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ORIGINAL ARTICLE

Steel-concrete composite highway bridges dynamic structural behaviour assessment considering the pavement progressive deterioration effect

Avaliação do comportamento estrutural dinâmico de pontes rodoviárias mistas (aço-concreto) considerando o efeito da deterioração progressiva do pavimento

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Abstract: Highway bridges are usually subjected to random dynamic actions of variable magnitude due to vehicles convoys crossing on the bridge pavement deck along their service life. In this context, the asphalt pavement deteriorated road surface condition represents a key issue to assess the bridge displacement and stress values. This way, this research work aims to develop an analysis methodology in order to assess the dynamic structural behaviour of steel-concrete composite highway bridges including the vehicles convoys dynamic actions and the pavement progressive deterioration effect. The conclusions of this investigation have indicated that the displacements and stresses values are considerably higher, with relevant amplifications up to four times, when the road pavement deterioration effect is considered in the bridge dynamic analysis.

Keywords: steel-concrete highway bridges, dynamic structural analysis, pavement progressive deterioration effect, finite element modelling.

Resumo: As pontes rodoviárias geralmente estão sujeitas a ações dinâmicas aleatórias de magnitude variável devido aos comboios de veículos que cruzam o tabuleiro da ponte ao longo de sua vida útil. Diante deste contexto, a condição de deterioração da superfície do pavimento asfáltico da estrutura torna-se relevante para avaliar os valores de deslocamentos e tensões. Deste modo, este trabalho de pesquisa tem como objetivo desenvolver uma metodologia de análise para avaliação da resposta estrutural dinâmica de pontes rodoviárias mistas (aço-concreto), considerando-se as ações dinâmicas oriundas do tráfego de comboios de veículos sobre a superfície irregular, além do efeito da deterioração progressiva da superfície do pavimento. As conclusões desta investigação indicam que os valores de deslocamentos e tensões são consideravelmente superiores, com amplificações de até quatro vezes, quando o efeito da deterioração do pavimento da pista é considerado na análise dinâmica da ponte.

Palavras-chave: pontes rodoviárias mistas (aço-concreto), análise estrutural dinâmica, efeito da deterioração progressiva do pavimento, modelagem em elementos finitos.

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1 INTRODUCTION

During the life cycle of a bridge, dynamic impacts related to random traffic loads and deteriorated road surfaces can induce significant increase of the displacements and stress values. These dynamic actions can generate the nucleation of fractures or even their propagation on the bridge deck. This is a relevant problem, especially in regions where road maintenance is not effective, causing premature deterioration of the bridge's superstructure and pavement [1]. Considering the relevance of this scenario, the scientific community has initiated a continuous effort since the middle 80's, associated to the assessment of the dynamic effects due to vehicles traffic on bridge irregular pavement surfaces.

Several research works have been developed [1]–[4] and it was made evident that the effects due to the dynamic interaction between the vehicle's wheels and the irregular pavement surface can be much more important than those produced by the vehicle's smooth movement [2]. The poor road-roughness levels could lead to high amplifications and in some cases, these effects are even larger than those due to the vehicle's static presence, increasing drastically with the decrease of the pavement surface quality [2].

Thus, approaches based on the use of a unique road-roughness level for the entire bridge life cycle can lead to unrealistic results or over-conservative life process whether an excellent or poor roughness level is adopted, respectively. 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. This influence was considered in the research works developed by Zhang and Cai [3], [5] conducting to more realistic estimations.

This way, having these thoughts in mind, in this paper an analysis methodology is developed in order to assess the steel-concrete composite highway bridge decks dynamic structural behaviour due to the vehicles crossing on the rough pavement surfaces, defined by a probabilistic model, considering the dynamic actions of vehicles convoys and also the progressive pavement surface deterioration effect [1].

Therefore, the studied structural model corresponds to a typical steel-concrete composite highway bridge deck, with straight axis, simple supported and spanning 13.0 m by 40.0 m [6]. The developed numerical model adopted the usual mesh refinement techniques present in Finite Element Method (FEM) simulations implemented in the ANSYS [7] program.

Initially, the investigated bridge road surface roughness was defined based on the use of the Power Spectral Density (PSD) function, as an expression of the road surface random irregularities. The road surface roughness was assumed as a zero-mean stationary Gaussian random process [2]. After that, a complete mathematical formulation associated to the bridge road pavement progressive deterioration effect, along the time, was investigated [1].

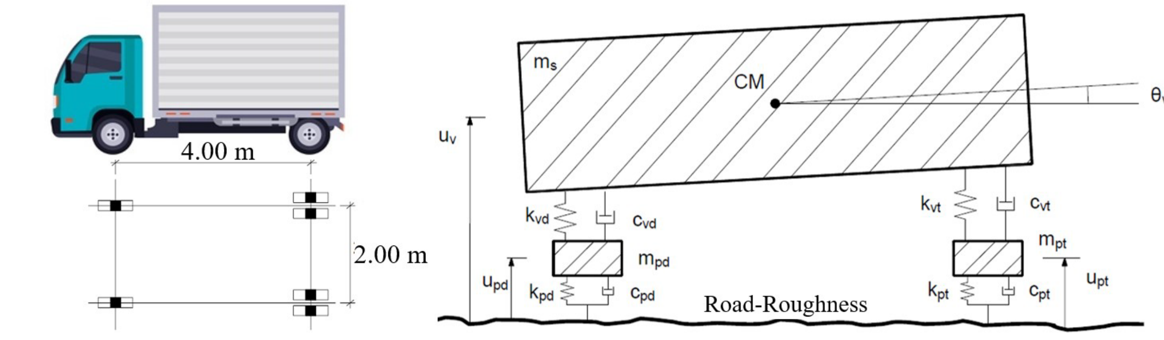
This way, the bridge dynamic structural response is obtained by the integration of its equations of motion, in the time domain, considering the excitation produced by the vehicles traffic on the bridge deck irregular pavement surface, incorporating the pavement progressive deterioration effect [1], [2].

