XLIII CILAMCE
21-25, November, 2022
FOZ DO IGUAÇU - PR - BRAZIL

Influence of the progressive pavement deterioration on the steel-concrete composite highway bridges service life

Ana Célia S. da Silva[1], Guilherme S. Alencar[1], José Guilherme S. da Silva[1]

[1]*Civil Engineering Postgraduate Programme (PGECIV), State University of Rio de Janeiro (UERJ)*
São Francisco Xavier St., 524, Maracanã, 20550-900, Rio de Janeiro/RJ, Brazil
anaceliasoares.eng@gmail.com, guilhermesalencar@gmail.com, jgss@uerj.br

Abstract. Nowadays, the significant increase associated to the vehicle's weight and traffic volume on the highway bridge decks has made these structures subjected to several degradation phenomena. In this context, structural fatigue is one of these progressive degradation events induced by vehicles dynamic impacts that can produce significant increasing of the stress values. Having these ideas in mind, this research work aims is to develop an analysis methodology to assess the fatigue performance of steel-concrete composite highway bridges, including the dynamic actions due to vehicles convoys and the pavement progressive deterioration effect. The developed analysis methodology is based on the use of Miner-Palmgren linear cumulative damage rule, Rainflow algorithm and S-N curves associated to the traditional design codes. This way, the investigated structural model corresponds to a steel-concrete composite highway bridge spanning 40 m subjected to vehicles traffic. The developed numerical model adopted the usual mesh refinement techniques present in Finite Element Method (FEM) simulations implemented in the ANSYS program. The main conclusions of this investigation focused on verify the extension of the dynamical effects on the service life of steel-concrete composite highway bridges due to vehicles crossing on the deteriorated pavement surface.

Keywords: dynamic analysis, progressive degradation, fatigue assessment.

1 Introduction

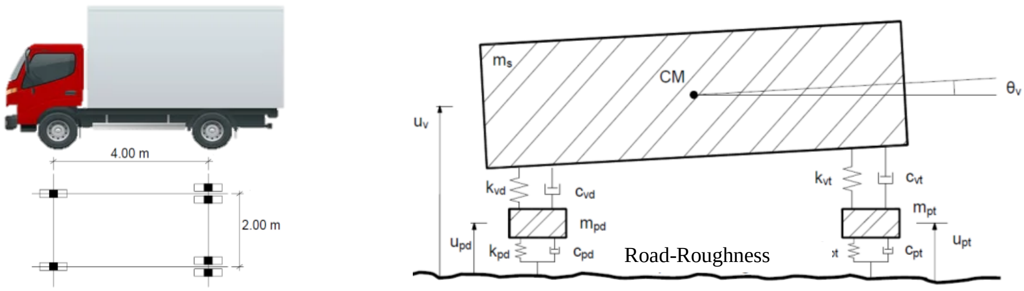
Dynamic impacts from random traffic loads and deteriorated road surface conditions can generate the nucleation of fractures or even their propagation on the bridge deck structure. This is a substantial problem, especially in regions where road maintenance is not effective, causing premature deterioration of the bridge's superstructure and pavement [1]. In this context, several researchers [1-4] attested that the effects due to the dynamic interaction between the vehicle wheels and the irregular pavement surface can be much more important than those produced by the vehicles smooth movement.

This way, having these thoughts in mind, 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.

The bridge dynamic structural response was investigated through an extensive parametric study based on the calculated displacements and stresses values. Thus, fatigue assessment was performed 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 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.

2 Vehicles mathematical modelling

The truck adopted in this work is presented in Fig. 1(a), and is one of the most common vehicles used in Brazilian roads [1,4]. The developed two-axle truck structural-mechanical mathematical model is shown in Fig. 1(b). The vehicle dynamic properties were determined based on experimental tests [5] and 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 2C vehicle: 2 axles [5]

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 Pavement progressive deterioration effect modelling

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]. Based on the studies carried out by Dodds and Robson [6], 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).

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

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 presented in eq. (2); and Ω_i = wave number (cycles/m).

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

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 (see Tab. 2).

