gets closer, while of the egress the irregularity is longer due to the vehicles' oscillations caused by the oscillations of the bridge superstructure. It is shown in [2] that the variability of the load on the rail sleeper (support) beyond the bridge superstructure has an oscillatory nature as it is caused by the oscillation of the vehicle. We note here that by the time of the survey, the tonnage



Pic. 1. Depths of hidden irregularities in the bridge approach zones.



Pic. 2. Loads applied to the rail support by the EVS-2 train before optimization.



Pic. 3. Oscillations of the first wheel of the first car.

passed after a major track overhaul was a mere 100 million tons gross.

The formation of the bridge approach pit is a result of ballasted track settlement, and the gaps grow in size «monotonously». The behavior of the train after leaving the bridge is completely determined by the oscillations of the bridge superstructure and the design of the bridge deck. The sizes of gaps under the sleepers are patterned according to the oscillations of the vehicle's elements that get excited as they pass through the oscillating bridge superstructure. Therefore, tackling the problem of the interface zone requires consideration of the entire section of the bridge crossing, including the bridge structures in order to develop the design of transition structures before and after the bridge and thus increase the service life of the interface between the bridge and the railway bed.

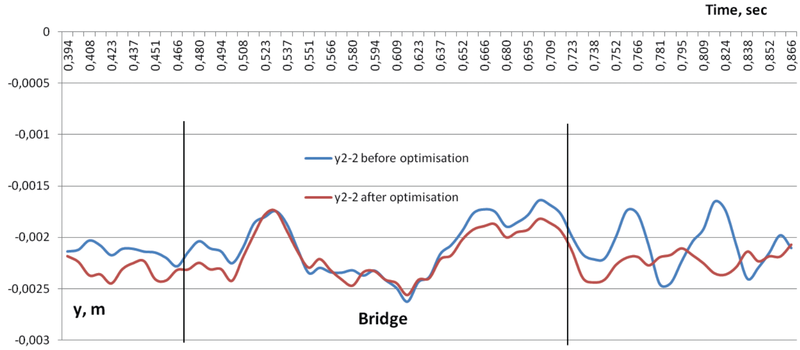
The other important conclusion is that the impact on the rail supports needs to be reduced below the value at which the formation of non-elastic residual deformations begins in the supports. According to [3], such a critical load level on the rail support (base plate) is approximately 31 kN.

For the purpose of developing a set of requirements to the structural elements of the transition zone, consideration of a single-span bridge is sufficient. Such a bridge certainly contains all the necessary elements: the approach to the bridge, the oscillating bridge span structure, and the egress from the bridge. The requirements were developed through the application of the optimal dynamic processes theory in the bridge-track-vehicle system as described in [46]. We note here that out of all the optimality criteria [6], the criterion of minimal sleeper load deviation from the pre-assigned value Q will be the most effective for ensuring track stability and for prevention of residual deformations of the ballast:

$$D = \int_L \int_T (\gamma(x)\delta(x)U(y_r - y_b) - Q)^2 \, dxdt \to \min, \qquad (1)$$

where $U(x, t) = \gamma(x)\,\delta(x)U'(y_r - y_b)$ is the reaction of the sleeper, $\delta(x)$; $\gamma(x)$ are the control functions modeling the longitudinal change in the stiffness of the rail support with the change in the sleeper spacing and the change in the stiffness of the track foundation that depends on both the thickness of the compressible layer (from infinite on an embankment to limited on a bridge span structure or zero on a ballastless bridge deck) and the rigidity of the baseplate in the fastening unit or the pad on the bottom surface of the sleeper; $y_p = y_p(x, t)$ is the function of the rail's vertical displacement; y_b is the same for a bridge span structure; L is the length of the modeled section that includes the bridge and the approaches; T is the time during which the train is present on the modeled section; Q is the mean assigned value of the load on the rail bed immediately under the wheels along the entire length of the transition section and over the entire period of the train passage.

In the integrand (1) we put the standard deviation of the loads U(x, t) from the assigned value Q, which standard deviation needs to be minimized. The control functions $\delta(x) = U3$, $\gamma(x) = U2$ that deliver the minimum to the quality functionality (1), are to be determined in the process of interaction optimization in the bridge-track-vehicle system.



Pic. 4. Oscillations of the second car's second wheel.

Reaching the minimum of the integral criterion for quality would mean a relatively even impact on the rail supports under all wheels of the train, and the value of this impact would be closest to the assigned mean value Q of the load. This approach makes it possible to prevent significant local overloads that lead to track profile issues and accumulation of residual deformations set forth in Pic. 1.