The bridge dynamic structural response was investigated through an extensive parametric study based on the calculated displacements and stresses values. This way, response spectra were generated considering the vehicles convoys velocities between 20 km/h and 80 km/h, and the increases on different levels of vehicles traffic on the deck structure along of a period of 15 years, aiming to investigate the road pavement progressive deterioration effect on the bridge dynamic response.

The main conclusions of this this research work focused on alerting structural engineers to the possible distortions associated with the steel-concrete composite bridge dynamic structural response, when subjected to dynamic actions due to vehicle convoys on the irregular pavement surface. The results have indicated that the displacements and stresses values were considerably higher, with relevant amplifications up to four times, when the road pavement deterioration effect was considered in the bridge dynamic analysis.

2 VEHICLES MATHEMATICAL MODELLING

The truck adopted in this work is presented in Figure 1a, and is one of the most common vehicles used in Brazilian roads [8], but other types are available [9]. The developed two-axle truck structural-mechanical mathematical model is shown in Figure 1b. The vehicle dynamic properties were determined based on experimental tests [10], and the vehicle mathematical model presents four degrees of freedom: three translations and one rotation. The dynamic properties (mass, damping and stiffness), including the truck tires and suspensions are listed in Table 1.



(a) Truck geometry: 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 (2C 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

The road surface roughness is generally defined as an expression of road surface irregularities and it is the primary factor affecting the dynamic response of both vehicles and bridges [2], [10]. Based on the studies carried out by Dodds and Robson [11], 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 Equation 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); and Ω_i = wave number (cycles/m).

The PSD function for road surface roughness was developed by Dodds and Robson [11] as presented in Equation 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 RRC, whose value is chosen depending on the road class presented in Table 2, EN 1991-2 (Annex B) [12].

Table 2. Average values of $G_d(\Omega_0)$ for different levels of road quality (in cm^3) [12].

Road class	Road quality level	$G_d(\Omega_0)$: lower	$G_d(\Omega_0)$: mean	$G_d(\Omega_0)$: 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

4 PAVEMENT PROGRESSIVE DETERIORATION EFFECT MODELLING

In this work a mathematical formulation associated to the bridge road pavement progressive deterioration effect was investigated. This way, in order to consider the road surface damages from loads or corrosions, a progressive deterioration model for the road-roughness is necessary when generating the random road profiles. Thus, Paterson and Attoh-Okine [13] have developed a model considering the International Roughness Index (IRI) with the values at any time after the service of the road surface being calculated, see Equation 3.

$$IRI_t = 1.04e^{\eta t} \left[IRI_0 + 263\,(1+ SNC)^{-5}\,(CESAL)_t \right] \tag{3}$$

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

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

where f_d = design lane factor; $n_{tr}(t)$ = cumulated number of truck passages for the future year t, estimated using Equation 5; and F_{Ei} = load equivalency factor for axle category i, calculated following the rules of AASHTO Guide for Design of Pavement Structures [14].

Due to the yearly traffic increase, the CESAL parameter is modified resulting in a change of the progressive deterioration function. Kwon and Frangopol [15], based on the Average Daily Truck Traffics (ADTTs) and traffic increase rate per year, also estimated the cumulated number of truck passages for the future year t using Equation 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 equal to 50,000, due to the localization of the bridge within a local road with a low traffic of trucks [12]; and α = traffic increase rate per year. In this research work the traffic increase rate per year was adopted equal to 0%, 3% and 5%.

The IRI formulation was developed in 1986 and is used to define the longitudinal profile of a travelled wheel track [16]. This coefficient (IRI) is based on the average rectified inclination (ARS), which is a filtered ratio of the accumulated movement of the standard vehicle suspension divided by the distance travelled by the vehicle during the measurement. According to Sayers et al. [17], since the World Bank published a technical report for conducting and calibrating the roughness measurements, IRI started to be used as a worldwide standard method for analysing the road longitudinal profile [1], [4].

Alternatively, the International Organization for Standardization [18] used RRC to define the road-roughness classification, and the ranges are listed in Table 3. This coefficient (RRC) was created to relate the tire characteristics to the road rolling resistance. Several correlations have been developed between the IRI and RRC indexes [19], [20]. Based on the corresponding ranges of the road-roughness coefficient and the IRI value [20], a relationship between IRI and RRC is used in the present study as shown in Equation 6.

$$RRC_t = G_d(\Omega_0)_t = 6.1972 \times 10^{-9} \times \exp\left[IRI_t / 0.42808 \right] + 2\times10^{-6} \tag{6}$$

Table 3. RRC Values for road-roughness classification [18].

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}

In sequence, Figure 2 presents a general flowchart for the pavement progressive deterioration mathematical modelling investigated in this research work, indicating the used equations and a logical sequence of application.

5 INVESTIGATED HIGHWAY BRIDGE

In this work, a typical simply supported steel-concrete composite highway bridge designed in accordance with AASHTO LRFD [21] and NBR 16694 [22] design specifications was investigated. The selected case study is a good representative of real simply-supported highway bridges with straight axis and spanning 13 m by 40 m [6]. The structural system is constituted by four composite girders and a 0.225 m thick concrete slab. In order to prevent web buckling, steel plate stiffeners are welded along the steel girders with 1880 mm spacing at the span-sections and 1200 mm spacing at the support-sections. Steel plate stiffeners were also adopted above the girder supports. Two different cross sections were adopted along the longitudinal composite beams, designated as support cross-section and span cross-section. The bridge structural system comprises cross diaphragms like steel trusses. The diaphragms are made of equal angle profiles with 10 mm wall thickness, see Figures 3 to 5.

