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Dynamic analysis of highway bridges considering a progressive pavement deterioration model

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Abstract. Highway bridges are subjected to random traffic loads along their lifecycle and also subjected to relevant impact dynamic loadings. The road-roughness of asphalt pavements represents a key issue to the significant increase of the displacement and stress values on the highway bridge decks. Considering this fact, the aim of this research work is to develop an analysis methodology to evaluate the displacement and stress values of a steel-concrete composite highway bridge, including the dynamic actions due to vehicles convoy and also the effect of the progressive deterioration of the pavement, taking into account the road surface damages. The analyzed structural model corresponds to a typical steel-concrete composite highway bridge deck, with straight axis, simple supported and spanning 13.0m by 40.0m. In this investigation, the numerical model developed for the dynamic analysis of the steel-concrete composite bridge, adopted the usual mesh refinement techniques present in Finite Element Method (FEM) simulations implemented in the ANSYS computational program. The main conclusions of this study focused on alerting structural engineers to the significant distortions associated to the bridge dynamic structural response, when subjected to dynamic actions produced by vehicles convoys on the irregular pavement surface.

Keywords: highway bridges, dynamic structural analysis, irregular pavement surface.

1 Introduction

During the life cycle of a bridge, dynamic impacts can induce significant increase of the displacement and stress values produced by random traffic loads and deteriorated road surface conditions. These dynamic actions can generate the nucleation of fractures or even their propagation on the bridge deck structure. Consequently, this problem is substantial for the superstructure and pavement of the bridge causing its premature deterioration, especially in regions where road maintenance is not effective [1].

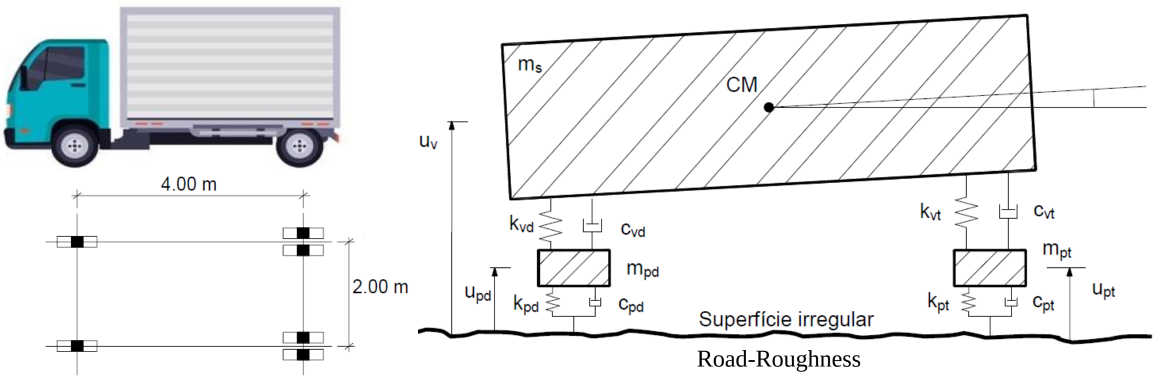
Since the middle 80's, the scientific community, in recognition of the major importance of this subject, has started a continuous effort on the study of the dynamic effects on bridge superstructures caused by vehicular traffic on irregular pavement surfaces. Several studies have been published from this period on making evident that the effects produced by the interaction of vehicle wheels with an irregular pavement surface, can be much more important than those produced only by the smooth movement of the vehicles [2, 3].

In view of the fact that approaches based on the use of a unique road-roughness level for the entire bridge lifecycle can lead to unrealistic results or over-conservative lifecycles whether an excellent or poor roughness level is adopted, it is necessary and more realistic to consider the influence of the progressive degradation of the road surface roughness based on the use of a vehicle-bridge interaction model [4, 5].

The proposed analysis methodology evaluates the dynamical effects on steel-concrete composite highway bridge decks due to vehicles crossing on the rough pavement surfaces defined by a probabilistic model, including the dynamic actions caused by vehicles convoys and also the effect of progressive deterioration of the pavement. The main conclusions have indicated that the road-roughness condition directly influences the dynamic structural response of the bridge, which over time, the more deteriorated the road condition is, the more it induces larger vertical translational displacement and stress values.

