

DYNAMIC ANALYSIS AND FATIGUE ASSESSMENT OF A STEEL- CONCRETE COMPOSITE HIGHWAY BRIDGE DECK UNDER TRAFFIC LOADING

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Keywords: Composite Highway Bridges, Dynamic Analysis, Fatigue Analysis, Finite Element Modelling, Fatigue Assessment, Service Life.

Abstract. *Steel and (steel-concrete) composite highway bridges are currently subjected to dynamic actions of variable magnitude due to convoy of vehicles crossing on the deck pavement. These dynamic actions can generate the nucleation of fractures or even their propagation on the bridge deck structure. Proper consideration of all of the aspects mentioned pointed our team to develop an analysis methodology with emphasis to evaluate the stresses through a dynamic analysis of highway bridge decks including the action of vehicles. In this work, the developed computational model adopted the usual mesh refinement techniques present in finite element method simulations implemented in the ANSYS program. The investigated highway bridge is constituted by four longitudinal composite girders and a concrete deck, spanning 40.0m by 13.5m. The analysis methodology and procedures presented in the design codes were applied to evaluate the fatigue of the bridge determining the service life of the structure. The main conclusions of this investigation focused on alerting structural engineers to the possible distortions, associated to the steel and composite bridge's service life when subjected to vehicle's dynamic actions.*

1 INTRODUCTION

Steel and steel-concrete composite highway bridges are usually subjected to dynamic actions of variable magnitude due to the actions of vehicles crossing on the deck pavement. Depending on the magnitude and intensity of these dynamic actions, these adverse effects may compromise the structural system response reliability that could also lead to a reduction of the expected bridge service life [1-5].

Proper consideration of all of the aspects earlier mentioned pointed our team to develop an analysis methodology with emphasis to evaluate the stresses through a dynamic analysis of highway bridge decks, including the action of vehicles crossing on the pavement surface [1-5].

This way, the present investigation utilises techniques for counting stress-cycles and for applying cumulative damage rules combined with S-N curves. The first steps of the composite highway bridge study involved an extensive literature review of the techniques used to define steel and composite bridges service life, a study of the theoretical aspects of fatigue in steel, and the recommended procedures present in steel and composite structural design codes [6-8].

The design codes recommend the adoption of the S-N curves associated with the Miner's damage rule to evaluate the steel and composite bridges fatigue and service life [7, 8]. These codes also recommend that the bridge structure designs should avoid local stress concentrations, to prevent possible fatigue points.

The investigated steel-concrete composite bridge has a roadway width of 12.50m and a concrete deck thickness of 0.23m, spanning 40.0m by 13.5m. The structural system consists of four longitudinal composite girders and a concrete deck [1, 2]. The computational model used in the composite bridge dynamic analysis, adopts the usual mesh refinement techniques present in finite element method simulations implemented in the ANSYS program [9].

The beam steel sections were simulated by three-dimensional beam and shell finite elements. The beam web was represented by shell finite elements. The top and bottom beam flanges and the longitudinal and vertical stiffeners were simulated by three-dimensional beam elements considering flexural and torsional effects. The bridge concrete slab was simulated by shell finite elements.

The proposed analysis methodology and the procedures presented in the design codes [7, 8] were used to evaluate the bridge fatigue response in terms of its structural service life. The main conclusions of this paper focused on alerting structural engineers to the possible distortions related to the bridge's service life when subjected to vehicle's dynamic actions.

2 STEEL-CONCRETE COMPOSITE BRIDGE MODEL

The structural model investigated in the present study corresponds to a steel-concrete composite highway bridge deck with straight axis, simple supported, spanning 13.0 m by 40.0 m [1, 2]. The structural system is constituted of four composite girders and a 0.23 m thick concrete slab, as shown in Figures 1 and 2.

The steel sections used were welded wide flanges (WWF) made with a 350 MPa yield stress steel grade and 485 MPa ultimate strength. A 2.05×10^5 MPa Young's modulus was adopted for the steel beams. The concrete slab possesses a 25 MPa specified compression strength and a 3.05×10^4 MPa Young's Modulus. Table 1 depicts the geometrical characteristics of the steel sections used in the structural model [1, 2], presented in Figures 1 and 2.