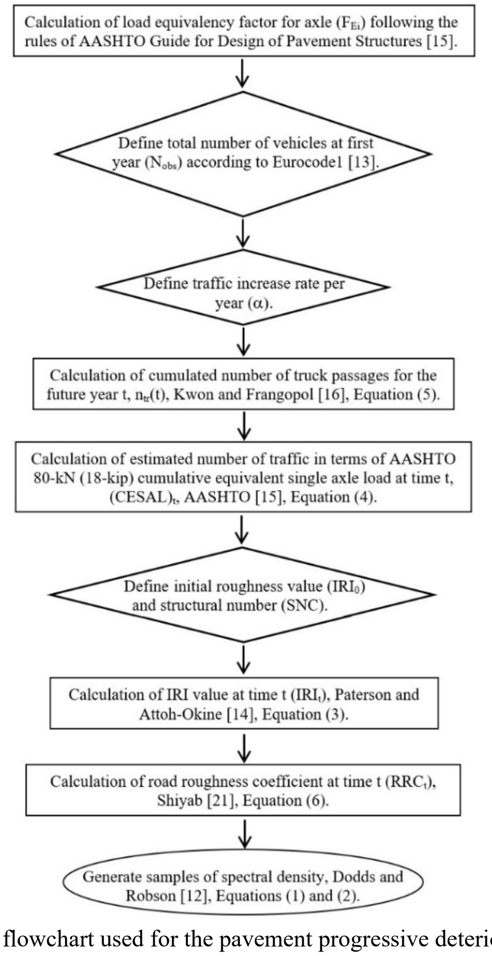


Figure 2. General flowchart used for the pavement progressive deterioration modelling.

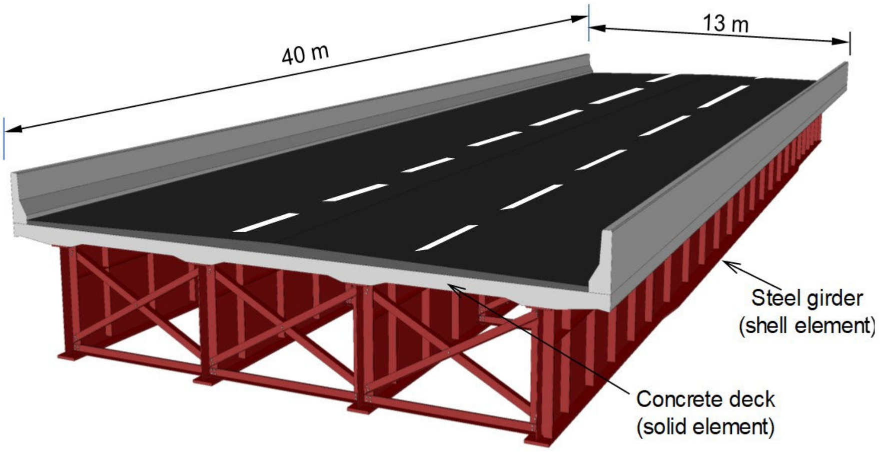


Figure 3. Simply supported steel-concrete roadway bridge: overall view.

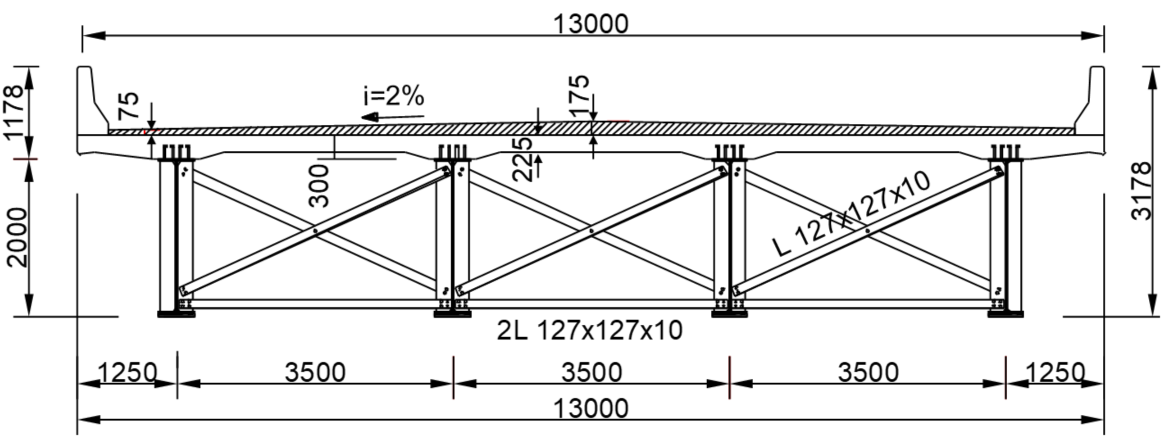


Figure 4. Bridge section at the support: units in millimetres [6].

The steel sections are composed by welded wide flanges made with A588 steel with 350 MPa yield strength and 485 MPa ultimate tensile strength. A $2.05{\times}10^5$ MPa Young's modulus with 0.3 Poisson's ratio and 7,850 kg/m^3 material density was adopted for the steel girders. The concrete slab has a density of 2,500 kg/m^3, 25 MPa compression strength and $3.05{\times}10^4$ MPa of Young's modulus, with 0.2 Poisson's ratio.