2 Mathematical modelling of the vehicles

The truck used in this work is presented in Fig. 1 a), being one of the most common vehicles in the local roads of Brazil. The two-axle truck structural-mechanical model used is shown in Fig. 1 b), with their dynamic properties based on experimental measurements [6]. This model presents 4 degrees of freedom. The geometry, mass distribution, damping, and stiffness of the tires and suspension systems of the truck are listed in Tab. 1.



a) Truck geometry scheme: 2C vehicle

b) Modelling of the rigid body, springs and dampers

Figure 1. Model of the two axle truck prototype

Table 1. Dynamic properties of the vehicle (2 axles)

Parameter	1st Axle	2nd Axle	Units
Suspension spring stiffness (k_v)	864	2,340	kN/m
Tire spring stiffness (k_p)	1,620	6,720	kN/m
Suspension mass (m_p)	635	1,066	kg
Total mass (m)	20.3		t
Truck body mass (m_s)	18,599		kg
Natural frequencies (f)	[1.17 ; 2.08 ; 10.00 ; 14.73]		Hz

3 Modelling of the road surface roughness

Road surface roughness is generally defined as an expression of irregularities on the road surface, and it is the primary factor in affecting the dynamic response of both vehicles and bridges, see Silva and Roehl [7]. Based on the studies carried out by Dodds and Robson [8], the road surface roughness was assumed as a zero-mean stationary Gaussian random process and it can be generated through an inverse Fourier transformation as shown in eq. (1):

$$r(x) = \sum_{i=1}^{N} \sqrt{2 \;.\; \Delta\Omega \;.\; G_d(\Omega_i)} \;.\; \cos(2\pi \;.\; \Omega_i x + \theta_i). \tag{1}$$

Where θ_i = random phase-angle uniformly distributed from 0 to 2π; $G_d(\Omega)$ = power spectral density (PSD) function (cm^3/cycle) for the road surface elevation; and Ω_i = wave number (cycles/m). The PSD function for road surface roughness was developed by Dodds and Robson [8], as presented in eq. (2):

$$G_d\,(\Omega_i) = G_d\,(\Omega_0)\left[\frac{\Omega}{\Omega_0}\right]^{-2}. \tag{2}$$

Where Ω = spatial frequency of the pavement harmonic i (cycles/m); Ω_0 = discontinuity frequency of $1/2\pi$ (equal to 1 rad/m); and $G_d(\Omega_0)$ = road roughness coefficient (m^3/cycle), also called Road-Roughness Coefficient (RRC), whose value is chosen depending on the road class shown in Tab. 2, EN 1991-2 [9].

Table 2. Average values of G_d (Ω_0) for different levels of road roughness quality (in cm^3), EN 1991-2 [9]

Road Class	Road Quality Level	G_d (Ω_0) (cm^3): Lower	G_d (Ω_0) (cm^3): Mean	G_d (Ω_0) (cm^3): Upper
A	Excellent	-	1	2
B	Good	2	4	8
C	Average	8	16	32
D	Poor	32	64	128
E	Very poor	128	256	512

3.1 Modelling of progressive deterioration for road surface

In order to consider the road surface damages as a result of loads or corrosions, a progressive deterioration model for the road-roughness is necessary when generating the random road profiles. Paterson and Attoh-Okine [10] have developed such model considering the International Roughness Index (IRI) with the values at any time after the road surface service is calculated using eq. (3):

$$IRI_t = 1{,}04e^{\eta t}\, IRI_0 + 263\,(1+ SNC)^{-5}\,(CESAL)_t. \tag{3}$$

Where IRI_t = IRI value at time t; IRI_0 = initial roughness value directly after completing the construction and before opening to traffic; t = time in years; η = environmental coefficient; SNC = structural number; and $(CESAL)_t$ = estimated number of traffic in terms of AASHTO 80-kN (18-kip) cumulative equivalent single axle load at time t, in millions.

