

Suitability assessment of the traditional unified railway bridge girders for high-speed traffic

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Abstract. The article deals with the parameters of high-speed trains effect on the unified simple-span girder reinforced concrete bridge superstructures on conventional railways using a simplified methodology of determining maximum vertical accelerations and live load dynamic factors. It is noted that the dynamic additions to the stress-strain behavior components (bending moments, transverse force and vertical sags) differ from each other. The presented methodology of dynamic factors determination helps to make a dynamic calculation and to identify general forces in simple-span girder superstructures without numerical simulation and straightforward dynamic analysis, which greatly reduces labor intensity of the design works. Based on the dynamic calculation results, a conclusion is made that the unified simple-span girder reinforced concrete bridge superstructures can be applied on high-speed railways, and the scope of their application is specified.

1 Introduction

An approach to establishing dynamic factors determining the properties of bridge and train interaction, which is prescribed by the majority of current bridge design standards, seems to have a number of internal inconsistencies from the methodological and physical viewpoint and is not supported by a proper theoretical foundation. The long-term empirically-based practice of bridge design accounting for dynamic effects has a very limited scope of application and may not be used for the purpose of bridge design for train speeds of more than 200 km/h [1]. However, dynamic processes accompanying train movement along a bridge determine the bridge structure, material, cross-sections, and rigidity (both of elements and the whole structure) to a very large extent [3-5].

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2 Definition of the task

Today, bridges for high-speed railways are designed with support of calculations and numerical simulations [6-11]. It should be noted that, as a rule, the dynamic problems analysis is carried out by software complexes based on the finite-element method (FE), which requires much time. However, to evaluate the nature of the high-speed rolling stock dynamic effect one can use simplified methodologies [12-14], which are also used in the present research.

The most widespread type of bridge superstructures on Russian railways are the simple-span girder reinforced concrete structures used on railway lines with train speed up to 200 km/h. The superstructures applied can be divided into plate and ribbed reinforced concrete (RC) structures. Typical cross-sections of ribbed RC superstructures are shown in Fig. 1. Thus, in order to specify their scope of application under high-speed conditions a special dynamic analysis was performed.

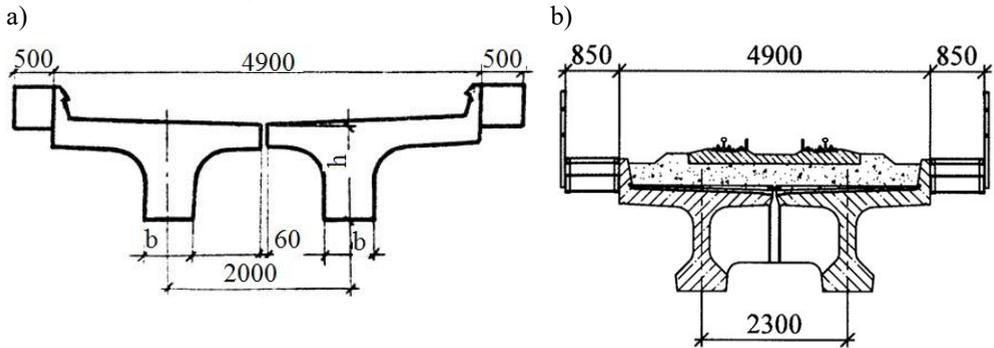


Fig. 1. Cross-section of a superstructure: a) with nonprestressed reinforcement; b) with prestressed reinforcement.

In the course of calculations, mass varieties of the superstructure parameters were considered (the minimum and the maximum values), which allowed to determine both maximum accelerations and minimum values of resonant speed. Fig. 2 shows the ratio between the value of the first natural bending frequency of various superstructures and the span length as well as the limit values of natural frequency according to European Norms EN [15] and Russian Standard [16].

