Fatigue assessment of steel-concrete composite highway bridges considering a progressive pavement deterioration model

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Abstract. Highway bridges are subjected to random traffic loads with relevant impact dynamic loadings along all their service life. The road-roughness of asphalt pavements represents a key issue to the significant decrease of the highway bridge decks service life. Having this context in mind, this article aims to develop an analysis methodology to assess the fatigue performance of highway bridges, including the dynamic actions due to vehicles and the effect of the progressive deterioration of the pavement, taking into account the road surface damages. The developed methodology is based on a linear cumulative damage rule, and the use of the Rainflow-counting algorithm and S-N curves from main design codes. The investigated structural model corresponds to a steel-concrete composite highway bridge deck, with straight axis, simple supported and spanning 13.0m by 40.0m. In this work, 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 results of a parametric analysis are presented aiming to verify the extension of the dynamical effects on the service life of highway bridges due to vehicles crossing on the irregular pavement surface.

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

1 Introduction

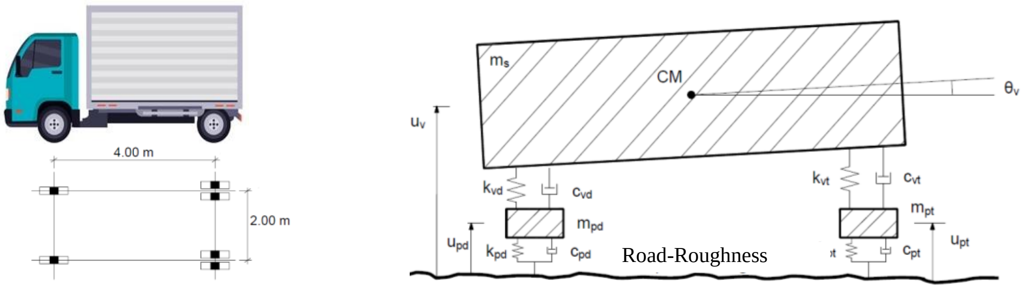
During the life cycle of a bridge, dynamic impacts due to random traffic loads and deteriorated road surface conditions can induce significant increase of the displacements and stresses values. These dynamic actions can generate the nucleation of fractures or even their propagation on the bridge deck structure. Especially in regions where road maintenance is not effective, this problem is substantial, causing premature deterioration of the bridge's superstructure and pavement [1].

The significant increase associated to the vehicle’s weight and volume of traffic currently on highway bridges has made these structures more subject to various degradation phenomena. Over time, these phenomena manifest themselves through the appearance of physical signs, such as cracking. Fatigue is one of these progressive degradation events induced by variations in stress due to road traffic action in bridge’s structure [2].

The proposed analysis methodology evaluates the fatigue performance of 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 developed methodology is based on a linear cumulative damage rule and Rainflow counting methods are used to calculate the numbers and magnitudes of the stress ranges. The main conclusions of this study focused on alerting structural engineers to the real possibility of fatigue damage increase, associated to the bridge dynamic structural response, when subjected to dynamic actions due to vehicles convoys on the irregular pavement surface.

2 Mathematical modelling of the vehicles

The truck used in this work is presented in Fig. 1a, being one of the most common vehicles in the local roads of Brazil. The two-axle truck structural-mechanical model is shown in Fig. 1b and presents 4 degrees of freedom. The geometry, mass distribution, damping, and stiffness are listed in Tab. 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 (2 axles) [3]

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 progressive deterioration for road surface

Road surface roughness is generally defined as an expression of irregularities on the road surface, and it is the primary factor affecting the dynamic response of both vehicles and bridges, see Silva and Roehl [4]. Based on the studies carried out by Dodds and Robson [5], the road surface roughness was assumed as a zero-mean stationary Gaussian random process and it could 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 [5], as presented in eq. (2):

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

Where Ω = spatial frequency of the pavement harmonic i (cycles/m); Ω_0 = discontinuity frequency of $1/2\pi$; and $G_d(\Omega_0)_t$ = road roughness coefficient (m^3/cycle), also called RRC, used by the International Organization for Standardization [6] to define the road-roughness classification, and the ranges are listed in Tab. 2.