Table 2. Average values of $G_d(\Omega_0)$ in cm^3 for different levels of road roughness quality [7]

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

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 [8] 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 eq. (3).

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

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 is modified from one region to another depending on the specifications adopted in each country for road constructions. In this work the adopted value 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. To estimate the traffic number in terms of AASHTO 80-kN (18-kip), the eq. (4) was used.

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

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 the rules of AASHTO Guide for Design of Pavement Structures [9]. Due to the yearly traffic increase, the CESAL parameter is modified resulting in a change of the progressive deterioration function. Kwon and Frangopol [10], 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 eq. (5).

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

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 [7]; and α = traffic increase rate per year. In this research work, α was adopted equal to 3% and 5%.

The IRI formulation was developed in 1986 and is used to define the longitudinal profile of a travelled wheel track [11]. 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. [12], 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 [13] used RRC to define the road-roughness classification, and the ranges are listed in Tab. 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 [14,15]. Based on the corresponding ranges of the road-roughness coefficient and the IRI value [15], a relationship between IRI and RRC is used in the present study as shown in eq. (6).

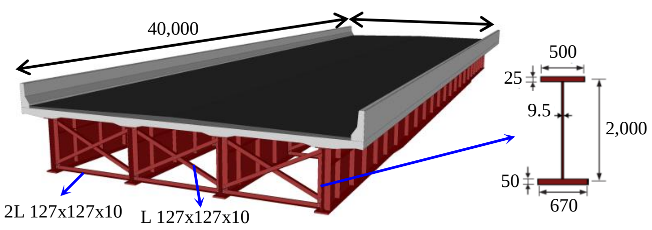
$$RRC_t = G_d(\Omega_0)_t = 6.1972 \times 10^{-9} \times e^{\frac{IRI_t}{0.42808}} + 2 \times 10^{-6} \tag{6}$$

Table 3. RRC values for road-roughness classification [13]

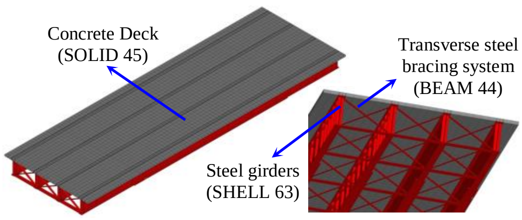
Road-roughness classification	Ranges for RRCs
Very good	2×10^{-6} to 8×10^{-6}
Good	8×10^{-6} to 32×10^{-6}
Average	32×10^{-6} to 128×10^{-6}
Poor	128×10^{-6} to 512×10^{-6}
Very poor	512×10^{-6} to 2048×10^{-6}

4 Investigated highway bridge and finite element model

In this work, the bridge structural model corresponds to a typical simply supported steel-concrete bridge deck with straight axis, spanning 13 m by 40 m and with a 0.225 m thick concrete slab (Fig. 2). 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 2.0×10^5 MPa Young's modulus with 0.3 Poisson's ratio was adopted for the steel girders. The concrete slab has 25 MPa compression strength with 3.05×10^4 MPa of Young's modulus and 0.2 Poisson's ratio.



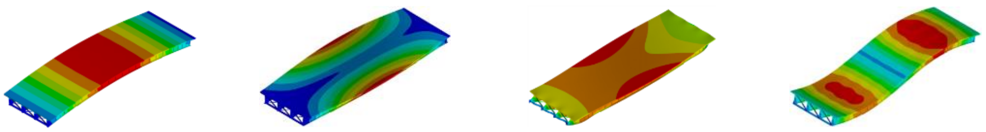
a) Structure design (units in millimetres)



b) Finite element model of the bridge

Figure 2. Investigated simply supported steel-concrete highway bridge

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 [16] computational program, see Fig. 2(b). 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 investigated steel-concrete composite bridge natural frequencies and vibration modes have been determined based on numerical methods of extraction (modal analysis), through a free vibration analysis (see Fig. 3).