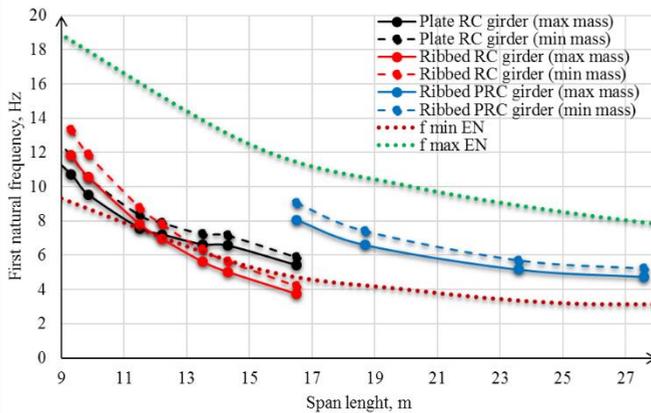


Fig. 2. A diagram showing the ratio between the value of the first natural bending frequency of various superstructures and the span length.

3 Method of solution

High-speed trains with length X_n (a distance between the front and the rear axles) may be represented as a consequence consisting of n loads P_i , whose abscissa is designated as x_i (a distance between the load P_i and the front axle). The excitation function is studied using Fourier series as follows [2, 13]

$$P(t) = \frac{2}{X_n} \int S_0(\lambda) \left(\cos \frac{2\pi Vt}{\lambda} + \sin \frac{2\pi Vt}{\lambda} \right) d\lambda \quad (1)$$

where λ is the length of an excitation wave

$$S_0(\lambda) = \sqrt{\left(\sum_{i=1}^n P_i \cos \frac{2\pi x_i}{\lambda} \right)^2 + \left(\sum_{i=1}^n P_i \sin \frac{2\pi x_i}{\lambda} \right)^2} \quad (2)$$

where $S_0(\lambda)$ is the train excitation that can be considered as the train dynamic performance. Once the excitation function is presented in this way, it is possible to simplify the dynamic calculation procedure for simple-span girders. Using the calculated parameters one can obtain the maximum vertical accelerations in the middle of the span with a train moving at speed V . This methodology can only be used for resonant behavior in the first bending mode.

The maximum value of acceleration a_{\max} is calculated by multiplying the following components:

$$a_{\max} = C \cdot A(L/\lambda) \cdot G(\lambda) \quad (3)$$

where C is the parameter determined by the superstructure weight

$$C = \frac{4}{m\pi} \quad (4)$$

$A(L/\lambda)$ only depends on the span length L and is a dynamic influence line calculated in the following way

$$A(L/\lambda) = \left| \frac{\cos \frac{\pi L}{\lambda}}{\left(\frac{2L}{\lambda} \right)^2 - 1} \right| \quad (5)$$

$G(\lambda)$ depends on the so called train dynamic performance and the superstructure damping coefficient ζ . This parameter causes the range of influence of the train, and it is measured in kN/m.

Knowing the spectral response of the effect one can determine the acceleration in the middle of the span for simple-span girders under critical train speeds

$$|G(\lambda) = Ma|\xi \left[\frac{1}{\zeta X_i} S_{0,i}(\lambda) \left(1 - \exp \left(-2\pi\zeta \frac{X_i}{\lambda} \right) \right) \right] \quad (6)$$

where X_i is the train length including i axles.

It should be noted that using A1–A10 high-speed train models (see Table 1) [15-16] is suitable, if the type of high-speed rolling stock has not been defined by the start of design works. So, the values of the envelope curve based on the maximum spectral response values of A1–A10 train models for the whole range of excitation wave length (λ) exceed the respective values for actual high-speed trains, both Russian and foreign (see Fig. 3) [16]. This results in a conclusion that the A1–A10 train models are sufficient for dynamic calculation, and there is no need in using all 22 models prescribed by the Russian standards.

Moreover, we have considered the effect of *Sapsan (Velaro RUS)* high-speed trains that are operated on Russian railways today.

Table 1. HSLM A1–A10 high-speed train models parameters.