3 VEHICLE MATHEMATICAL MODEL

In this investigation, the adopted vehicle mathematical model was developed by Almeida [3-5]. This model is considered discrete and constituted by "mass-spring-damper" systems with three masses and a four freedom degree. The vehicle model was based on the NBR 7188

vehicle “TB-12” [6] and consists of a three masses model with four degrees of freedom (three translational and one rotational) and two axles to simulate each single vehicle, as presented in Figure 3. Translational vertical displacements and rotational displacements are considered in the vehicle model.

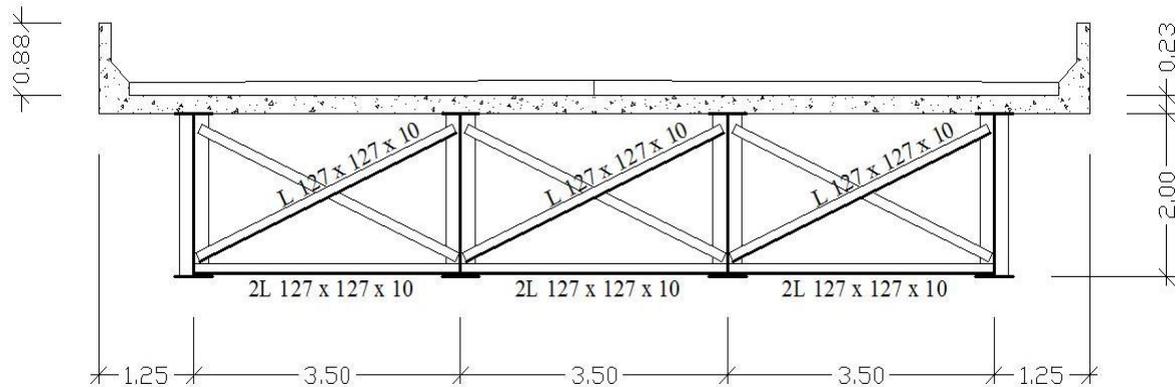


Figure 1: Bridge cross section.

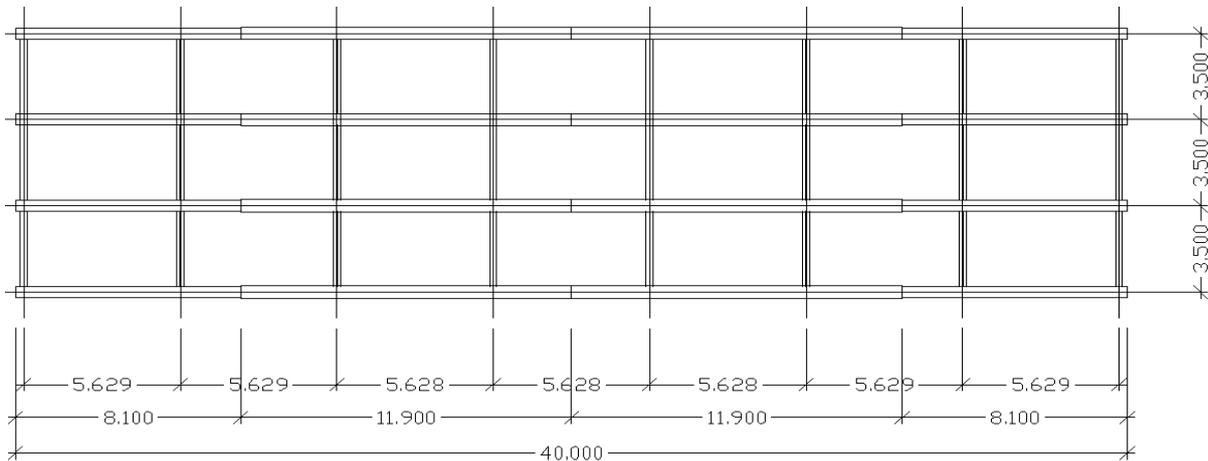


Figure 2: Bridge top view.