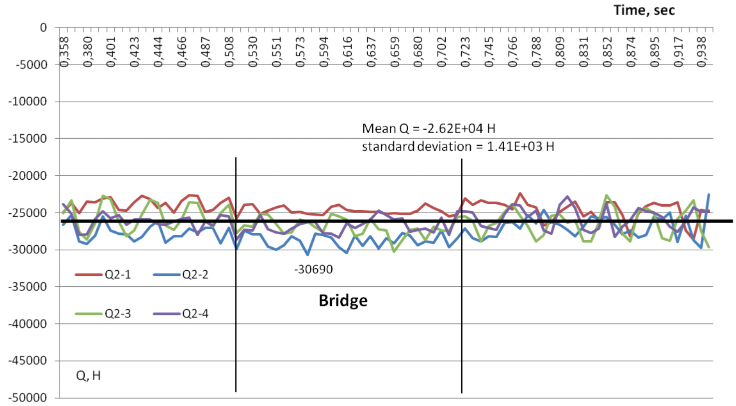
2.

First, let us consider the behavior of the bridge-track-vehicle system in the absence of any special track structures in the approaches, assuming a bridge track with concrete sleepers at sleeper density of 1,840 sleepers/km, with dirty and moist ballast; a concrete 23.6 m bridge span of a box-like section. With synchronous loading of the two tracks, the greatest oscillations of the bridge span and therefore extreme interactions in the system are observed. The amplitude of the bridge span's oscillations, and thus the impact on rail supports outside of the bridge with moist and dirty ballast turned out 7 % higher than with clean and dry ballast. From this point on, we shall consider precisely such conditions.

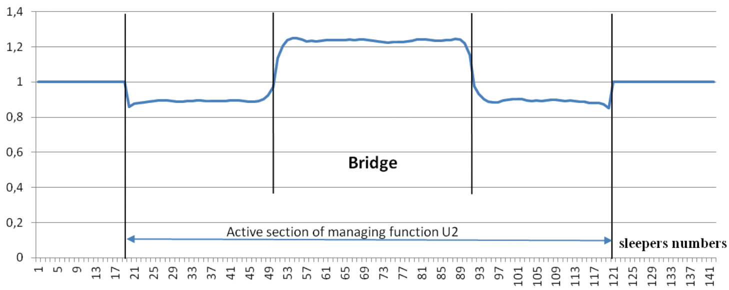
Pic. 2 shows baseplate load diagrams under the wheels of the second car of the EVS-1 train that is passing a bridge with a 23.6 m bridge span at the speed of 400 km/h. Hereinafter, the vertical axis shows the load on the baseplate (H); the horizontal axis, time t (in seconds) for the second wheel of the second car (t = 0 at the entry of this wheel onto the modeled section); the modeled car is moving from the left to the right. In such span structures, steady-state oscillations with an amplitude of 0.918 mm are observed as early as the passage of the second car. This, in turn, is the cause of significant oscillations of the cars and track overloading beyond the bridge structure, while the spike in stiffness at the entry to the bridge remains unnoticed. The load on the rail support reaches 40 kN beyond the bridge, while on the bridge and before the bridge it amounts to approximately 35 kN, with 30.8 kN being the average value for the whole of the bridge zone.

The aforementioned «unnoticed» entry of the second car's wheel on the bridge span, manifested by the absence of a noticeable increase of the load on the first rail support on the bridge, requires a comment.

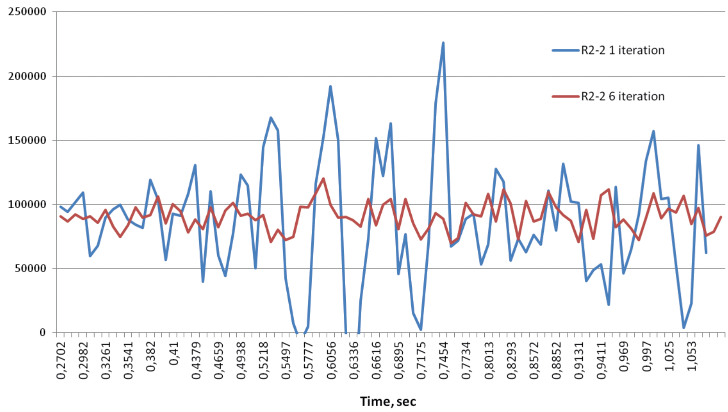
Oscillations of the first car's first wheel (Pic. 3) match the «classic» concept of interactions in the bridge-track-vehicle system in a static setting and relatively low speeds. As can be seen from Pic. 3, before the oscillations of the span structure begin (both before and after optimization), the first wheel reacts to the change in the stiffness of the rail support at the transition from the embankment to the bridge, which is manifested in a rather sharp reduction in the rail deflection under this wheel, and in the excitation of the wheel's oscillations (the wheel «jumps»). It is also seen that the span structure deforms under the first bogie, and the wheel travels on the deformed profile. The optimization smooths out the transition, and the oscillations excited at the entry are noticeably less pronounced. Further on, however, oscillations in the system are largely determined by the oscillations of the span structure rather than the