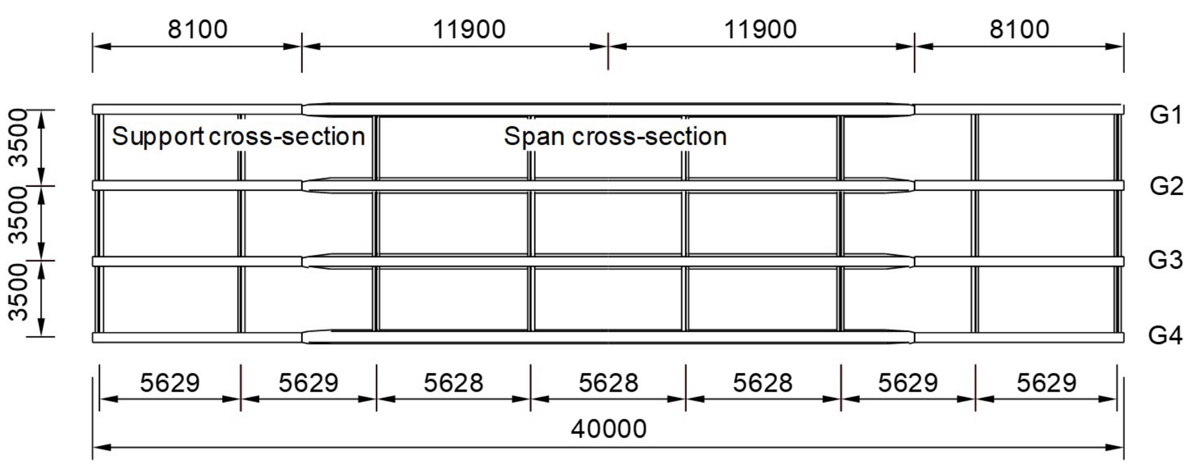


Figure 5. Bridge steel girder's top view: units in millimetres [6].

6 STEEL-CONCRETE COMPOSITE BRIDGE FINITE ELEMENT MODELLING

The numerical model developed for the steel-concrete composite bridge dynamic analysis adopted the usual mesh refinement techniques present in finite element method simulations implemented in the ANSYS [7] computational program, see Figure 6. The bridge girders top and bottom flanges, the girders web and the longitudinal and vertical stiffeners were represented based on the use of shell finite elements (SHELL63). The bridge concrete slab was simulated by solid finite elements (SOLID45). The transverse steel bracing systems were simulated by beam finite elements (BEAM44). The developed bridge finite element model used 17,542 nodes, 16,112 elements, which resulted in a numeric model with 105,252 degrees of freedom.

The strain compatibility between the solid elements (concrete slab) and the shell elements (steel plate girders) was guaranteed by coupling the corresponding degrees of freedom, simulating the composite bridge decks' full interaction. The damping ratio was assumed to be 0.005 (ξ = 0.5%), as stated by EN 1991-2 [12] for steel and steel-concrete composite bridges. In sequence, Figure 6 illustrates the investigated highway bridge finite element model. The investigated steel-concrete composite bridge natural frequencies (eigenvalues) and vibration modes (eigenvectors) have been determined based on numerical methods of extraction (modal analysis), through a free vibration analysis and using the ANSYS program [7]. The associated bridge main global vibration modes are shown in Figure 7.

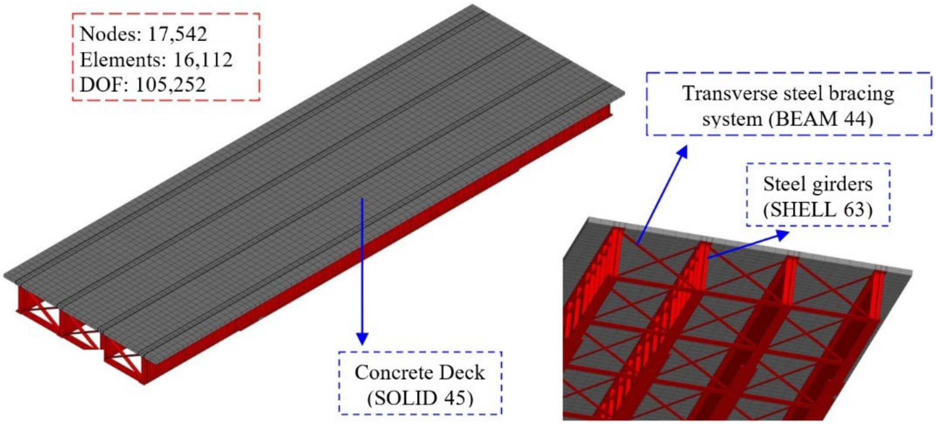


Figure 6. Finite element model of the roadway bridge.

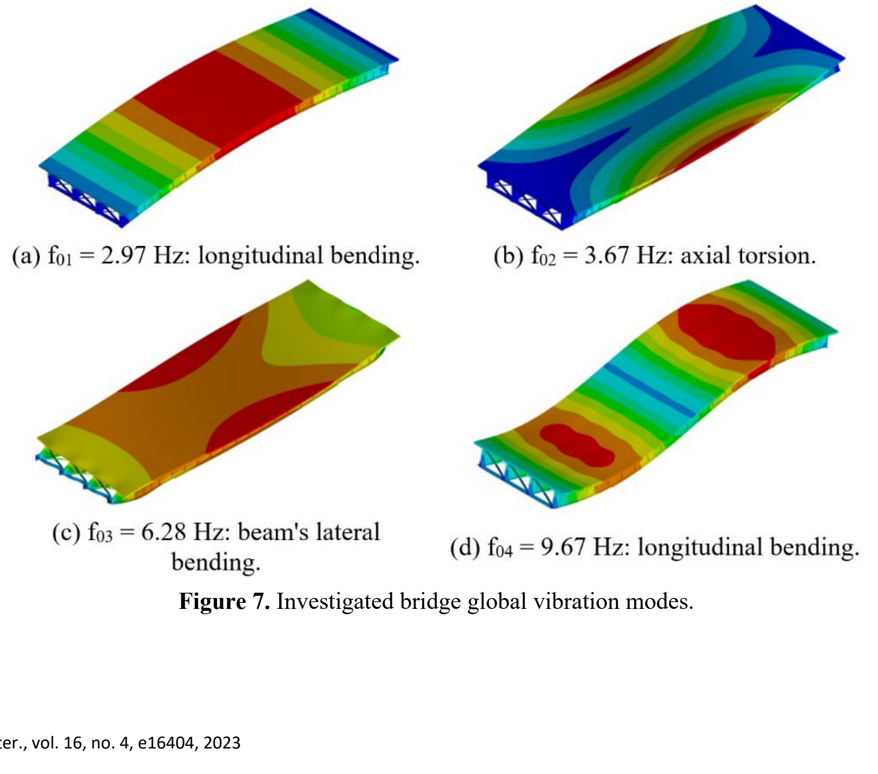


Figure 7. Investigated bridge global vibration modes.

