Non-deterministic dynamic structural analysis and fatigue assessment of the Honestino Guimarães bridge steel central span (Brasília/DF)

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Abstract. The Honestino Guimarães bridge was inaugurated in the mid-1970s and is the only structure of its kind in Brasília/DF, Brazil, designed by the renowned architect Oscar Niemeyer, where his understanding of structural systems was used aiming to create a symbolic and harmonious environment, having in mind a structural system composed by steel and concrete. The central span of the bridge is made of steel, with an orthotropic deck, presenting an arched shape, Gerber joint connection, and longitudinal symmetry. This way, this research work aims to investigate the non-deterministic dynamic structural behaviour and assess the structural fatigue on the central span of the bridge. A global finite element model and a numerical submodel were developed using the ANSYS software, incorporating the main physical and mechanical characteristics of the bridge's central span, which measures 58.00 m in length and 13.50 m in width, with the concrete slab thickness varying from 0.07 m to 0.12 m. Thus, the hot-spot stresses were determined from the submodel created from shell elements (SHELL188), and identified through the use of the Vehicle Bridge Interaction (VBI) computational software, which integrates the ANSYS and MATLAB software. It was observed that, as the vehicle velocity increases, the time interval in which hot-spot stresses reach the maximum values decreases. The interval between maximum stresses shortens as the vehicle velocity increases, indicating faster and more concentrated dynamic loads. The intensity of hot-spot stresses is more influenced by the number of vibration modes than by the vehicle velocity, highlighting the importance of appropriate numerical modelling in order to capture the dynamic effects.

Keywords: Dynamic Structural Analysis, Numerical Modelling, Pavement Deterioration, Hot-spot Stress Method. Fatigue Assessment.

1 Introduction

The dynamic analysis of highway bridges under moving loads is a fundamental challenge in structural engineering, essential for ensuring the safety and durability of roadway infrastructure. The forces generated by vehicular traffic, coupled with inevitable pavement deterioration, induce stress cycles that can lead to fatigue and, consequently, structural failures. The stochastic nature of traffic and the complex interaction with the structure make the accurate modelling of these phenomena a challenging, yet critical, task [1] .



Figure 1. Location of the orthotropic steel box girder on the bridge [2].

This study focuses on an icon of Brazilian architecture and engineering: the central steel span of the Honestino Guimarães Bridge in Brasília, DF, Brazil (Fig. 1), which is the only structure of its kind designed by Oscar Niemeyer in the capital city. Given its historical importance, age, and the constant traffic growth in the Federal District, the assessment of its structural integrity is a primary concern. This investigation aims to contribute to the preservation of this architectural heritage.

To achieve this objective, the research employs an advanced non-deterministic analysis methodology, utilizing a Vehicle-Bridge Interaction (VBI) algorithm [3]. This approach integrates the ANSYS [4] and MATLAB [5] computational platforms to simulate the dynamic response of the bridge to the passage of the HL-93 fatigue vehicle. The analysis considers annual heavy traffic volumes of 125.000, 150.000, and 175.000 vehicles, simulated on a detailed and calibrated numerical model [6] at speeds of 40, 60, 80, 100, and 120 km/h. A fundamental feature of this study is the incorporation of progressive pavement deterioration, allowing for a more realistic analysis of long-term impacts.

Thus, the main purpose is to examine the dynamic behaviour and verify the fatigue resistance of the bridge's central steel box-girder span, analysing how traffic growth and pavement deterioration impact the structural displacements and stresses. By employing sophisticated methods such as the hot-spot stress approach and the cumulative damage rule, the study culminates in an estimation of the service life of the central span.