Train Option	Intermediate Cars			Load Magnitude per Axle
	Number of Cars ¹⁾	Car Length	Bogie Wheelbase	
	<i>N</i>	<i>l</i>	<i>d</i>	<i>P</i>
	items	m	m	kN
A1	18	18	2	170
A2	17	19	3.5	200
A3	16	20	2	180
A4	15	21	3	190
A5	14	22	2	170
A6	13	23	2	180
A7	13	24	2	190
A8	12	25	2.5	190
A9	11	26	2	210
A10	11	27	2	210

¹⁾ The total number of cars is plus 4 (2 locomotives + 2 service cars)

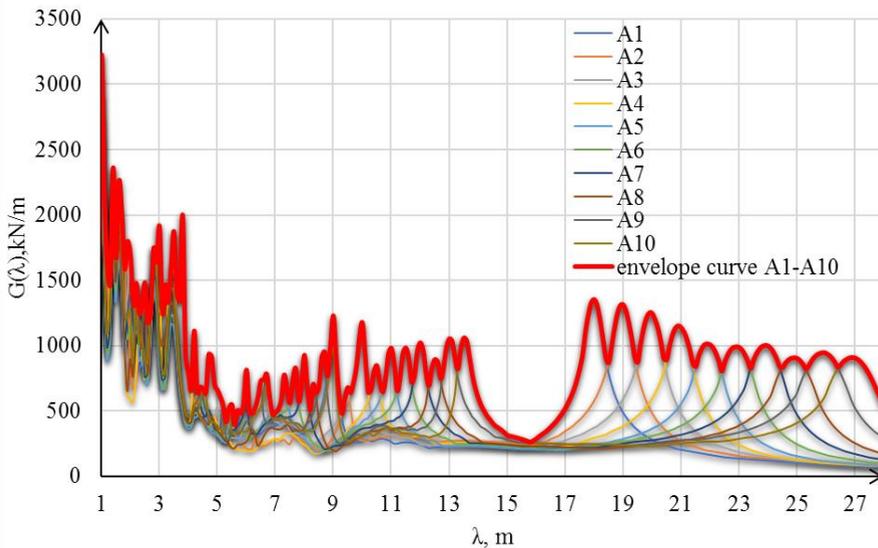


Fig. 3. Spectral performance of A1–A10 high-speed train models and envelope curve.

As it is known, the increase in the dynamic effect of the rolling stock load is due to significant forces of inertia that determine the magnitude of the dynamic factor, i.e. the increase in the stress-strain behavior components (bending moments, transverse force, movements). Thus, one should study the distribution of the forces of inertia along the superstructure, which can be calculated by multiplying the mass per unit of length and the acceleration of the part of the girder.

Accelerations appearing under resonance oscillations in the middle of the span reach their maximum at the moment when the train leaves the span. The distribution of accelerations can be precisely approximated as a sinusoid half wave

$$a(x) = a_{\max} \sin\left(\frac{\pi x}{L}\right) \quad (7)$$

where $a(x)$ stands for vertical accelerations at resonance oscillations in the girder section having x coordinate; a_{\max} is the maximum acceleration at resonance oscillations in the middle of the girder.

It should also be noted that, once the train leaves the superstructure, the distribution of accelerations changes its sign but retains the value. This is truly essential for determination of minimum structural forces needed for fatigue check.

Thus, in case of single-valued influence line of the structure stress-strain behavior factor (bending moment in the middle of the girder, transverse force in the support cross-section, etc.) the maximum dynamic addition will be reached when the superstructure is fully loaded with the force of inertia per unit of length characterized by the following intensity

$$k(x) = m \cdot a_{\max} \sin\left(\frac{\pi x}{L}\right) \quad (8)$$

where $k(x)$ is the force of inertia at resonance oscillations in the girder section having x coordinate; m is the superstructure mass per unit of length accounting for the weight of the bridge deck, t/m .