7 MODELLING OF THE VEHICLE-BRIDGE DYNAMIC INTERACTION

The moving load is modelled by an infinite series of equal vehicles, regularly spaced, and running at constant velocity, υ. If "L" is the distance between two successive vehicles and if these cars enter the bridge deck one by one, a time repeated movement variation governed by the frequency, $f = \upsilon/L$, is created, and is associated with the vehicle movement on the bridge [2]. After a certain time period, t_1, named "crossing period", the first convoy vehicle reaches the far end of the bridge and from this instant on the total mass of the vehicles on the bridge remains practically constant, considering the condition that the distance L is sufficiently small. Under these conditions the bridge will soon reach a steady-state response situation, which includes the repetition of the maximum values [2].

The mathematical model simulates the bridge structure and the vehicle series as a system, the "vehicle-bridge system". In this system, the vehicle series and the bridge deck form just one system. Consequently, its frequencies are modified by the vehicle properties and the vehicle-bridge dynamic interaction force incorporates the bridge flexibility influence. The effect of the pavement roughness is introduced in the vehicle-bridge system equations of motion as a load vector analogous to what would be considered if the vehicle was subjected to a base movement equal to the irregular pavement profile [2].

The bridge dynamic response was obtained by the integration of its equations of motion, in the time domain, considering the excitation produced by the vehicles convoys' traffic on the bridge deck irregular pavement surface, taking into account the pavement progressive deterioration effect [1], [2]. The interaction force vector, F(t), is modified throughout time and, for each time interval (or "steps"), a dynamic force is applied on the bridge deck. The dynamic loading position changes according to the positions of the vehicles on the bridge deck, and the generated time function has a space and time description. This procedure encompasses the actual bridge structural response, where the dynamic loads can induce displacements and stresses greater than the maximum bridge response values.

8 PAVEMENT PROGRESSIVE DETERIORATION ANALYSIS

The road-roughness classification is defined in accordance with ISO 8608 [18] (Table 3) and Figure 8 illustrates RRC values calculated from Equation 6. The presented results are classified as very good for the first 10 years, as the RRC value is less than 8×10^{-6} (see Table 3). After 11 and 12 years of the pavement deterioration, the roughness is classified as good for the three investigated scenarios. The roughness after 13 years is classified as average for all studied traffic increase values, and also for $\alpha = 0\%$ and $\alpha = 3\%$ after a 14 years period. However, considering the age of 14 years onwards, the pavement is considered poor for a 5% increase in traffic and, after 15 years, the roughness rating is poor for all analysed scenarios.

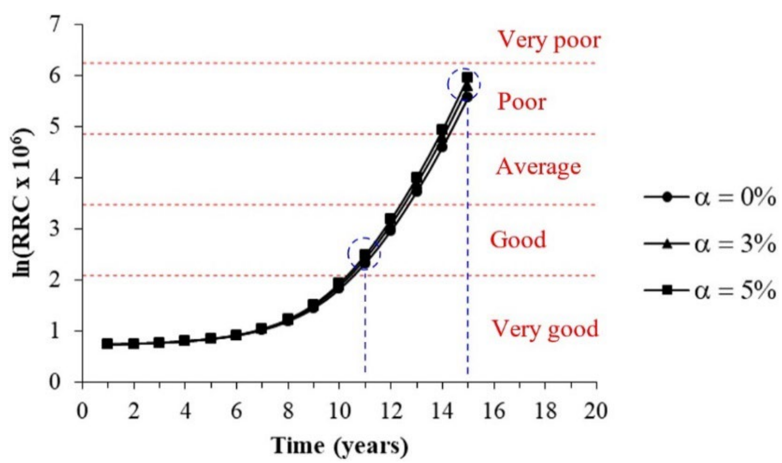


Figure 8. Deterioration of road-roughness condition in terms of ln(RRC×10^6).

9 DYNAMIC LOADINGS APPLICATION STRATEGY

In this study, the dynamic actions on the bridge are considered due to vehicle convoy's traffic on the irregular pavement surface and the effect of the pavement progressive deterioration, taking into account the road surface damages. The vehicles passage on the bridge deck may have different velocities and can be randomly or simultaneously located in different traffic lanes.

The dynamic loads on the bridge deck can be assumed as a series of identical vehicles [1]–[5] or even the passage of one vehicle at time and different vehicle configurations [10], [23]. In this research work it must be emphasized that the adopted dynamic loads application strategy, based on the use of a series of identical vehicles positioned in one bridge deck traffic lane, is relevant to allow the highway bridge decks steady-state response assessment. This way, the bridge deck width was divided in three equal lanes adopted for the vehicles traffic simulation, discounting the two New Jersey barriers widths, see Figure 9.

Moreover, the vehicles were positioned at the centre of the traffic lanes, having in mind each studied loading situation. Thus, the vehicle convoys are crossing the bridge deck with constant velocity up to a specified number of crossings (defined as t/t_1), which assumed in this work to be 10 ($t/t_1 = 10$). The spacing between the single axle and the double wheel single axle of two consecutive vehicles was limited by the bridge length and by a minimum space between consecutive vehicles (L = 11 m), as presented in Figure 10.

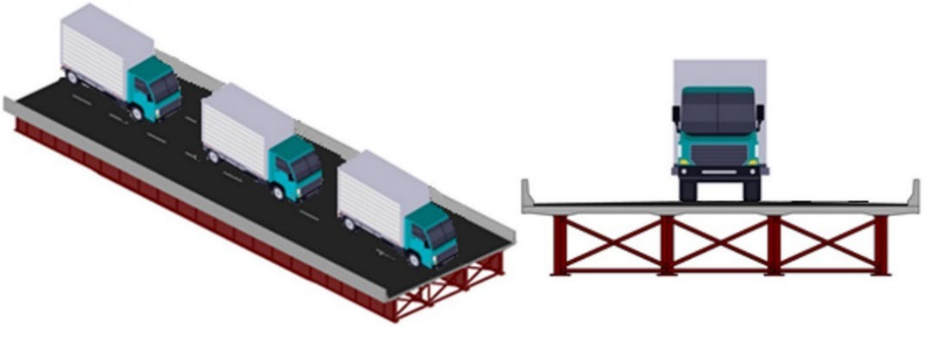


Figure 9. Vehicle convoys on the bridge deck: central traffic lane.