2 Geometric description and modal analysis of the model

The bridge's central span is a composite steel-concrete structure, characterized by an arched, orthotropic steel box-girder with Gerber connections. Designed as a continuous beam, the deck extends over a total span of 58.00 meters, with a concrete slab whose thickness varies between 7 and 12 centimetres (see Fig. 2). The bottom profile features an arch plate designed to reduce deflections caused by self-weight and service loads. Regarding its geometry, the deck's cross-section has a height that varies from 2.20 m at the centre of the section to 2.00 m at the edge [2]. The steel box-girder is internally subdivided into three cells, while the walkway structures and pipelines are positioned at the lateral extremities of the bridge. This symmetrical configuration defines the deck as orthotropic.

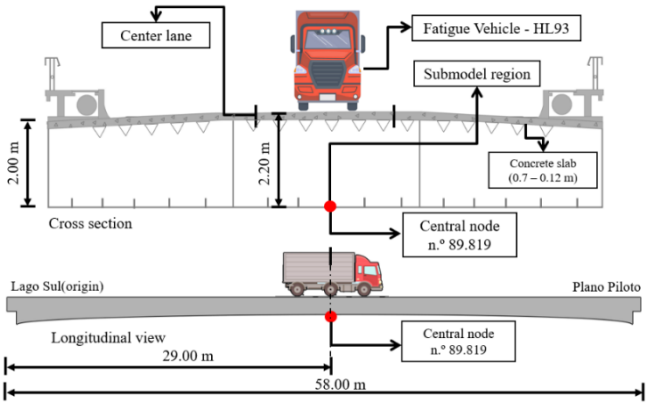
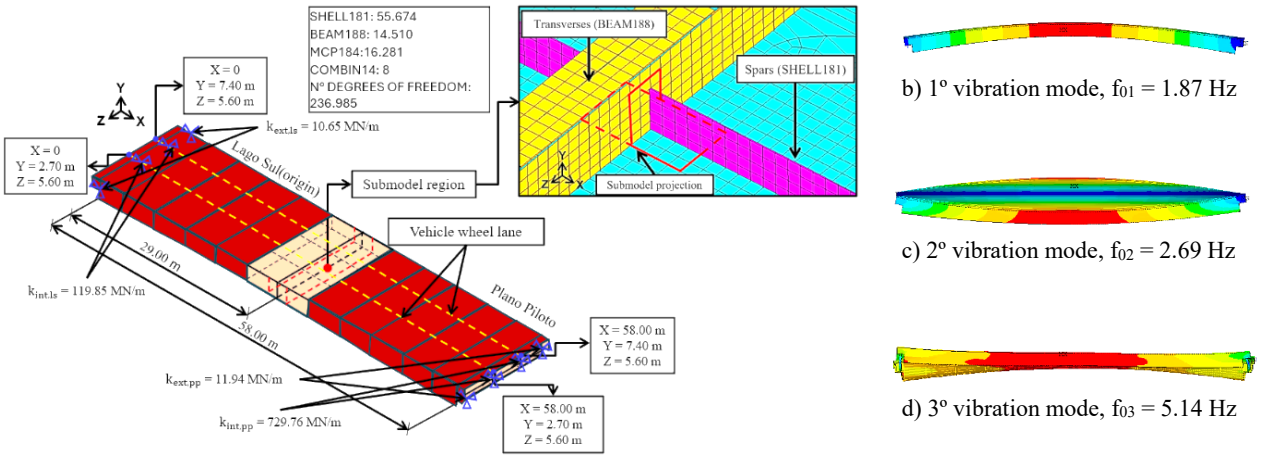


Figure 2. Cross section and longitudinal view.

The submodel region, designated for a more refined finite element analysis, is positioned at the geometric centre of the superstructure's central span, at the bottom flange. This area of interest encompasses the slab section directly underlying the wheel paths of the HL-93 [7] fatigue design vehicle, indicated in the Fig. 3, by the dashed lines. The vehicle's trajectories cross the structure longitudinally and symmetrically relative to its central axis. The placement of the submodel is strategic, as it aligns with the area of maximum load demand, thereby enabling a detailed analysis of the hot-spot stresses and critical strains induced by traffic loading.