Imposing inertia load on the triangular single-valued influence line (see Fig. 2) one can obtain the dynamic addition to the force

$$\Delta N = m \cdot a_{\max} \left(\int_0^s \frac{c}{s} x_1 \sin \frac{\pi}{L} x_1 dx_1 + \int_0^{L-s} \frac{c}{L-s} x_2 \sin \frac{\pi}{L} x_2 dx_2 \right) \quad (9)$$

Now we can apply the partial integration method and introduce a substitution

$$\sin \frac{\pi}{l} (l - s) = \sin \frac{\pi}{l} s, \cos \frac{\pi}{l} (l - s) = -\cos \frac{\pi}{l} s \quad (10)$$

Once the integration and trigonometric transformations are performed, we finally get

$$\Delta N = \frac{m \cdot a_{\max} \cdot c \cdot L^3}{s \cdot \pi^2 \cdot (L - s)} \sin\left(\frac{\pi}{L} s\right) \quad (11)$$

Thus, the maximum bending moment in the middle of the span equals to

$$\Delta M_{0.5} = \frac{m \cdot a_{\max} \cdot L^2}{\pi^2} \quad (12)$$

Similarly, by imposing load on the triangular influence line we obtained an equation for the transverse force in the support cross-section

$$\Delta Q_0 = \frac{m \cdot a_{\max} \cdot L}{\pi} \quad (13)$$

To determine the dynamic sag in the superstructure the inertia load should be imposed on the curved influence line; after that the dynamic addition to the sag can be found

$$\Delta \delta = 2m \cdot a_{\max} \int_0^{L/2} \left(\frac{x^3}{12EJ} - \frac{L^2 x}{16EJ} \right) \sin \frac{\pi}{L} x dx \quad (14)$$

Once the integration and trigonometric transformations are performed, we finally get

$$\Delta \delta = \frac{m \cdot a_{\max} L^4}{\pi^4 EJ} \quad (15)$$

To determine the dynamic factors $(1 + \mu_1)$ associated with the high-speed train live load and with various stress-strain behavior factors the following equations can be used.

For strain analysis

$$1 + \mu_1 = 1 + 0.788 \frac{m \cdot a_{max}}{v_{0.5}} \tag{16}$$

For bending moment analysis

$$1 + \mu_1 = 1 + 0.811 \frac{m \cdot a_{max}}{v_{0.5}} \tag{17}$$

For transverse force analysis

$$1 + \mu_1 = 1 + 0.637 \frac{m \cdot a_{max}}{v_0} \tag{18}$$

where a_{max} stands for the maximum vertical accelerations at superstructure resonance oscillations in the middle of the span; m is the superstructure mass per unit of length accounting for the weight of the bridge deck, t/m; $v_{0.5}$ and v_0 stand for the equivalent uniform load caused by high-speed trains, t/m, at $\alpha = 0.5$ and 0.0 respectively.

4 Results of the dynamic analysis

The performed dynamic analysis resulted in a number of ratios between the maximum vertical accelerations of various superstructure types and the A1–A10 train models effect at the minimum and the maximum possible weight of a structure (Fig. 4–6).