Table 2. RRC values for road-roughness classification [6]

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 order to consider the road surface damages because of loads or corrosions, a progressive deterioration model for the road-roughness is necessary when generating the random road profiles. Paterson and Attoh-Okine [7] have developed such model considering the International Roughness Index (IRI) with the values at any time after the road surface service (IRI_t).

The IRI was developed in 1986 and is used to define the longitudinal profile of a travelled wheel track [8]. Various correlations have been developed between the indices RRC and IRI [9, 10]. Based on the corresponding ranges of the road-roughness coefficient and the IRI value [10], a relationship between the IRI and the RRC is utilized in the present study, as presented in eq. (3) and IRI_t being calculated using eq. (4):

$$RRC_t = G_d(\Omega_0)_t = 6.1972\ 10^{-9}\ e^{IRI_t/0.42808} + 2\ 10^{-6} \tag{3}$$

$$IRI_t = 1{,}04e^{\eta t}\ IRI_0 + 263\ (1+SNC)^{-5}\ (CESAL)_t \tag{4}$$

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, this value was adopted equal to 0.90 m/km. The environmental coefficient (η) varies from 0.01 to 0.7 and depends on usually adopted dry/wet, freezing/non-freezing conditions, equal to 0.10 for bridges exposed in 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 (5) was used to estimate the number of traffic in terms of AASHTO 80-kN (18-kip):

$$(CESAL)_t = f_d\ n_{tr}(t)\ F_{Ei}\ 10^{-6} \tag{5}$$

Where f_d = design lane factor; $n_{tr}(t)$ = cumulated number of truck passages for the future year t, estimated using eq. (6); and F_{Ei} = load equivalency factor for axle category i, calculated following strictly the rules of AASHTO Guide for Design of Pavement Structures [11].

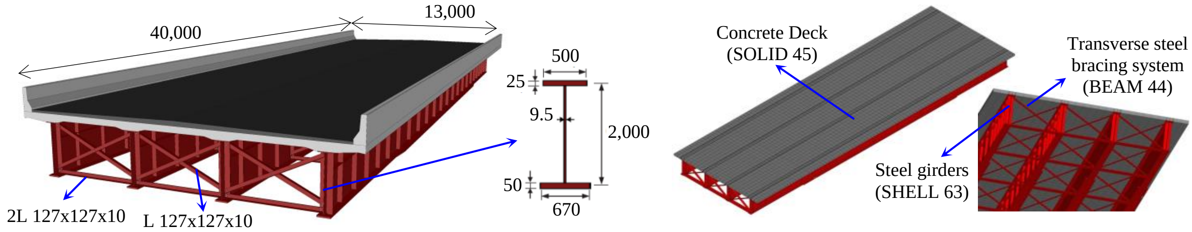
CESAL changes in consequence of the yearly traffic increase, also resulting in a change of the progressive deterioration function. Kwon and Frangopol [12] based on the ADTT and traffic increase rate per year, estimated the cumulated number of truck passages for the future year t using eq. (6):

$$n_{tr}(t) = N_{obs}\left[\frac{(1+\alpha)^t - 1}{\ln(1+\alpha)}\right] \tag{6}$$

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 [13]; and α = traffic increase rate per year, adopted in this investigation is equal to 3% and 5%.

4 Investigated highway bridge and finite element model

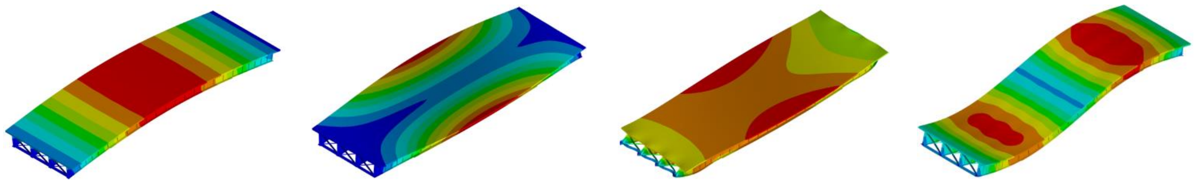
The investigated structural model corresponds to a typical steel-concrete composite highway bridge deck, with straight axis, simple supported, and spanning 13.0m by 40.0m. The structural system is constituted by four composite girders and a 0.225m thick concrete slab, see Fig. 2. The steel sections considered 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 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 EUROCODE 1[13] 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