The initial IRI_0 varies from one region to another depending on the specifications for road construction adopted in each country. In this work, 0.90 m/km is adopted. The environmental coefficient, η, varies from 0.01 to 0.7 and depends on usually adopted dry/wet, freezing/non-freezing conditions, equaling 0.10 for bridges exposed to general environment conditions. Structural number, SNC, is a parameter that is calculated from data on the strength and thickness of each layer in the pavement, herein adopted is equal to 4. Equation (4) was used to estimate the number of traffic in terms of AASHTO 80-kN (18-kip).

$$(CESAL)_t = f_d\, n_{tr}(t)\, F_{Ei}\, 10^{-6}. \tag{4}$$

Where f_d = design lane factor; $n_{tr}(t)$ = cumulated number of truck passages for the future year t, estimated using eq. (5); and F_{Ei} = load equivalency factor for axle category i, calculated following strictly the rules of AASHTO Guide for Design of Pavement Structures [11].

CESAL changes in consequence of the yearly traffic increase, also resulting in a change of the progressive deterioration function. Kwon and Frangopol [12] based on the ADTT and traffic increase rate per year, estimated the cumulated number of truck passages for the future year t using eq. (5):

$$n_{tr}(t) = N_{obs}\left[\frac{(1+\alpha)^t - 1}{\ln(1+\alpha)}\right]. \tag{5}$$

Where subscript tr means trucks only; t = number of years; N_{obs} = total number of vehicles at first year, considered equaling 50,000, due to the localization of the bridge within a local road with a low traffic of trucks [9]; and α = traffic increase rate per year, adopted in this investigation is equal to 3% and 5%.

The IRI was developed in 1986 and is used to define the longitudinal profile of a traveled wheel track [13]. The IRI is based on the average rectified slope (ARS), which is a filtered ratio of a standard vehicle's accumulated suspension motion divided by the distance traveled by the vehicle during the measurement. In contrast, the International Organization for Standardization [14] used RRC to define the road-roughness classification, and the ranges are listed in Tab. 3. It should be noted that RRC is based on the road profiles only. Various correlations have been developed between the indices [15, 16]. Based on the corresponding ranges of the road-roughness coefficient and the IRI value [16], a relationship between the IRI and the RRC is utilized in the present study, as presented in eq. (6):

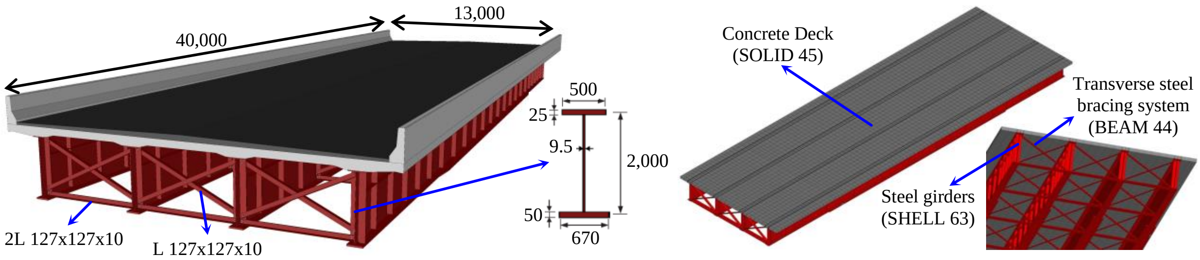
$$RRC_t = G_d(\Omega_0)_t = 6{,}1972 \times 10^{-9} \times \exp[\, IRI_t / 0{,}42808\,] + 2 \times 10^{-6}. \tag{6}$$

Table 3. RRC Values for road-roughness classification [14]

Road-roughness Classification	Ranges for RRCs
Very good	2×10^{-6} to 8×10^{-6}
Good	8×10^{-6} to 32×10^{-6}
Average	32×10^{-6} to 128×10^{-6}
Poor	128×10^{-6} to 512×10^{-6}
Very poor	512×10^{-6} to 2048×10^{-6}