a) f_{01} = 2.97 Hz: longitudinal bending

b) f_{02} = 3.67 Hz: axial torsion

c) f_{03} = 6.28 Hz: beam's lateral bending

d) f_{04} = 9.67 Hz: longitudinal bending

Figure 3. Investigated bridge global vibration modes [16]

5 Fatigue assessment

Initially, it is important to point out that 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 [17]. 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 [13], Tab. 3. Based on the RRC, eq. (6), three traffic increase rates were investigated (α = 0%, 3% and 5%) in a 15-year period. The road condition in the first nine years was classified as very good, in the tenth year as good, in the eleventh as average, in the twelfth as average to the traffic increase rate at 0% and poor to the traffic increase rates of 3% and 5%, and from thirteenth to fifteenth as poor. It is noteworthy that the following results are only for situation without deterioration (t = 0) and for t = 15 years.

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 and in lateral lane. The speed parameter of vehicles convoy varies from 20 to 80 km/h, in 10 km/h intervals, resulting in seven different speeds, to evaluate the project response spectra, for each of the traffic conditions. It should be noted that the situation with vehicles convoys in central lane was chosen for fatigue damage assessment approach.

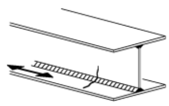
During the life cycle of a bridge, variable stress range cycles due to multiple random dynamic loads might lead to fatigue damage accumulations at structure's details. Due to the progressive deteriorations and accumulated fatigue damages under dynamic vehicle loads, it is essential to ensure the structure's safety [17]. This way, according to Tab. 4, 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.

Table 4. Average daily traffic and number of cycles

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 represented in Tab. 5. In sequence, Fig. 4 presents a general flowchart for the analysis methodology used to assess the investigated highway bridge fatigue performance.

Table 5. 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 G1-G4. Connection between the web and the bottom flange. Bridge central section.

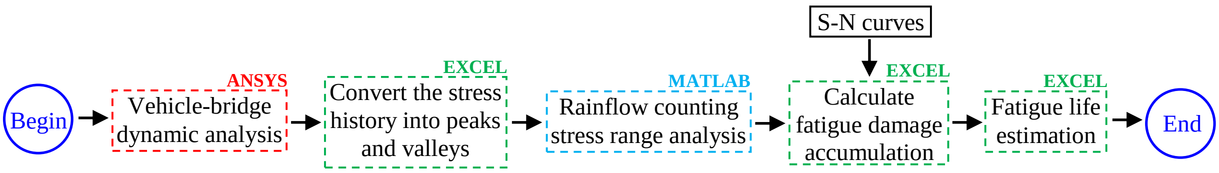
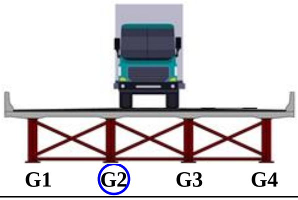


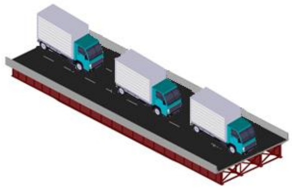
Figure 4. Flowchart: fatigue assessment

Fatigue life estimations based on Palmgren-Miner's rule were performed considering Detail I (Tab. 5). The fatigue damage is then calculated considering four different surface conditions: without deterioration (Case 1) and fifteen years of progressive deterioration (t = 15 years) with traffic increase rates of 0%, 3% and 5% (Cases 2, 3 and 4). The speed parameter of vehicles convoy varies from 20 to 80 km/h, in 10 km/h intervals, resulting in seven different speeds. Table 6 shows the calculated fatigue life estimation in years for the analysed detail considering not only the four different surface conditions, but also the seven different speeds.

Under these conditions, it is possible to observe in Tab. 6 the presence of two worst cases associated with speeds of 70 km/h and 30 km/h. The first case (70 km/h) is associated with crossing frequencies equals to 1.30 Hz able to vibrate in the second harmonic (2.60 Hz) the fundamental frequency of structure (f_{01} = 2.97 Hz). On the other hand, the second case (30 km/h) vibrates the fundamental frequency of the structure in the fourth harmonic.