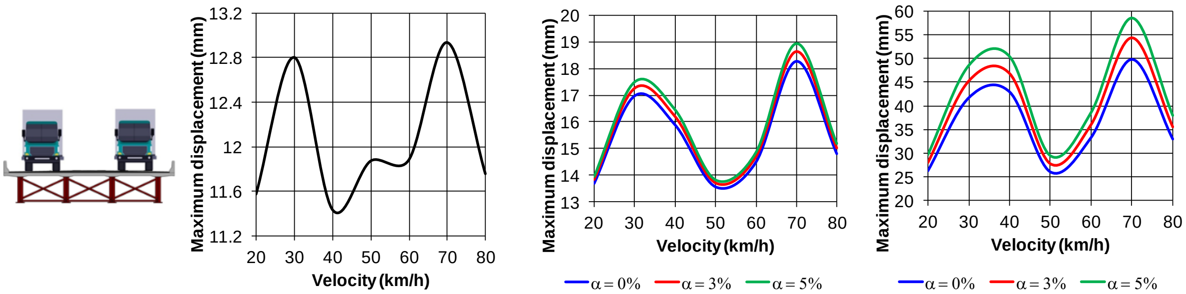
Figure 3. Main global vibration modes of the investigated bridge obtained using the finite element modelling

5 Fatigue assessment

Variable stress ranges from dynamic vehicle loads can induce fatigue damage accumulations at certain bridge components and accelerate the road surface deterioration in bridges' life cycle. The interactions of the road surface deterioration and dynamic vehicle loads might accelerate the fatigue damage accumulations and lead to serious fatigue failures when such damages increase to a certain limit [14]. In this context, 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. Thus, it is necessary and more realistic to consider the influence of the progressive degradation of the road surface roughness.

The road-roughness classification is defined in accordance with ISO 8608 [6], see Tab. 2. Based on the RRC, calculated from eq. (3), 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, in 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. 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 (Fig. 4).

In order to extend the study of the dynamic behavior of the structure to different traffic conditions, the vehicles convoys were positioned, separately, in central lane, in lateral lane and in two lateral lanes. The speed parameter of vehicles convoy varies from 20 to 80 km/h, in 10 km/h intervals, resulting in 7 different speeds, to evaluate the project response spectra, for each of the traffic conditions. It can be observed that the situation with two lateral lanes of trafficking led to the highest displacement results. Therefore, this case was chosen for fatigue damage assessment approach. It was possible to build seven displacement spectra from the maximum displacements obtained for each speed for this case (two lateral lanes), see Fig. 4.



a) Without deterioration (t=0)   b) t = 11 years   c) t = 15 years

Figure 4. Response spectra: translational vertical displacement values

In the spectra for a scenario without deterioration [Fig. 4a], 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 equals to 1.30 Hz (f = 70/3.6/15) due to the mobility between single axles of two consecutive vehicles, spaced 15 m, able to vibrate in the second harmonic (2.60 Hz) the fundamental frequency of structure (f_{01} = 2.97 Hz).

However, it can be observed that for t = 15 years [Fig. 4c] 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.

Fatigue, due to an accumulation of damage, is one of the main forms of deterioration for structures and can be a typical failure mode. During the life cycle of a bridge, variable stress range cycles due to multiple random dynamic loads might lead to fatigue damage accumulations at structure's details. Due to the progressive deteriorations and accumulated fatigue damages under dynamic vehicle loads, it is essential to ensure the structure's safety [14].

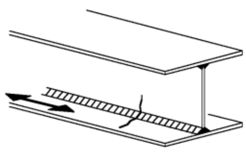
This way, according to Tab. 3, which indicates values of daily traffic average (MDT) and number of cycles according to the type of road, a number of cycles of 2 million per year will be considered. The proposed analyses are carried out for the convoy shown in Fig. 4, considering t = 15 years, α = 5% and υ = 70 km/h.

Table 3. Average daily traffic and number of cycles [15]

Type of road	Case	MDT	Number of cycles
Express highways, secondary highways, roads and streets	I	2.500 or more	2.000.000
Express highways, secondary highways, roads and streets	II	Less than 2.500	500.000
Other highways, roads and streets not included in cases I or II	III	-	100.000

Fatigue assessment of steel structures in current steel standards is based on the SN curves approach, with typical structural details organized into different categories. Each detail category is represented by the corresponding SN curve, where the fatigue strength, $\Delta\sigma$, is a function of the number of cycles, N_i. The structural detail analysed in this work is in accordance with EUROCODE 3 [16] and is represented in Tab. 4.