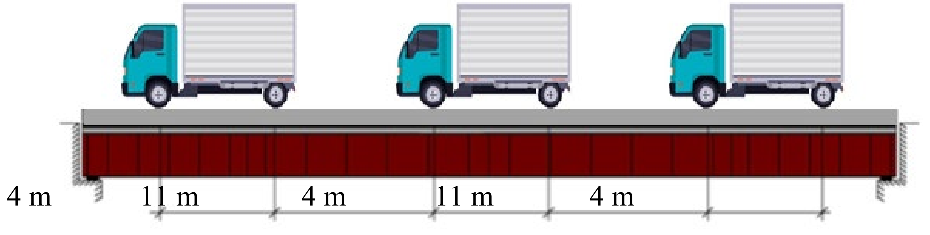


Figure 10. Vehicle convoys spacing: three vehicles.

10 BRIDGE RESPONSE SPECTRA ASSESSMENT

Aiming to assess the steel-concrete composite bridge dynamic structural behaviour, several response spectra were generated for each traffic condition (Figures 10 to 12). To do this, the vehicles velocities vary from 20 km/h to 80 km/h (intervals of 10 km/h), resulting in seven different velocities in each spectrum. Furthermore, for each considered vehicle velocity three different traffic increase levels were analysed in this study. This way, response spectra were generated to study the global bridge dynamic structural response, as presented in Figure 11.

Therefore, first of all, it is noteworthy that along the bridge dynamic analysis (forced vibration analysis) the following results considered the situation without the bridge deck pavement deterioration (t = 0), and also for t = 11 years and t = 15 years, that characterize the change in RRC (see Table 3), to a road-roughness classification from very good to good and from average to poor, respectively, as shown in Figure 8.

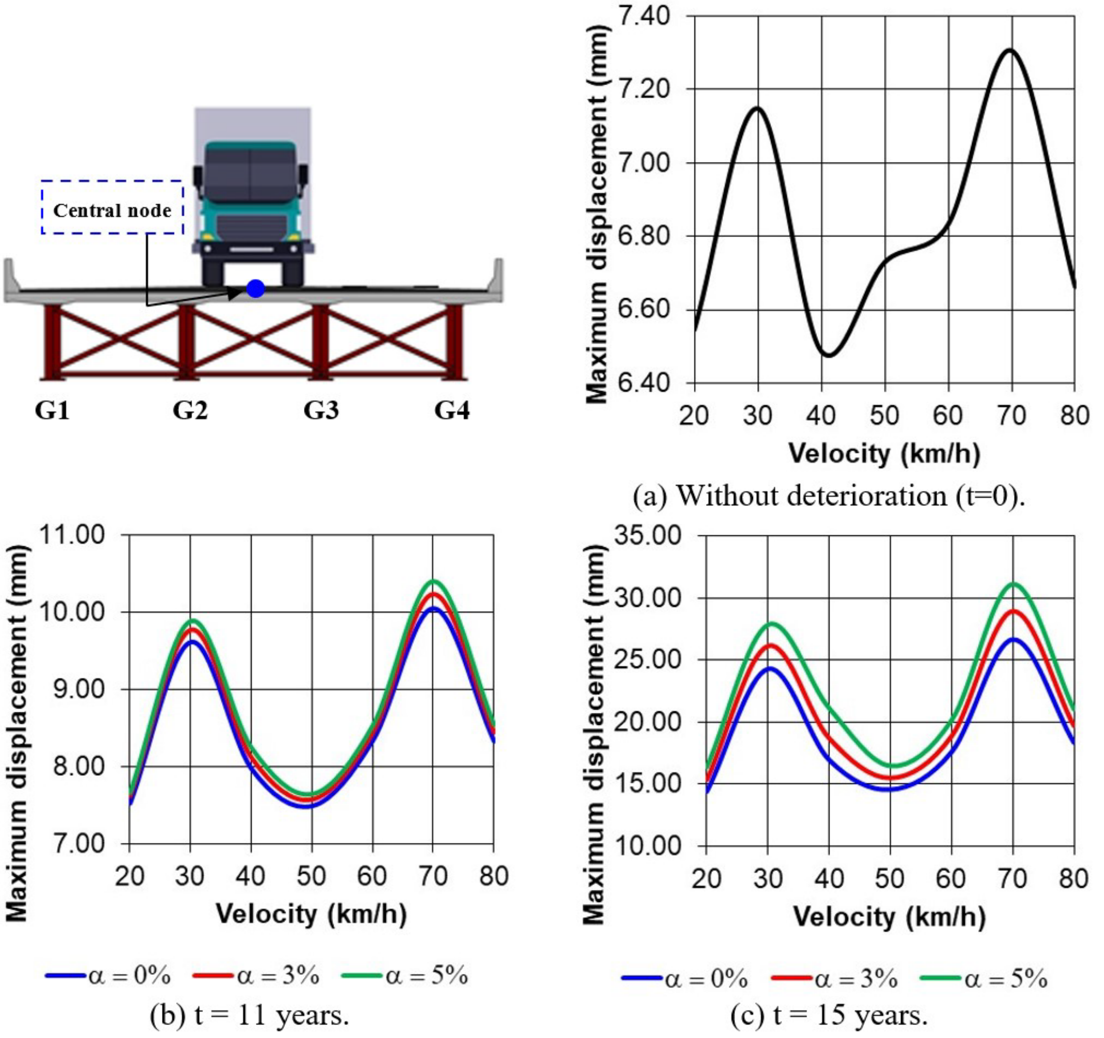


Figure 11. Response spectra: translational vertical displacements (see **Figure 9).**