4 Investigated highway bridge and finite element model

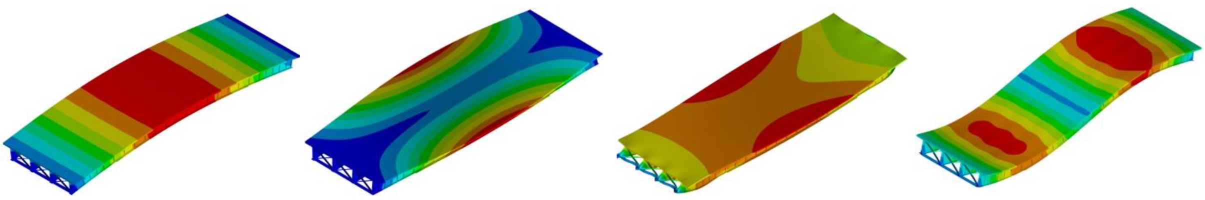
The investigated structural model corresponds to a typical steel-concrete composite highway bridge deck, with straight axis, simple supported, and spanning 13.0m by 40.0m. The structural system is constituted by four composite girders and a 0.225m thick concrete slab, see Fig. 2. The steel sections considered are related to welded wide flanges made with A588 steel with 350 MPa yield strength and 485 MPa ultimate tensile strength.



a) Structure design (units in millimetres)   b) Finite element model of the bridge

Figure 2. Investigated simply supported steel-concrete highway bridge deck

The computational model developed for the dynamic analysis of the composite bridge adopted the usual mesh refinement techniques present in the finite element method simulations implemented in the ANSYS program. The girder top and bottom flanges, the girder web, and the longitudinal and vertical stiffeners were represented by shell finite elements (SHELL63). The bridge concrete slab was simulated by solid finite elements (SOLID45). The transverse steel bracing system was simulated by beam finite elements (BEAM44). The final computational model adopted used 17,452 nodes, 16,112 elements, which resulted in a numeric model with 105,252 degrees of freedom. The damping ratio is assumed to be 0.5 %, as stated by EN 1991-2 [9] for steel and composite steel-concrete bridges. The associated composite bridge main global vibration modes are shown in Fig. 3.



a) f_{01} = 2.97 Hz   b) f_{02} = 3.67 Hz   c) f_{03} = 6.28 Hz   d) f_{04} = 9.67 Hz

Figure 3. Main global vibration modes of the investigated bridge obtained using the finite element modelling

5 Effects of the progressive deterioration for road surfaces

The road-roughness classification is defined in accordance with ISO 8608 [14], see Tab. 3. Based on the RRC, calculated from eq. (6), three traffic increase rates were investigated (α = 0%, α = 3% and α = 5%) in a 15-year period. The road condition in the first 10 years was classified as very good, the eleventh and twelfth years as good, the thirteenth as average, the fourteenth as average to the traffic increase rate at 3% and poor to the traffic increase rate at 5%, and fifteenth as poor (Fig. 4).

In order to extend the study of the dynamic behavior of the structure to different traffic conditions, the vehicles convoys were separately positioned in a central lane, a lateral lane and two other lateral ones. The spacing between the single axle and the double wheel single axle of two consecutive vehicles that was adopted is equal to 11.0m (Fig. 5). To evaluate the project response spectra, for each of the traffic conditions, the speed parameter of vehicles convoy varies from 20 to 80 km/h, in 10 km/h intervals, resulting in 7 different speeds. Furthermore, for each of these rates the three levels of traffic increase (α = 0%, α = 3%, α = 5%) were considered.