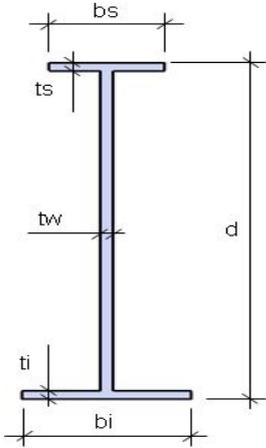
Cross Section	Extremity Cross Section: geometric properties (mm)	
	Height (d)	2000
	Top flange width (b_s)	450
	Top flange thickness (t_s)	25
	Bottom flange width (b_i)	450
	Bottom flange thickness (t_i)	50
	Web thickness (t_w)	9.5
	Central Cross Section: geometric properties (mm)	
	Height (d)	2000
	Top flange width (b_s)	500
	Top flange thickness (t_s)	25
	Bottom flange width (b_i)	670
	Bottom flange thickness (t_i)	50
	Web thickness (t_w)	9.5

Table 1: Geometrical characteristics of the beam steel sections.

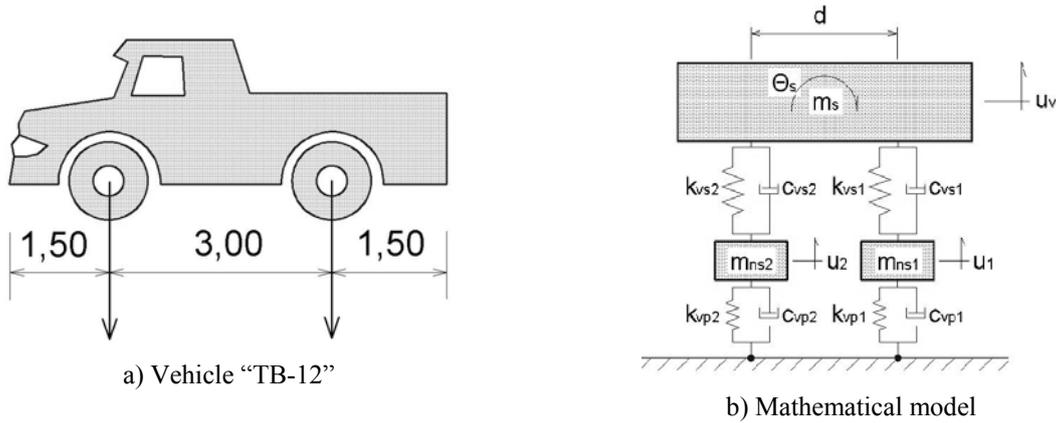


Figure 3: Vehicle model.

Figure 3 illustrates the vehicle model and uses the following notation: u_v , is the vehicle vertical displacement of the suspended mass; θ_s , is the vehicle rotational displacement related to the suspended mass; u_1 and u_2 are the vehicle vertical displacements of the non-suspended mass; m_s is the vehicle suspended mass; m_{ns} is the vehicle i^{th} non-suspended mass (for each axle), respectively; k_{vs} and k_{vp} are the i^{th} stiffness coefficients related to vehicle suspension and tires (for each axle), respectively; c_{vs} and c_{vp} are i^{th} the damping coefficients related to vehicle suspension and tires (for each axle), respectively.

The vehicle natural frequencies oscillating on rigid base (vertical motion), corresponding to the vehicle suspended mass (suspension system) and non-suspended mass degrees of freedom (tires), see Figure 3, were made equal to 3.0 Hz and 20.0 Hz, respectively [3-5]. However, the vehicle model has a lower natural frequency, associated to the vehicle suspended mass (suspension system) degree of freedom which is equal to 2.3 Hz (rotational motion) [3-5].

The relative damping coefficient of the vehicles vibration mode with predominant movement of the vehicles suspended mass is assumed to be $\xi = 0.1$ ($\xi = 10\%$) [3-5]. The total mass used in this vehicle model is equal to 12 t ($m_s = 10.667$ kg; $m_{ns1} = 666.5$ kg and $m_{ns2} = 666.5$ kg), corresponding to 120 kN weight. The relation between the suspended mass and non-suspended masses was considered equal the 8.0 [3-5].