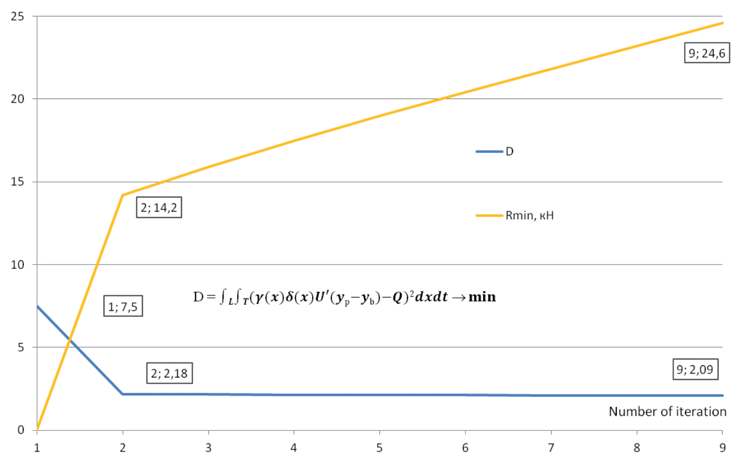
Pic. 5. Loads after optimization.



Pic. 6. U2 = y(x) function delivering the minimum to the quality functional.



Pic. 7. Sample diagram of the vertical force in the contact of the wheel and the rail after optimization.

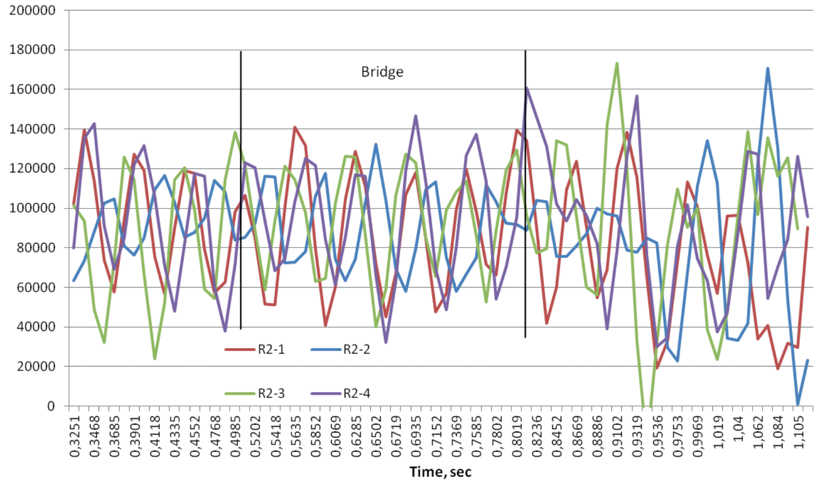


Pic. 8. Evolution of the quality criterion D and of the minimum wheel pressing force in the process of optimization.

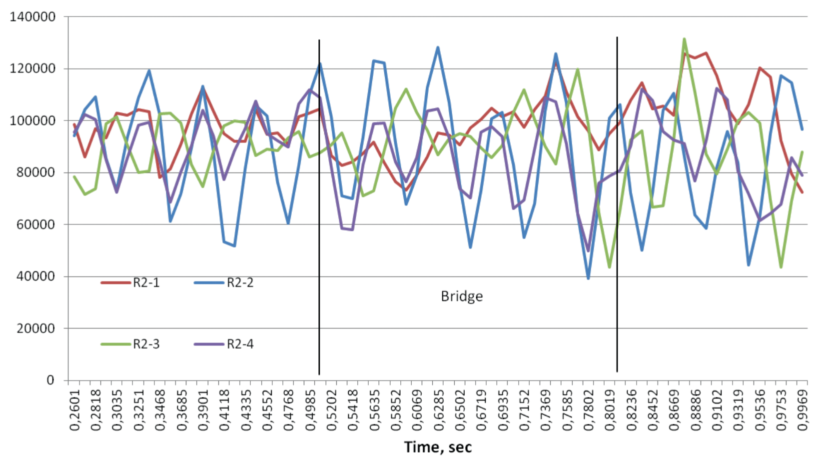
change in the stiffness of the rail support. The motion of the wheel at 0.523 sec in Pic. 4 is caused by the oscillation phase of the span structure and not by the spike in stiffness [8].