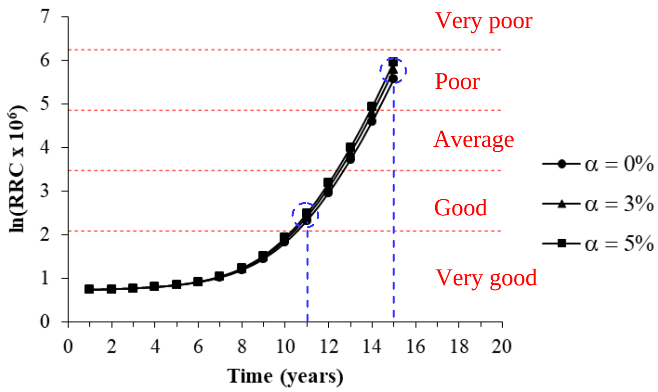


Figure 4. Deterioration of road-roughness condition in a 15-year period in terms of ln (RRCx10^6)

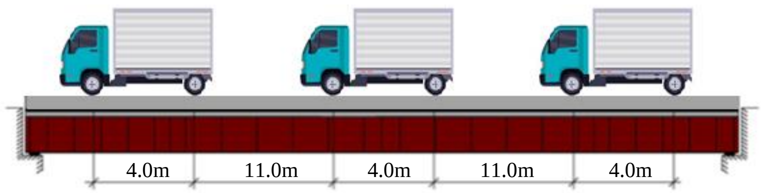


Figure 5. Vehicles convoy spacing

It is noteworthy that the following results are only for situation without deterioration (t = 0) and for t = 11 and 15 years that characterize the change in RRC classification from very good to good and from average to poor, respectively, as shown in Fig. 4.

It was possible to build twenty-one displacement spectra from the maximum displacements obtained for each speed. These spectra were grouped in Fig. 6, with seven spectra in each column, which show the spectra built considering vehicle convoy in three positions, as follows: central lane [column a)], lateral lane [column b)], and two lateral lanes [column c)], respectively, for a no deterioration condition (t = 0) and considering the three levels of traffic increase for a t = 11 and 15 year condition.

In the three cases of vehicle convoy position analyzed in Fig. 6 spectra, for a scenario without deterioration [Fig. 6 a.1), b.1), c.1)] and for t = 11 years [Fig. 6 a.2), b.2), c.2)], it is possible to observe the presence of two peaks: one of greater magnitude associated with the speed of 70 km/h and the other of lesser magnitude associated with the speed of 30 km/h. The most important peak (70 km/h) is associated with crossing frequencies and equals to 1.30 Hz (f = 70/3.6/15) due to the mobility between single axles of two consecutive vehicles, spaced 15.0m (Fig. 5), able to vibrate in the second harmonic (2.60 Hz) which is the fundamental frequency of structure (f_{01} = 2.97 Hz) causing resonance.

Regarding the lower peak observed for 30 km/h, it is associated with crossing frequencies related to a smaller spacing between axes, due to the very lower speed. In this particular case, the spacing is 11.0m, between the single directional axis and the single axis of double wheels of two consecutive vehicles (Fig. 5), whose predominant crossing frequency is 0.76 Hz (30/3.6/11), can only vibrate in the fourth harmonic (3.04 Hz) which is the fundamental frequency of the structure (f_{01} = 2.97 Hz). For this reason, the peak associated with 30 km/h speed is smaller than the one associated with the speed of 70 km/h.

However, it can be observed that for t = 15 years [Figs. 6 a.3), b.3), c.3)] the peak reaches 40 km/h. This is because vehicle convoys with 30 km/h speed vibrate the fundamental frequency of the structure in the fourth harmonic, while those of 40 km/h are capable of vibrating in the third harmonic of the structure's fundamental frequency. For this reason, the peak is tending towards 40 km/h rather than 30 km/h.