The bridge response spectra illustrated in Figure 11 presents two main energy transfer peaks: the first peak of greater magnitude is associated with the vehicles traffic velocity of 70 km/h, and the other one is related to the velocity of 30 km/h. The most important peak (υ = 70 km/h) is associated with the dynamic excitation frequency (vehicles convoy crossing frequency on the bridge deck) equal to 1.30 Hz ($f = \upsilon/L = 70/3.6/15 = 1.30$ Hz), due to the mobility between two single axles spaced by 15 m and corresponding to consecutive vehicles (see Figure 10). This way, this dynamic excitation frequency mobilizes the second harmonic (2.60 Hz), associated to the bridge fundamental frequency (f_{01} = 2.97 Hz), close to the structure resonance frequency range.

On the other hand, the energy peak transfer (υ = 30 km/h), see Figure 11, is related to the dynamic excitation frequency (vehicles convoy crossing frequency on the bridge deck) equal to 0.76 Hz ($f = \upsilon/L = 30/3.6/11 = 0.76$ Hz), due to the mobility between the vehicles single directional axis and the double wheels single axis associated to two consecutive vehicles spaced by 11 m (see Figure 10). This way, this dynamic excitation frequency mobilizes only the fourth harmonic (3.04 Hz), associated to the bridge fundamental frequency (f_{01} = 2.97 Hz), close to the bridge resonance frequency range. For this reason, the peak associated with the velocity of 30 km/h is smaller than the one associated with the velocity of 70 km/h.

It must be emphasized that all the response spectra resonance peaks, presented in Figure 11, are related to the proximity of the excitation frequency produced by the vehicles convoy passage on the bridge deck and the natural frequencies of the system (vehicle-pavement-bridge system), having in mind the investigated traffic conditions, see Figure 9. In sequence, the investigated steel-concrete composite bridge maximum vertical translational displacements values are presented in Table 4.

Table 4. Translational vertical displacement values (see Figure 9).

Maximum displacements (mm): bridge central section							
Velocity	Without	α = 0%		α = 3%		α = 5%	
(km/h)	deterioration	t = 11	t = 15	t = 11	t = 15	t = 11	t = 15
20	6.54	7.53	14.40	7.63	15.40	7.66	16.37
30	7.15	9.62	24.25	9.78	26.19	9.89	27.88
40	6.49	7.98	16.92	8.12	18.72	8.25	21.12
50	6.73	7.5	14.54	7.58	15.54	7.65	16.46
60	6.83	8.35	17.58	8.45	18.92	8.53	20.15
70	**7.3**	**10.05**	**26.62**	**10.24**	**28.97**	**10.4**	**31.15**
80	6.66	8.33	18.32	8.45	19.73	8.54	21.04

11 STUDY OF THE PAVEMENT PROGRESSIVE DETERIORATION EFFECT

In this section, aiming to study the bridge general global behaviour, based on the pavement progressive deterioration progressive mathematical model, it was adopted the loading case considering the vehicles convoys positioned on the traffic central lane (Figure 9). This way, based on the bridge response spectra results (Figure 11), two velocities are considered in this analysis: 30 km/h and 70 km/h.

On the other hand, three scenarios of traffic increase (α = 0%, 3% and% 5%) are studied for a 15-year period, resulting in 90 different situations. Thus, Figures 12 and 13 present the progressive deterioration progressive effect, based on the displacements and stresses values analysis related to the investigated bridge central section.

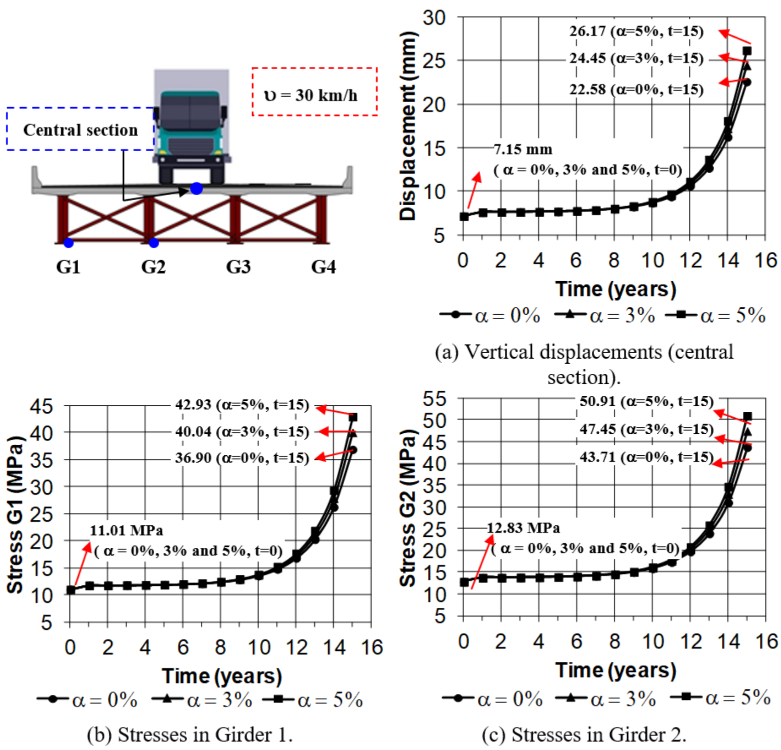
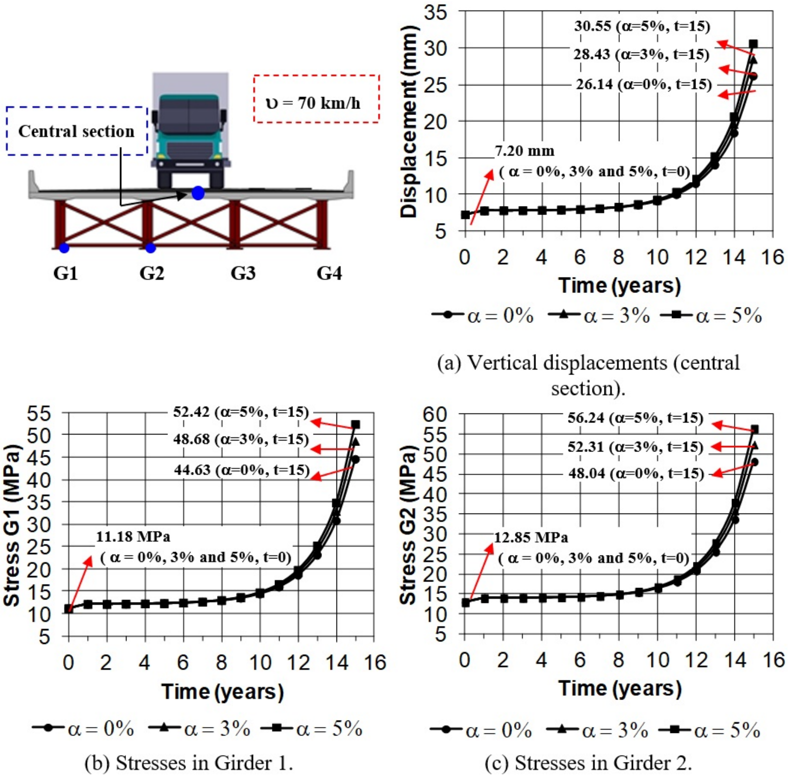