The bridge's central steel box girder deck was modelled using the ANSYS APDL 17.2 [4] software. In the numerical model of the bridge, the finite element used to represent the Winkler spring-type supports was COMBIN14. The resulting finite element model, comprises 88.136 nodes, 86.473 elements, and 236.985 degrees of freedom, with a discretized mesh size of approximately 5.00 cm, only in this region. The concrete slab was integrated through rigid connections using 16.281 MCP184 elements, which establish the link to the deck's top flange plate. The bottom flange plate, along with the side and top closure plates, were modelled using 55.674

SHELL181 shell elements. The longitudinal girders (stringers), cross-beams, and transverse stiffeners accounted for a total of 14.510 BEAM188 beam elements. The first three vibration modes of the structural model (steel central span), are predominantly bending and torsion. The corresponding natural frequencies are 1.87 Hz, 2.69 Hz, and 5.14 Hz, respectively, for each mode. This entire model was calibrated and updated with experimental measurements [2]. Where the errors of the calibrated vibration modes, respectively, were 1.1% (1º mode), 0.30% (2º mode) and 0.20% (3º mode).



a) Geometric characteristics, vehicle passage position, submodel region

b) 1º vibration mode, f_{01} = 1.87 Hz

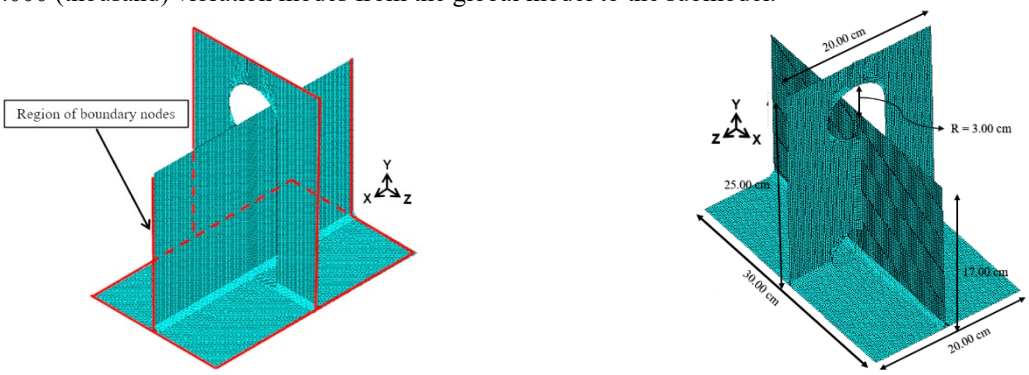
c) 2º vibration mode, f_{02} = 2.69 Hz

d) 3º vibration mode, f_{03} = 5.14 Hz

Figure 3. Geometric characteristics, vehicle passage position, submodel region and vibration modes.

3 Submodelling and stress hot-spots

The numerical submodel was created on the ANSYS [4] platform, adhering to the FAT90 structural detail for fatigue analysis, a conservative classification common this kind of seam welds [8]. The unconstrained model consists of 29.559, SHELL181, shell elements, totalling 178.710 degrees of freedom, with a finely discretized mesh of 2.00 mm. To perform stress verifications, the Vehicle-Bridge Interaction (VBI) codes import the prescribed nodal modal displacements (derived from the modal analysis aiming to subsequently perform a mode superposition analysis) from the global numerical model to the boundaries of the submodel. The symmetrical numerical submodel has the following dimensions: a length of 30.00 cm, a width of 20.00 cm, a cross-beam height of 25.00 cm, a stringer height of 17.00 cm, and a radius of 3.00 cm for the internal hole and quadrant (according to Fig 4). This interpolation modal nodal displacements at the of boundary nodes occurred according to an analysis for a 1.000 (thousand) vibration modes from the global model to the submodel.