Table 6. Fatigue assessment of structural detail I





Speed	Fatigue life estimation (years)	AASTHO [18] Class B	EUROCODE 3 [19] Class 125	NBR 8800 [20] Class B
20 km/h	Without deterioration (t = 0)	188111	190995	187076
	α = 0%; t = 15 years	261.25	265.26	258.10
	α = 3%; t = 15 years	207.31	210.49	204.75
	α = 5%; t = 15 years	163.75	166.26	161.68
30 km/h	Without deterioration (t = 0)	60751	61682	60374
	α = 0%; t = 15 years	44.23	44.89	43.78
	α = 3%; t = 15 years	34.41	34.99	33.26
	α = 5%; t = 15 years	30.62	31.41	29.63
40 km/h	Sem deterioração (t = 0)	272518	276696	271329
	α = 0%; t = 15 years	281.22	285.53	277.88
	α = 3%; t = 15 years	181.04	183.82	178.83
	α = 5%; t = 15 years	160.82	163.28	158.81
50 km/h	Without deterioration (t = 0)	158946	161383	158116
	α = 0%; t = 15 years	1062.51	1078.80	1050.99
	α = 3%; t = 15 years	730.43	741.63	722.28
	α = 5%; t = 15 years	571.46	580.22	564.93
60 km/h	Without deterioration (t = 0)	142162	144341	141385
	α = 0%; t = 15 years	181.26	184.04	179.12
	α = 3%; t = 15 years	133.10	135.14	131.47
	α = 5%; t = 15 years	103.70	105.29	102.41
70 km/h	Without deterioration (t = 0)	37930	38511	37675
	α = 0%; t = 15 years	27.45	27.98	25.28
	α = 3%; t = 15 years	22.14	23.71	21.18
	α = 5%; t = 15 years	20.61	21.93	19.39
80 km/h	Without deterioration (t = 0)	69415	70479	69036
	α = 0%; t = 15 years	274.71	278.92	271.44
	α = 3%; t = 15 years	195.87	198.87	193.46
	α = 5%; t = 15 years	155.92	158.31	153.98

A good convergence was observed between the results of these three methods: AASTHO [18], EUROCODE 3 [19] and NBR 8800 [20]. In the worst case scenario (70 km/h, α = 5%, t =15 years), the calculated fatigue life was 20.61 years (see Tab. 6). However, it is realized that for a situation without deterioration (α = 0% and t = 0) the calculated fatigue life was much longer (infinite). These results indicate the importance of considering the road pavement progressive deterioration effect on bridge fatigue analysis.

6 Conclusions

In this investigation, a fatigue assessment was carried out based on AASTHO [18], EUROCODE 3 [19] and NBR 8800 [20] recommendations for a structural detail of a steel-concrete composite highway bridge. In this context, seven different vehicle traffic speeds and three different traffic increase rates (α = 0%, 3% and 5% for t = 15 years) were considered. This way, the following conclusions can be drawn from the results presented in this work:

1. The dynamic structural response is directly affected not only by the road surface roughness, but also by the convoys position. Thus, this dynamical effect influences the fatigue life estimation of steel-concrete composite highway bridges.

2. Moreover, it can be observed that the convoys speed also influences the stress values and, consequently, modify the fatigue life estimation. In the case under study, two worst cases were identified: one associated with the speed of 70 km/h and another one associated with the speed of 30 km/h.
3. Over time, the more deteriorated road condition induces larger vertical translational displacements, which leads to bigger stress values. This way, more deteriorated road condition induces shorter fatigue life.
4. The results demonstrated, for all studied cases, that the fatigue life estimates when progressive deterioration was considered (α = 5%; t = 15 years) were considerably lower when compared to the situation without deterioration (α = 0%; t = 0).
5. Under these conditions, considering the worst case (α = 5%, t = 15 years, ν = 70 km/h) the calculated bridge fatigue life was equal to 20.61, 21.93 and 19.39 years, respectively, when AASTHO [18], EUROCODE 3 [19] and NBR 8800 [20] methodologies were used. On the other hand, for a situation without deterioration (α = 0% and t = 0) the calculated fatigue life was much longer (infinite). Therefore, it can be observed that considering progressive pavement deterioration is really relevant for bridge fatigue analysis.