The moving load is modelled by an infinite series of equal vehicles, regularly spaced, and running at constant velocity, v [3-5]. If l is the distance between two successive vehicles and as these cars enter one by one into the bridge deck, it is created a time repeated movement variation governed by the frequency, $f_t = v/l$, associated with the movement of the vehicles on the bridge, which can be called traversing frequency.

After a certain time period, t_1 , denominated crossing period, the first vehicle in the train reaches the far end of the bridge and from this instant, on the total mass of the vehicles on the bridge remains practically constant. Under these conditions the bridge will soon reach a steady-state response situation, which includes repetition of maximum values directly related to the fatigue and the service life of the structure [1-5].

4 FINITE ELEMENT MODEL

The computational model, developed for the composite bridge dynamic analysis, adopted the usual mesh refinement techniques present in finite element method simulations implemented in the ANSYS program [9].

The beam steel sections were simulated by three-dimensional beam and shell finite elements. The beam web thickness was represented by shell finite elements (SHELL63 [9]). The beam top and bottom flange and the longitudinal and vertical stiffeners were simulated by

three-dimensional beam elements (BEAM44 [9]), where flexural and torsion effects were considered. The bridge concrete slab was simulated by shell finite elements (SHELL63 [9]).

The computational model used rigid connections like “offsets” to guarantee the strain compatibility between plate elements (concrete slab) and three-dimensional beam elements (steel beams), simulating the composite highway bridge deck with full interaction.

The final computational model adopted used 4992 nodes, 2264 three-dimensional beam elements and 4324 shell elements, which resulted in a numeric model with 29952 degrees of freedom. Figure 4 illustrates the composite bridge finite element model. Figure 5 illustrates this strategy considering the Load Model I (LM-I, see Figure 7).

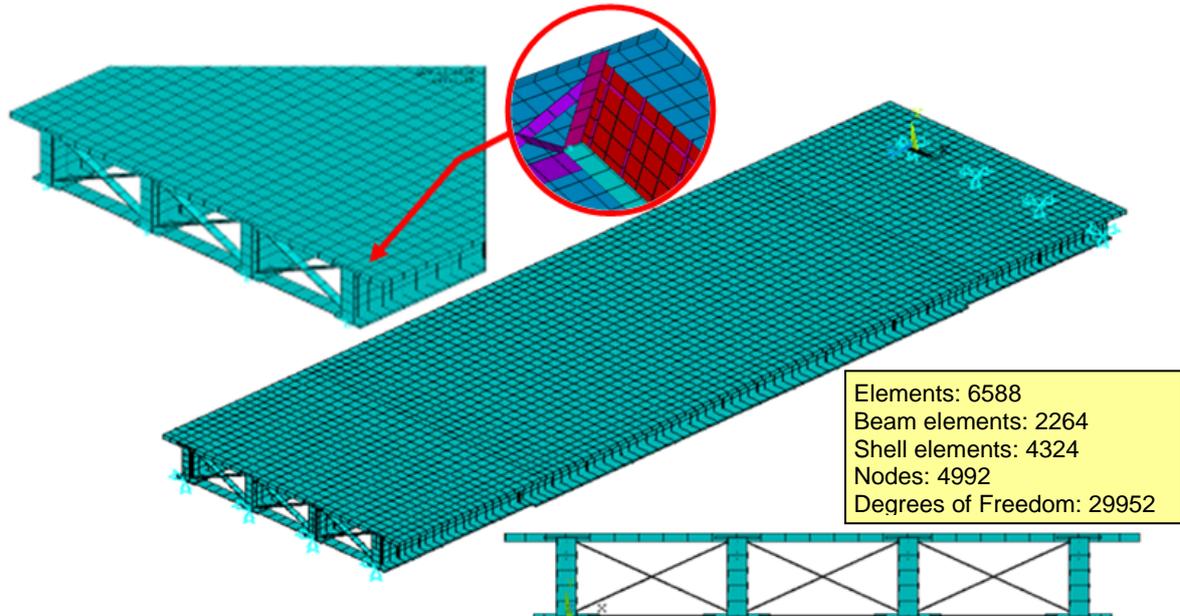


Figure 4: Composite (steel-concrete) highway bridge deck finite element model.