Figure 12. Pavement progressive deterioration effect: υ = 30 km/h (traffic central lane).



(a) Vertical displacements (central section).

(b) Stresses in Girder 1.

(c) Stresses in Girder 2.

Figure 13. Pavement progressive deterioration effect: υ = 70 km/h (traffic central lane).

Based on the Figures 12 and 13 results, it must be emphasized that over time, as the bridge deck pavement deterioration process increases, the calculated vertical translational displacements and stresses are higher. Considering the vehicle velocity of 30 km/h (υ = 30 km/h), after 15 years of deterioration, the displacement values exceed more than three times when compared to those associated to the pavement without deterioration (t = 0). On the other hand, when the velocity of 70 km/h (υ = 70 km/h) is investigated this value is up to four times higher, see Figure 12a and Figure 13a.

The same scenario can be observed when the stresses values are analysed, based on the bridge dynamic response and having in mind the pavement deterioration process. Over the years, the stresses values in the bridge G1 and G2 girders are up to three times higher than those obtained without deterioration (t = 0), when the vehicles velocity is equal to 30 km/h (υ = 30 km/h), and up to four times higher for 70 km/h (υ = 70 km/h), than those found without deterioration (t = 0), see Figures 12b and 12c and also Figures 13b and 13c.

Finally, based on the response spectra investigation, the bridge dynamic response maximum effects (displacements and stresses) occurs when the vehicles velocity is equal to 70 km/h (υ = 70 km/h), and the displacements and stresses values, after 15 years of pavement deterioration, are up to four times higher than those found without deterioration (t = 0). These results clearly indicate the relevance of considering the pavement deterioration effect, when the highway bridges structural behaviour subjected to dynamic loads is investigated.

CONCLUSIONS

In this research work an assessment of steel-concrete composite highway bridge decks dynamic structural behaviour is presented, considering the road surface roughness pavement progressive degradation effect, and based on the use of a vehicle-bridge-pavement interaction model.

The main objective was to determine the displacements and stresses values considering the vehicles convoys' traffic on the bridge deck taking to account the irregular pavement surface progressive deterioration up to 15 years of the structure service life. This way, having in mind the steady-state response assessment, in this research work the dynamic loads on the bridge deck were assumed as series of three 2C vehicles, regularly spaced and moving at constant velocity, and three different scenarios of traffic increase (α = 0%, 3% and 5%) were considered in this investigation.

The main conclusions of this work focused on alerting structural engineers to the possible distortions associated with the steel-concrete composite bridge dynamic structural response, when subjected to dynamic actions due to vehicle convoys traffic on the irregular pavement surface, considering the progressive deterioration along the time. This way, the following conclusions can be drawn from the results presented in this study:

1. The vehicles velocities affect the bridge dynamic structural response (displacements and stresses values). In all investigated loading situations, the structure dynamic response was modified when the vehicle velocity was changed.
2. The vehicles traffic configurations present significant influence on the bridge displacements and stresses values. These positions vary according to the vehicles transverse location and spacing between vehicles on the bridge deck. It must be emphasized that these vehicles convoy's configurations are directly related to the dynamic excitation frequency (vehicles convoy passage on the bridge), which can induce a resonance condition.
3. The road-roughness condition directly influences the investigated steel-concrete composite highway bridge dynamic structural response. It must be emphasized that over time, as the bridge deck pavement deterioration process increases, the calculated vertical translational displacements and stresses values are higher than the bridge pavement is found without deterioration (t = 0).
4. Based on the traffic rates increase ($\alpha = 0\%$ to $\alpha = 5\%$), after 15 years of the bridge road pavement deterioration, the displacement and stresses values exceed up to four times when compared to those calculated when the bridge pavement is assumed without deterioration (t = 0). The results obtained in this paper clearly indicate the relevance of considering the bridge pavement progressive deterioration effect, when the dynamic structural behaviour is investigated.

Finally, it must be emphasized that the results obtained in the present investigation are associated mainly for vehicles convoys moving over irregular pavement surfaces of steel-concrete composite highway bridge decks, considering the pavement progressive deterioration effect. However, the analysis methodology presented in this paper is completely general and is the author's intention to sophisticate the vehicles convoy mathematical modelling, based on the use of different vehicles models with different dynamic properties, and apply this solution strategy considering other highway bridge structural systems.

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