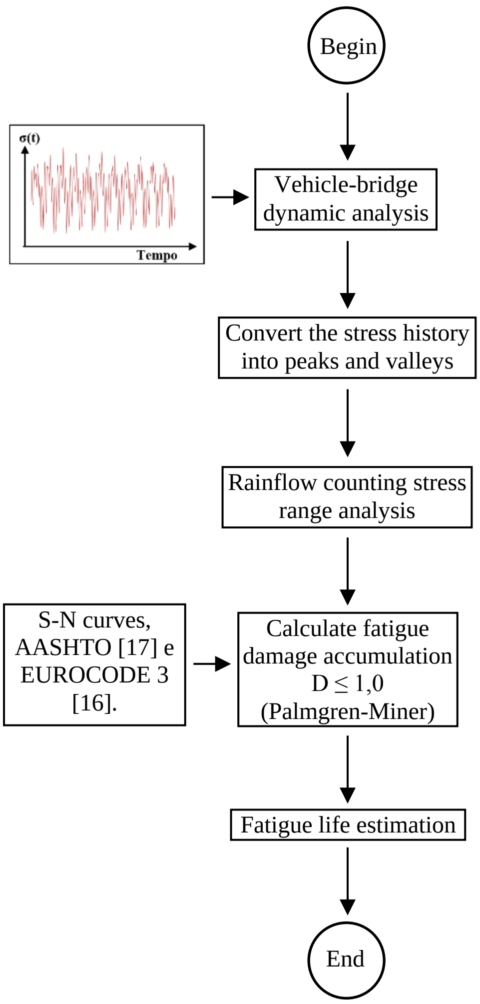
Table 4. Description of the analysed structural detail

Description	Structural detail	Detail position
Detail I: automatic or fully mechanical butt-welding without interruption performed on both sides.		Beams V1-V4. Connection between the web and the bottom flange. Bridge central section.

Fatigue life estimations based on Palmgren-Miner's rule were performed considering Detail I (Tab. 4). The fatigue damage is then calculated considering surface deterioration over time based on the increase of the traffic rates of 5% and t = 15 years. Table 5 shows the fatigue damage and the calculated fatigue life estimation in years, respectively, for the analysed detail. Under these conditions, the calculated fatigue life was equal to 30.09 years and 29.64 years, respectively, when AASTHO [17] and EUROCODE 3 [16] recommendations were used (see Tab. 5). This service life can be seen as a measure of roadway bridge's fatigue life estimation since its opening without any safety factor. A good convergence was observed between the results of these methods. However, it is realized that for a situation without deterioration (α = 0% and t = 0) the calculated fatigue life was equal to 102.69 years and 101.16 years, respectively, when were used AASTHO [17] and EUROCODE 3 [16] methodologies.