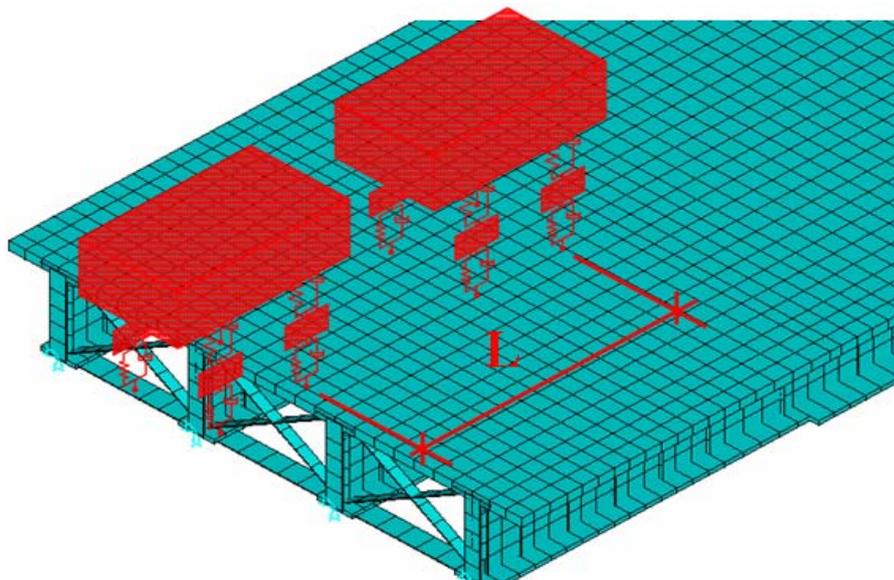


Figure 5: Vehicles crossing the investigated composite bridge.

5 DYNAMIC ANALYSIS

The composite bridge natural frequencies were determined with the aid of the numerical simulations, as presented in Table 2. The associated composite bridge vibration modes are shown in Figure 6. It can be clearly noticed from Table 2 results, that there is a very good agreement between the finite elements frequencies values and the frequencies obtained by Silva [3] and Murray *et al.* [10]. Such fact validates the numeric model here presented, as well as the results and conclusions obtained throughout this work.

Bridge natural frequencies: f_{0i} (Hz) [9]						f_{01} (Hz) Silva [3]	f_{01} (Hz) Murray <i>et al.</i> [10]
f_{01}	f_{02}	f_{03}	f_{04}	f_{05}	f_{06}		
2.90	3.64	6.87	9.63	11.03	12.85	2.85	2.65

Table 2: Composite bridge natural frequencies.