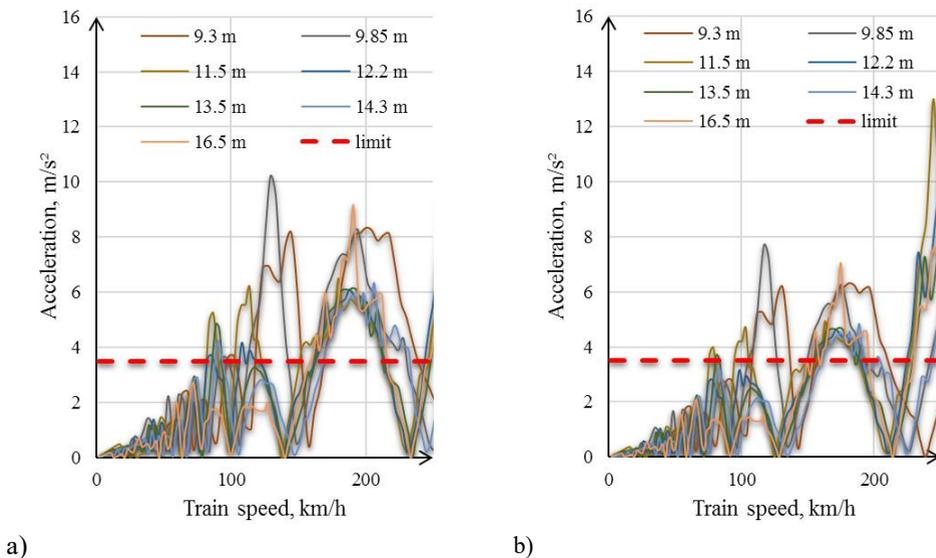


Fig. 4. A diagram showing the ratio between the maximum vertical accelerations of plate RC superstructures and the A1–A10 trains effect: a) with minimum weight; b) with maximum weight.

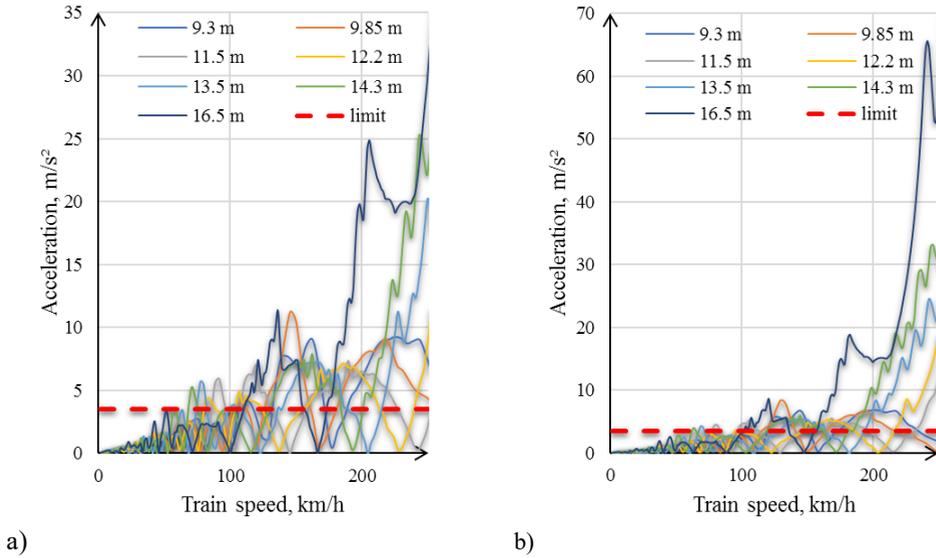


Fig. 5. A diagram showing the ratio between the maximum vertical accelerations of ribbed RC superstructures and the A1–A10 trains effect: a) with minimum weight; b) with maximum weight.

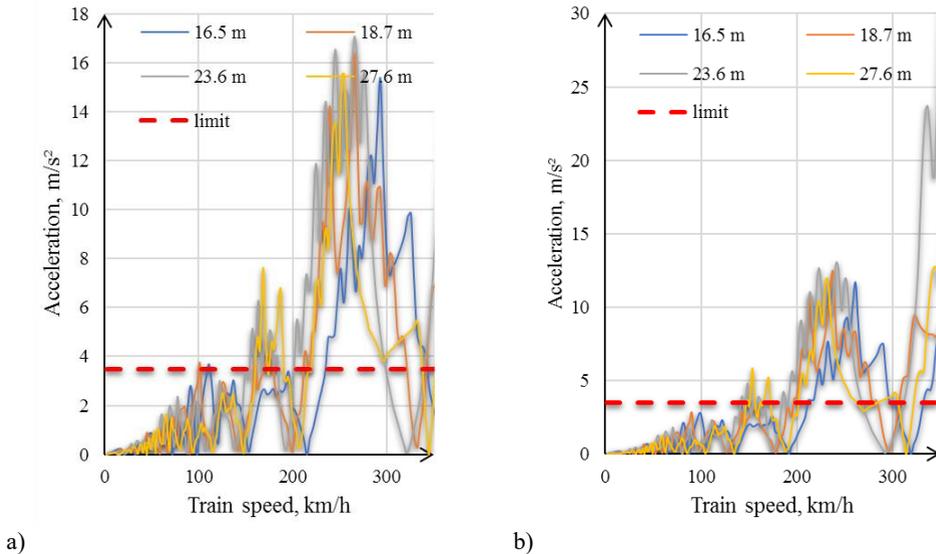


Fig. 6. A diagram showing the ratio between the maximum vertical accelerations of ribbed prestressed RC superstructures and the A1–A10 trains effect: a) with minimum weight; b) with maximum weight.