Acknowledgements. The authors gratefully acknowledge the financial support for this research work provided by the Brazilian Science Foundation's CNPq, CAPES and FAPERJ.

Authorship statement. The authors hereby confirm that they are the sole liable persons responsible for the authorship of this work, and that all material that has been herein included as part of the present paper is either the property (and authorship) of the authors, or has the permission of the owners to be included here.

References

[1] A. C. S. Silva. *Dynamic Analysis of Steel-Concrete Composite Highway Bridge Decks Considering the Effect of the Progressive Degradation of the Road Surface.* MSc dissertation (In Portuguese), State University of Rio de Janeiro, 2020.
[2] J. G. S. da Silva, "Dynamical performance of highway bridge decks with irregular pavement surface*". Computer and Structures*, vol. 82, pp. 871–881, 2004.
[3] W. Zhang and C.S. Cai, "Fatigue reliability assessment for existing bridges considering vehicle speed and road surface conditions". *Journal of Bridge Engineering*, vol. 17, pp. 443–453, 2012.
[4] G. S. Alencar. *Dynamic Analysis and Fatigue Assessment of Composite Highway Bridges Subjected to the Traffic of Vehicles Over the Irregular Pavement Surface.* MSc dissertation (In Portuguese), State University of Rio de Janeiro, 2015.
[5] R. C. Battista, M. S. Pfeil and E. M. L. Carvalho, "Fatigue life estimates for a slender orthotropic steel deck". Journal of Constructional Steel Research, vol. 64, n. 1, pp. 134–143, 2008.
[6] C. J. Dodds and J. D. Robson, "The description of road surface roughness". *Journal of Sound and Vibration*, vol. 31, n. 2, pp. 175–183, 1973.
[7] EUROCODE 1. *Actions on Structures-Part 2: Traffic Loads on Bridges*. European Committe for Standardization, 2003.
[8] W. D. O. Paterson and B. Attoh-Okine, "Simplified models of paved road deterioration based on HDM-III". *Annual Meeting Transportation Research Board*, vol. 1344, pp. 99–105, 1992.
[9] AASHTO. *Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, 1993.
[10] K. Kwon and D. M. Frangopol, "Bridge fatigue reliability assessment using probability density functions of equivalent stress range based on field monitoring data". *International Journal of Fatigue*, vol. 32, n. 8, pp. 1221–1232, 2010.
[11] M. W. Sayers and S. M. Karamihas. *The Little Book of Profiling: Basic Information about Measuring and Interpreting Road Profiles.* University of Michigan, 1998.
[12] M. W. Sayers, T. D. Gillespi and W. D. O. Paterson, "Guidelines for conducting and calibrating road roughness measurements". *World Bank Technical Paper: Technical Report WTP 46*, 1986.
[13] ISO 8608. *Mechanical Vibration-Road Surface Profiles-Reporting.* International Standard Organization, 1995.
[14] W. D. O. Paterson, "International roughness index: relationship to other measures of roughness and riding quality". *Transportation Research Record*, vol. 1084, pp. 49–59, 1986.
[15] A. Shiyab. *Optimum use of the Flexible Pavement Condition Indicators in Pavement Management System.* PhD thesis, Curtin University of Technology, 2007.
[16] ANSYS Swanson Analysis Systems Inc. Theory Reference (R-12.1), 2009.
[17] W. Zhang; C. S. Cai and F. Pan, "Nonlinear fatigue damage assessment of existing bridges considering progressively deteriorated road conditions". *Engineering Structures*, vol. 56, pp. 1922–1932, 2013.
[18] AASHTO. Bridge Design Specifications. American Association of State Highway and Transportation Officials, 2012
[19] EUROCODE 3. Design of Steel Structures - Part 1-9: Fatigue. European Committe for Standardization, 2003.
[20] NBR 8800. *Design of Steel Structures and Steel-Concrete Composite Structures of Buildings* (In Portuguese). Brazilian Association of Technical Standards, 2008.