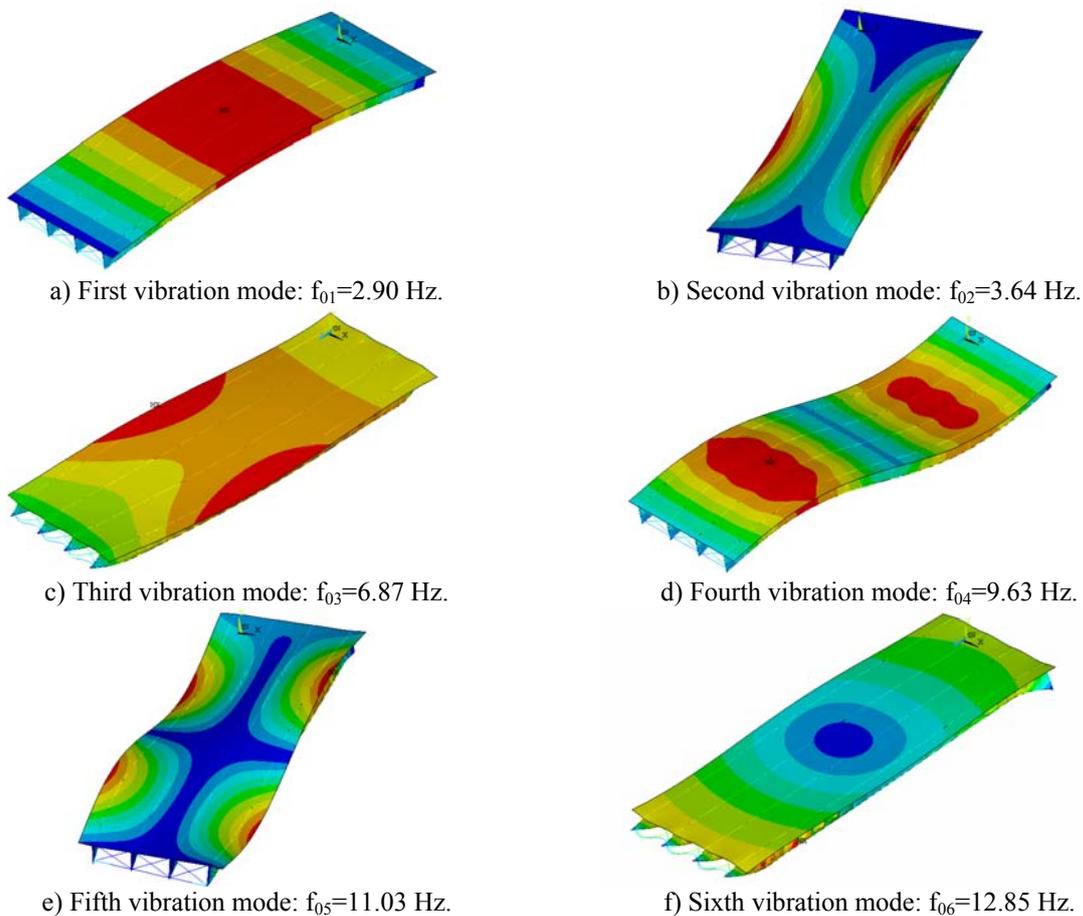


Figure 6: Composite bridge vibration modes.

The analysis proceeds with the evaluation of the steel-concrete composite bridge dynamic response (displacements and stresses). The modal damping coefficient adopted in this work is equal to 0.03 ($\xi = 3\%$) [1, 2, 10]. Three loading models (LM-I, LM-II and LM-III) were investigated based on the central track passage of the vehicles on the bridge deck, see Figure 7 to 9. Figure 10 presents the sections where the stresses were evaluated on the bridge deck structure. The stress values obtained in the dynamic analysis, due to load mobility effect, were used to evaluate the fatigue of the bridge determining the service life of the structure.

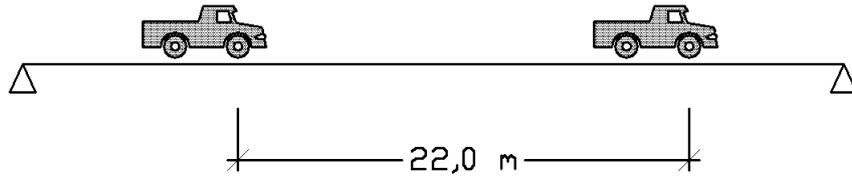


Figure 7: Two vehicles crossing the bridge: $v = 80$ km/h and $l = 22$ m (Load Model I - LM-I).

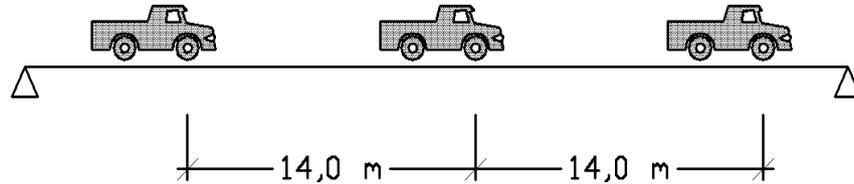


Figure 8: Three vehicles crossing the bridge: $v = 80$ km/h and $l = 14$ m (Load Model II - LM-II).