The diagrams also show the limit value of admissible vertical accelerations of superstructures at the bridge deck level, which is regulated by European and Russian standards [15, 16]. In case of ballast bridge deck to ensure safety and reliability of the track superstructure the maximum vertical accelerations are limited to 3.5 m/s^2 . Similar calculations for bridge superstructures were carried out regarding the effect of *Sapsan* (*Velaro RUS*) high-speed trains that are operated on Russian railways today (Fig. 7–9).

It should be noted that the effect of high-speed train model has a significantly broader spectrum, so, in order to assess the suitability of bridge superstructures for high-speed

traffic with the type of rolling stock already defined it is viable to make a special computational check regarding the exact type of the high-speed train.

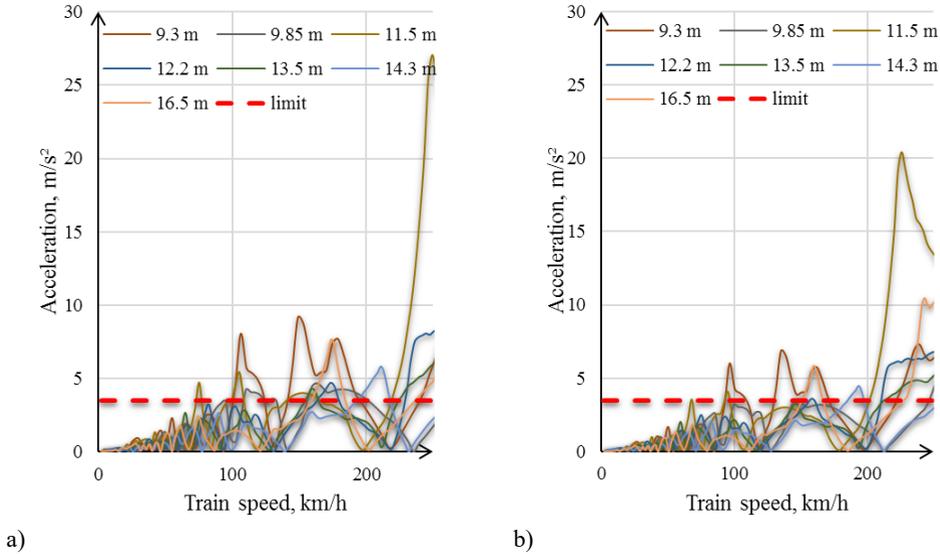


Fig. 7. A diagram showing the ratio between the maximum vertical accelerations of plate RC superstructures and the *Sapsan (Velaro RUS)* trains effect: a) with minimum weight; b) with maximum weight.