As already mentioned, at baseplate loads in excess of 31 kN, residual deformations begin to show up, and optimization aims to preclude them. We note here that the above values of the loads inevitably lead to the emergence of non-elastic (residual) deformations in the ballast, which in turn will become the cause exciting additional oscillations of unsprung masses, track profile deflections, and deterioration of the interaction in the bridge-track-vehicle system, which should not be allowed.

Optimization brings about significant changes to the interaction in the bridge-track-vehicle system



Pic. 9. Wheel-rail contact force before the optimization.



Pic. 10. Wheel-rail contact force after the optimization.

(Pic. 5). The optimal control function U2 that is related to the distribution under the rail of baseplate rigidity (or related to the pads on the bottom surface of the sleeper) and that changes the stiffness of the rail support is shown in Pic. 6. Such stiffness distribution is relatively easy to implement in practice.

It can be seen in Pic. 5 that the mean value of the load on the rail support is significantly (by a factor of 3.6) smaller than the standard deviation. Most importantly, the maximum value of the load is reduced to 30.7 kN, which in theory rules out non-elastic residual deformations forming under the sleeper (the threshold for the emergence of residual deformations in the ballast is 31 kN). It should be noted that the said residual deformation emergence threshold load was established for concrete sleepers on crushed rock ballast. The proposed solution involves the use of elastic pads on the bottom surface of sleepers, which would noticeably raise this threshold.

The application of the special design solution for the bridge deck starts at sleeper No. 20 (Pic. 6). Up to this point, U2 = 1, which corresponds to the conventional track design: the sleeper density of 1,840 sleepers/km, baseplates with a stiffness of 90 kN/mm, dirty and moist ballast. The feature that distinguishes this design from the conventional one is that by sleeper No. 20, the guardrail (a set of guard angles) after the shuttle is placed closest to the running rail (U1 = 0.85), and further change of this position will be suppressed up to sleeper No. 120. Beyond that sleeper, the guard rails start to converge into a shuttle, and baseplates have standard stiffness.

It is extraordinarily important that optimization of the interaction process leads to a substantial increase of the minimal vertical force in the contact between the wheel and the rail: Rmin = 24.55 kN. It is shown in [7, 8] that at a value of this force below 23.814 kN neither the stability of the wheel on the rail nor traffic safety are ensured in any, even infinitely short, period when the vertical projection of the contact force is at its critical value. Notably, separation of the wheel from the rail is registered before the optimization. A sample force diagram in the wheel-rail contact is provided in Pic. 7: separation of the wheel is registered in iteration 1; in iteration 6, the interaction between the wheel and the rail is returned in the safe range.

The optimization process is set forth in Pic. 8. In that Pic. D is the value of the integral quality functional (1) (decreased by 11 orders of magnitude for convenience of showing in a single diagram), and it is this parameter that needs to be minimized; Rmin is the minimum value of the vertical force in the contact of all wheels and the rail.

It can be seen in Pic. 8 that as early as in the second iteration we succeed at significantly reducing the integral quality functional (1) that is the integral by the time of the train's passage through the model section of the sum of standard deviations of the load on the support from the assigned value Q under all the eight wheels of the train. In the second iteration, no separation of the wheel from the track is observed: over the entire period of train passage Rmin is never below 14.2 kN, and by the 9th iteration Rmin increases to the acceptable value 24.6 kN.

At first glance, the solution suggested by the optimal control theory is paradoxical: on approaches, track stiffness should be noticeably reduced, and conversely, it should be increased on the span structure. However, such a solution is explained by the unusually stiff span structures on the Moscow-Kazan HSR. For example, the span structure deflection in the process of oscillations never exceeds 1 mm and its value is close to rail deflection on an embankment. Reduced track stiffness on the approaches provides for a decreased interaction between the wheels and the rail, especially that caused by oscillations caused by various factors, including passage through a span structure. It should be proper to note here that the stiffness of pads on a ballastless track (that determines track stiffness almost completely) stands at 25 kN/mm on Chinese HSRs [9], which is significantly lower than the stiffness of pads used by the RZD whose stiffness averages 90 kN/mm.