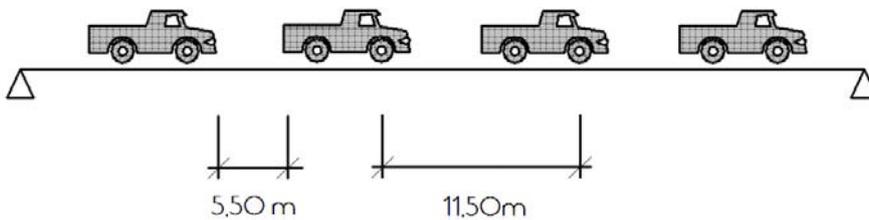


Figure 9: Four vehicles crossing the bridge: $v = 60$ km/h and $l = 11.50$ m (Load Model III - LM-III).

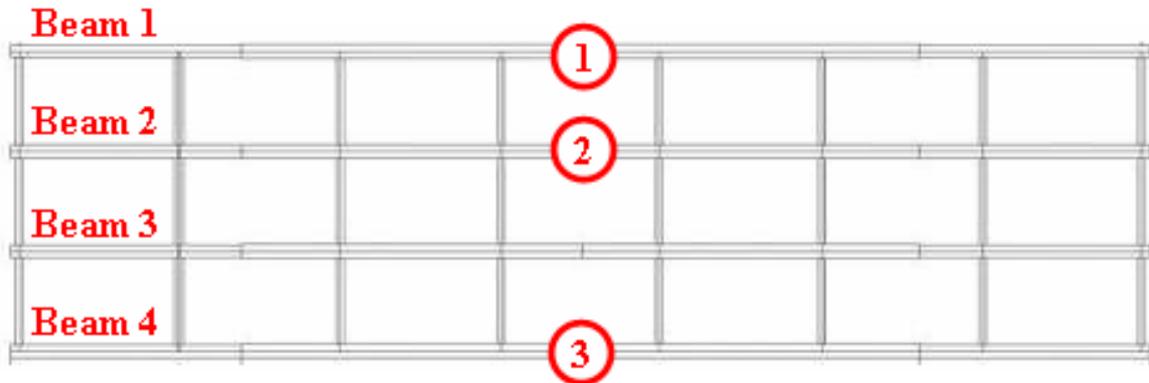


Figure 10: Location where the stresses were obtained on the bridge structure.

6 FATIGUE ANALYSIS

Fatigue is a localized damage process of a component produced by cyclic loading. It is the result of the cumulative process consisting of crack initiation, propagation, and final fracture of a structural component. To determine the fatigue service life and cumulated damage it is necessary to obtain the stress ranges during the vehicles moving on the bridge structure.

The fatigue strength for nominal stresses is represented by a series of logarithmic curves (S-N curves), which correspond to typical detail categories. An S-N curve defines alternating stress values versus the number of cycles required to cause failure at a given stress ratio. The Y-axis represents the stresses (S) and the X-axis represents the number of cycles (N).

The S-N curves of each investigated design code [7, 8] were used to obtain the cumulative damage and respectively the service life of each analysed structural element. It must be emphasized that each design code [7, 8] considers different structural details classification. The types of structural details analysed in this work are presented in Table 3.

The bridge steady state dynamic response was considered to obtain the maximum stress values due to vehicles traffic. The Rainflow cycle counting was applied to determine the stress ranges. Each associated stress cycle was used proportionally with 2×10^6 cycles [7, 8]. Tables 4 to 6 illustrate the bridge service life when LM-I, LM-II and LM-III were considered, see Figures 7 to 9 (σ_{\max} : maximum stresses and $\Delta\sigma_{\max}$: stresses maximum variation).

Detail	Picture	Type	Design code element fatigue class	
			AASHTO [7]	EUROCODE 3 [8]
1		Weld between web and flange	B	125
2		Cross section union weld	B	125
3		Connectors weld	C	80
4		Bottom stiffeners weld	C	80

Table 3: Structural details classification and fatigue class.