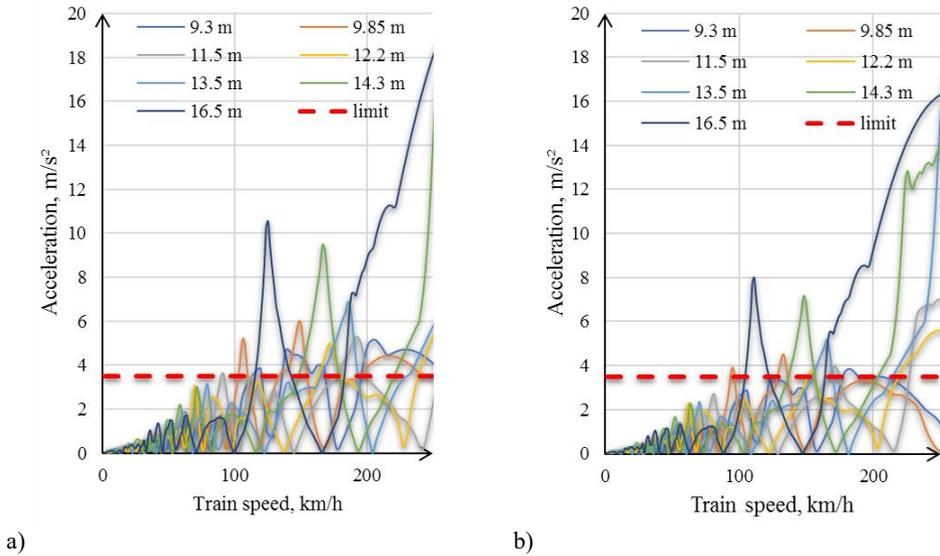


Fig. 8. A diagram showing the ratio between the maximum vertical accelerations of ribbed RC superstructures and the *Sapsan (Velaro RUS)* trains effect: a) with minimum weight; b) with maximum weight.

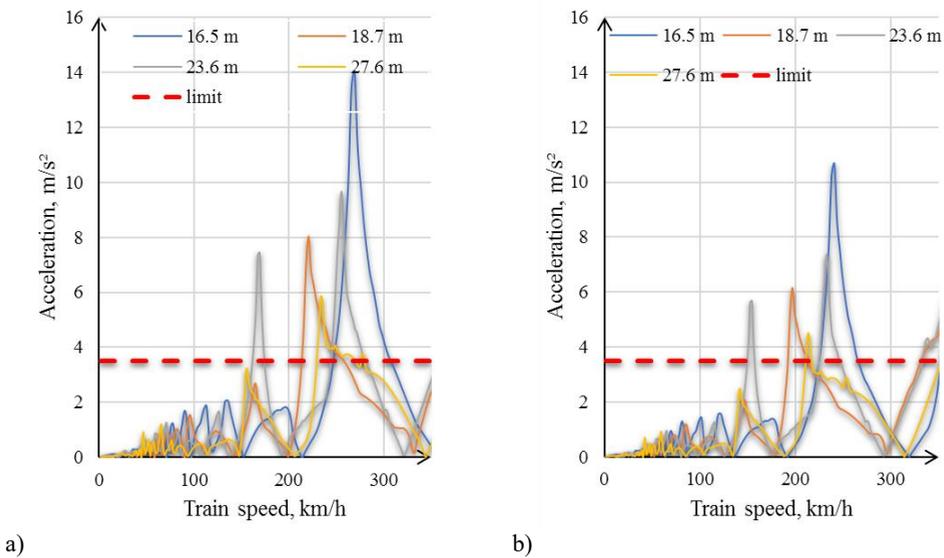


Fig. 9. A diagram showing the ratio between the maximum vertical accelerations of ribbed prestressed RC superstructures and the *Sapsan (Velaro RUS)* trains effect: a) with minimum weight; b) with maximum weight.

5 Conclusion

Based on the results of calculations the following conclusions can be made:

- Plate RC superstructures with relatively high value of weight do not meet the limits of maximum accelerations at train speed of more than 100 km/h.
- Ribbed RC superstructures with much lower value of weight compared to the plate RC ones are more sensitive to the dynamic effect of high-speed trains and cease to meet the limits of maximum accelerations at train speed of more than 100 km/h.
- Ribbed prestressed RC superstructures have relatively high stiffness allowing to ensure that limits of maximum accelerations are not violated at train speed up to 150 km/h.
- Provided that plate and ribbed RC superstructures get modernized, they may be used on railway lines with train speed up to 200 km/h.
- For high-speed railways with train speed up to 350 km/h one should prefer ribbed prestressed RC superstructures; however, they also need some additional design solutions.
- Use of existing unified bridge superstructures on high-speed railways without special adaptation will lead to destabilization of the ballast section and development of track irregularities, which in its turn will have a negative effect on passenger comfort and traffic safety.

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