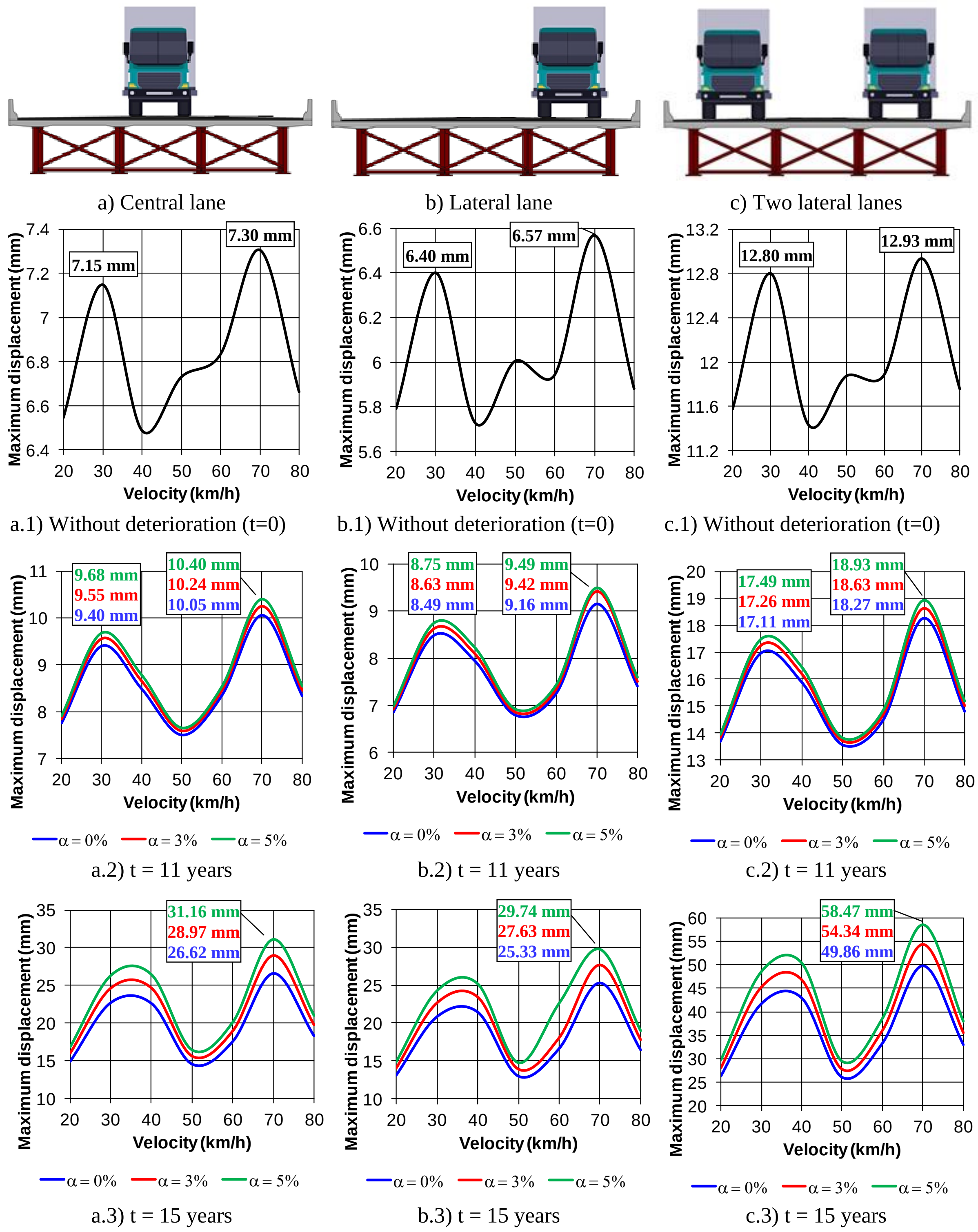


Figure 6. Response spectra: translational vertical displacement values

6 Conclusions

The main conclusions of this study focused on alerting structural engineers to the possible distortions, associated to the dynamic structural response of a steel-concrete composite bridge when subjected to dynamic actions such as vehicle convoys on the irregular pavement surface. This way, the following conclusions can be drawn from the results presented in this study:

1. The vehicle velocity affects the vertical translational displacement values of the bridge. In all investigated situations, the bridge dynamic structural response was modified when velocity was changed.
2. The road-roughness condition influences directly the dynamic structural response of the steel-concrete composite highway bridge. Over time, more deteriorated road condition induces larger vertical translational displacement which leads to higher stress values.
3. Based on the increase of the traffic rates (α = 0% to 5%), it must be emphasized that the vertical translational displacements were considerably higher. The quantitative modifications observed on the bridge dynamic structural response, associated to the situation without deterioration and the worst case analyzed (α = 5%, t = 15) were higher than three times.

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