Detail	Beam	σ_{\max} (MPa)	$\Delta\sigma_{\max}$ (MPa)	Bridge service life in years	
				AASHTO [7] 75 years*	EUROCODE 3 [8] 120 years*
1 and 2	1	63.03	12.00	19555.02	19854.81
	2	91.03	39.00	1065.71	1082.05
	4	63.03	12.00	19555.02	19854.81
3 and 4	1	63.03	12.00	7165.20	4987.30
	2	91.03	39.00	390.49	271.80
	4	63.03	12.00	7165.20	4987.30

*Minimum bridge service life

Table 4: LM-I: Central track. Load mobility effect.

Detail	Beam	σ_{\max} (MPa)	$\Delta\sigma_{\max}$ (MPa)	Bridge service life in years	
				AASHTO [7] 75 years*	EUROCODE 3 [8] 120 years*
1 and 2	1	96.59	11.00	14154.66	14371.66
	2	127.95	52.00	567.44	576.14
	4	96.59	11.00	14154.66	14371.66
3 and 4	1	96.59	11.00	5186.44	3610.00
	2	127.95	52.00	207.92	144.72
	4	96.59	11.00	5186.44	3610.00

*Minimum bridge service life

Table 5: LM-II: Central track. Load mobility effect.

Detail	Beam	σ_{\max} (MPa)	$\Delta\sigma_{\max}$ (MPa)	Bridge service life in years	
				AASTHO [8] 75 years*	EUROCODE 3 [9] 120 years*
1 and 2	1	27.28	35.00	296.20	300.74
	2	29.67	53.00	71.83	72.93
	4	27.28	35.00	296.20	300.74
3 and 4	1	27.28	35.00	108.53	75.54
	2	29.67	53.00	26.32	18.32
	4	27.28	35.00	108.53	75.54

*Minimum bridge service life

Table 6: LM-III. Central track. Load mobility effect.

The stress values obtained in the dynamic analysis, due to load mobility, were used to evaluate the fatigue performance of the bridge measured in terms of its service life. However, the analysis of the results is very complex, with numerous stress peaks of diversified stress magnitudes. Thus, manual stress cycle counting is, for these structures, an impossible task. This way, the present investigation adopts the Rainflow method for stress cycle counting.

The results presented in Tables 4 to 6, related to the three loading cases LM-I, LM-II and LM-III (see Figures 7 to 9), indicated that the investigated steel-concrete composite highway bridge will perform safely with an acceptable probability that indicates that a failure by fatigue cracking will not occur.

In fact, in most of the analysed cases, the composite bridge service life values were higher than those proposed by the design codes [7, 8], ensuring that the members, connections and joints subjected to dynamic actions related to the load mobility effects will not failure by fatigue cracking.

However, it must be emphasized that when four vehicles related to the LM-III (see Figure 9) are crossing the deck with velocity of 60 km/h on the bridge central track path, the dynamic actions have generated stress values that could compromise the bridge service life. Base on the results presented in Table 23, it was concluded that the service life values proposed by the design codes [7, 8] were surpassed in some design situations.

7 CONCLUSIONS

In this investigation, an analysis methodology was presented to evaluate the fatigue of the steel and composite highway bridges in terms of the structural system service life. The composite bridge service life results corroborated the importance of considering the roughness of the pavement surface and other design parameters like: floor thickness, structural damping and beam cross section geometrical properties in the bridge dynamic and fatigue analysis.

The analysis methodology has considered a vehicle-structure mathematical model, which includes the interaction between their dynamical properties and is developed to evaluate the vehicle-structure response running in the time domain.

The investigated steel-concrete composite highway bridge will perform safely with an acceptable probability that indicates that failure by fatigue cracking will not occur. The structural system service life values were higher than those proposed by the design codes [7, 8], ensuring that the members, connections and joints will not failure by fatigue cracking.

On the other hand, when the dynamic actions related to four vehicles crossing the deck with velocity of 60 km/h on the bridge central track path were applied on the bridge deck, it was observed that the service life values proposed by the design codes [7, 8] were surpassed.

8 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support for this work provided by the Brazilian Science Foundations CAPES, CNPq and FAPERJ.

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