Table 5. Fatigue assessment of structural Detail I

$\Delta\sigma$ (N/mm^2)	AASTHO [17] (Class B) Limit: 75 years		EUROCODE 3 [16] (Class 125) Limit: 120 years		Flowchart
	N_i	n_i/N_i	N_i	n_i/N_i	
1	3.61E+11	1.21E-08	3.56E+11	1.23E-08	
2	4.51E+10	5.86E-08	4.45E+10	5.95E-08	
3	1.34E+10	8.60E-08	1.32E+10	8.73E-08	
6	1.67E+09	1.38E-07	1.65E+09	1.40E-07	
8	7.05E+08	3.26E-07	6.95E+08	3.31E-07	
7	1.05E+09	2.18E-07	1.04E+09	2.22E-07	
11	2.71E+08	8.48E-07	2.67E+08	8.60E-07	
12	2.09E+08	5.50E-07	2.06E+08	5.59E-07	
16	8.81E+07	2.61E-06	8.68E+07	2.65E-06	
21	3.90E+07	5.90E-06	3.84E+07	5.99E-06	
22	3.39E+07	3.39E-06	3.34E+07	3.44E-06	
30	1.34E+07	1.72E-05	1.32E+07	1.75E-05	
34	9.18E+06	5.01E-05	9.05E+06	5.08E-05	
37	7.13E+06	3.23E-05	7.02E+06	3.27E-05	
41	5.24E+06	4.39E-05	5.16E+06	4.46E-05	
43	4.54E+06	5.06E-05	4.47E+06	5.14E-05	
45	3.96E+06	2.90E-05	3.90E+06	2.95E-05	
48	3.26E+06	7.04E-05	3.22E+06	7.15E-05	
52	2.57E+06	8.95E-05	2.53E+06	9.09E-05	
54	2.29E+06	1.00E-04	2.26E+06	1.02E-04	
57	1.95E+06	1.18E-04	1.92E+06	1.20E-04	
58	1.85E+06	1.24E-04	1.82E+06	1.26E-04	
61	1.59E+06	7.23E-05	1.57E+06	7.34E-05	
63	1.44E+06	1.59E-04	1.42E+06	1.62E-04	
66	1.26E+06	1.83E-04	1.24E+06	1.86E-04	
70	1.05E+06	2.18E-04	1.04E+06	2.22E-04	
73	9.28E+05	7.43E-04	9.14E+05	7.54E-04	
75	8.56E+05	2.69E-04	8.43E+05	2.73E-04	
78	7.61E+05	3.02E-04	7.49E+05	3.07E-04	
79	7.32E+05	3.14E-04	7.21E+05	3.19E-04	
80	7.05E+05	3.26E-04	6.95E+05	3.31E-04	
81	6.79E+05	3.38E-04	6.69E+05	3.44E-04	
83	6.31E+05	3.64E-04	6.22E+05	3.70E-04	
84	6.09E+05	1.89E-04	6.00E+05	1.92E-04	
87	5.48E+05	2.10E-04	5.40E+05	2.13E-04	
92	4.64E+05	2.48E-04	4.57E+05	2.52E-04	
94	4.35E+05	5.29E-04	4.28E+05	5.37E-04	
97	3.96E+05	5.81E-04	3.90E+05	5.90E-04	
101	3.50E+05	3.28E-04	3.45E+05	3.33E-04	
104	3.21E+05	7.16E-04	3.16E+05	7.27E-04	
106	3.03E+05	7.58E-04	2.99E+05	7.70E-04	
107	2.95E+05	7.80E-04	2.90E+05	7.92E-04	
110	2.71E+05	2.54E-03	2.67E+05	2.58E-03	
115	2.37E+05	4.84E-04	2.34E+05	4.92E-04	
135	1.47E+05	1.57E-03	1.45E+05	1.59E-03	
138	1.37E+05	3.35E-03	1.35E+05	3.40E-03	
156	9.51E+04	1.69E-02	9.37E+04	1.72E-02	
$\Delta\sigma_{máx}=156$	$D_i=\sum_{i=1}^{k}\frac{n_i}{N_i}$	3.32E-02	$D_i=\sum_{i=1}^{k}\frac{n_i}{N_i}$	3.37E-02	
	T=1/D (years)	**30.09**	T=1/D (years)	**29.64**	



6 Conclusions

In this study, a fatigue assessment was carried out for a structural detail of a steel-concrete composite highway bridge considering the traffic of vehicles with velocity of 70 km/h and a poor road condition [α = 5% and t = 15 years], having in mind the AASTHO [17] and EUROCODE 3 [16] recommendations. This way, the following conclusions can be drawn from the results presented in this investigation:

1. The road-roughness condition affects directly the dynamic structural response of the steel-concrete composite highway bridge and this dynamical effect influences the service life of highway bridges.
2. Over time, the more deteriorated road condition induces a larger vertical translational displacement, which leads to a bigger stress values. Thus, more deteriorated road condition induces shorter fatigue life.
3. Based on the performed analysis (α = 5% and t = 15 years), it should be emphasized that the fatigue life values were considerably lower, representing a relevant decrease of up to three times when compared to the situation without deterioration of the road condition (α = 0% and t = 0).
4. Under these conditions, considering α = 5% and t = 15 years, the calculated bridge fatigue life was equal to 30.09 years and 29.64 years, respectively, when AASTHO [17] and EUROCODE 3 [16] methodologies were used. On the other hand, for a situation without deterioration (α = 0% and t = 0) the calculated fatigue life was equal to 102.69 years and 101.16 years, respectively. This way, it can be observed that it is very relevant to consider the influence of the progressive degradation of the road surface roughness.

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