Increased track stiffness levels out rail deformations on a «soft» approach and on the span structure itself, so that the wheel «never notices» the span structure.

It is not hard to see that, to achieve the optimal result, comparatively simple design measures would be required: a guard rails (a set of guard angles) positioned closest to the running rail throughout the entire length of the transition; the sleeper density stays unchanged at 1,840 sleepers/km. To control the stiffness of rail supports, pads of various stiffness values are needed, and they are installed in a simple pattern.

Let us take a closer look at the traffic safety aspects. If the vertical force pressing the wheel to the rail is insufficient, the flange of the wheel may roll onto the rail head under the impact of lateral forces, which may result in derailing. At high speeds of movement, such an occurrence will lead to a train wreck with dire consequences, especially in a bridge zone [8] recommends 23.8 kN as the minimum value of the vertical force in the contact between the wheel and the rail. Pics. 9 and 10 show diagrams of the vertical forces in the wheel-rail contact of the second car of the EVS-2 train before and after the optimization. As can be seen, before the optimization not only the force repeatedly drops below the safe level; but detachment of the wheel from the rail also occurs. The optimization smooths out the interaction between the wheel and the rail; this interaction becomes «calmer», with the vertical force never dropping below 40 kN.

Conclusions

In addressing the issue of transition zones in approaches to bridges, the entire bridge transition section needs to be considered, including span structures that have material effect on the oscillations of the cars beyond the bridge.

The applied optimal control theory makes it possible to find quite practical solutions to significantly reduce the average load on rail supports, and the non-uniformity of impacts exerted on the supports. Also, the maximum value of load can be brought to a level below the threshold when residual non-elastic deformations emerge in the ballast. The proposed measures ensure both a long service life of the rail support structures and traffic safety.

REFERENCES

1. Klinov, S. I., Polyakov, V. I. Methodology and Results of Determining Hidden Track Irregularities on Approaches to Artificial Structures [*Metodika i rezul'taty opredeleniya potajnykh nerovnostej puti na podkhodakh k iskusstvennym sooruzheniyam*]. Collection of Inter-University Research Papers, 1984, Iss. 759, MIIT Publishers, pp. 70–76.

2. Polyakov, V. Y. Impact of Rolling Stock on the Track in the Bridge Zone on a HSR [*Vozdejstvie na put' podvizhnogo sostava v zone mosta na VSM*]. Track and Track Infrastructure, 2014, Iss. 10, pp. 24–27.

3. Gasanov, A. I. Modeling the Concrete Sleeper in View of Track Foundation Settlement [*Raschet zhelezobetonnoj shpaly pri uchete osadochnosti osnovaniya*]. Proceedings of MIIT, 1996, Iss. 901, pp. 47–49.

4. Polyakov, V. Yu. The interaction of rolling stock with elements of a bridge at high speed movement. D.Sc. (Tech) thesis. [*Vzaimodeystvie podvizhnogo sostava s elementami mostovogo perehoda pri vysokoskorostnom dvizhenii. Dis... dok. tehn. nauk*]. Moscow, 1994, 395 p.

5. Polyakov, V. Y. Numerical modeling of rolling stock interaction with bridge structures in high-speed traffic [*Chislennoe modelirovanie vzaimodejstviya podvizhnogo sostava s mostovymi konstruktsiyami pri vysokoskorostnom dvizhenii*]. Structural Mechanics and Computation of Structures, 2016, Iss. 2, pp. 54–60.

6. Polyakov, V. Y. Synthesis of optimal span structures for a high-speed railway [*Sintez optimal'nykh proletnykh stroenij dlya vysokoskorostnoj magistrali*]. Structural Mechanics and Computation of Structures, 2016, Iss. 3, pp. 35–42.

7. Verigo, M. F., Kogan, A. Ya. Interaction of track and rolling stock [*Vzaimodeystvie puti i podvizhnogo sostava*]. Moscow, Transport publ., 1986, 559 p.

8. Polyakov, V. Y. Safety on a Bridge at High Speeds [*Bezopasnost' pri vysokikh skorostyakh na mostu*]. *World of Transport and Transportation*, Vol. 12, 2014, Iss. 6, pp. 182–188.

9. Wanming, Zhai *et al.* Experimental investigation into ground vibrations induced by very high speed trains on a non-ballasted track. *Soil Dynamics and Earthquake Engineering*, Vol. 72, May 2015, pp. 24–36.

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