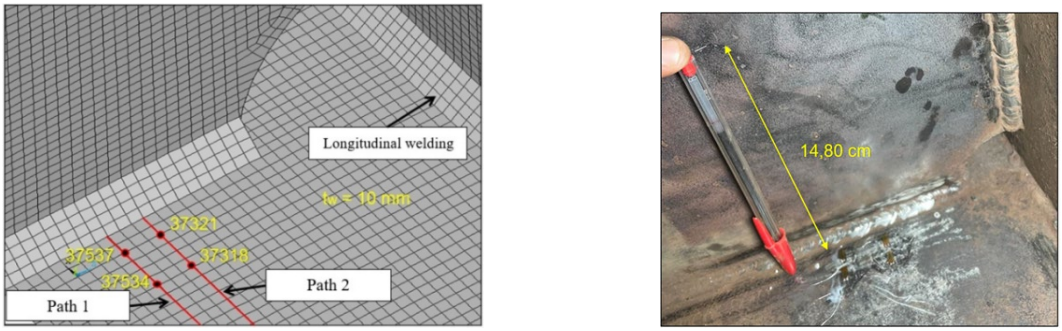


a) Submodel: region of boundary nodes b) Submodel: geometry

Figure 4. Submodel characteristics.

Thus, for linear extrapolation in submodel built with shell elements (SHELL188), the nodal modal hot-spot stresses near the welds (σ_{hs}), can be determined from the stresses at distances of 0,4 × t_w (σ_{hs}) and 1,0 × t_w ($\sigma_{1,0tw}$), where t_w is the profile thickness [9], according to Eq. (1) and visible in Fig. (5).

$$\sigma_{hs} = 1{,}67\ \sigma_{0,4tw} - 0{,}67\ \sigma_{1,0tw} \quad (1)$$



a) Paths and nodes of the submodel b) Detail on site

Figure 5. Assessment region for modal hot-spot stresses.

4 Pavement irregularities

Pavement imperfections on traffic bridges can manifest in various ways, stemming from a variety of factors related to design, construction, and the environmental conditions to which they are exposed. Furthermore, high and constant loading accelerates the pavement's deterioration process, diminishing its strength and intensifying vibrations in the structure [10]. For this study, a random model based on the pavement's spectral density was employed. Eq. (2) demonstrates how to approximate a set of irregularities through a series of harmonics.

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

Being: $\Delta\Omega$ the variation within a discretization interval; $G_d(\Omega_i)$ the spectral density of the irregularity cycles (cm³/cycle); Ω_i the frequency of harmonic i (cycle/m); and θ_i the phase angle at harmonic i, in the range 0 to 2π. Then, the spectral density of irregularity cycles, $G_d(\Omega_i)$, is described by Eq. (3):

$$G_d(\Omega_i) = G_d(\Omega_d)_t \times \left(\frac{\Omega_i}{\Omega_0}\right)^{-2} \tag{3}$$

Such that, $G_d(\Omega_d)_t$, is the road roughness coefficient known as RRC (cm³/cycle); Ω_0 is the frequency of the discontinuity, equal to $1/2\pi$, and Ω_i represents the spatial frequency of harmonic i (cycles/m). Therefore [11], relates the roughness coefficient, RRC, with the International Roughness Index, IRI, according to Eq. (4):

$$RRC_t = \Phi(\Omega_0)_t = 6{,}1972 \times 10^{-9} \exp\left(\frac{IRI_t}{0{,}42808}\right) + 2 \times 10^{-6} \tag{4}$$

Within the scope of modelling these non-deterministic irregularities, the adopted approach starts from the mathematical representation of the profile. This representation is materialized by the irregularities function, designated as Vb(x). The construction of this function is based on the use of its complex Fourier spectrum, which decomposes the profile into its various frequency components, as shown in Eq. (5):

$$V_b(x) = \sum_{i=1}^{n} V_{bi} \cos\left[\omega_i x - \varphi_i\right] \tag{5}$$

Where: V_{bi}, is the real amplitude of the harmonic part; ω_i, corresponds to the frequency of harmonic i; φ_i, the phase angle of a determined harmonic i; and N, the number of harmonics.

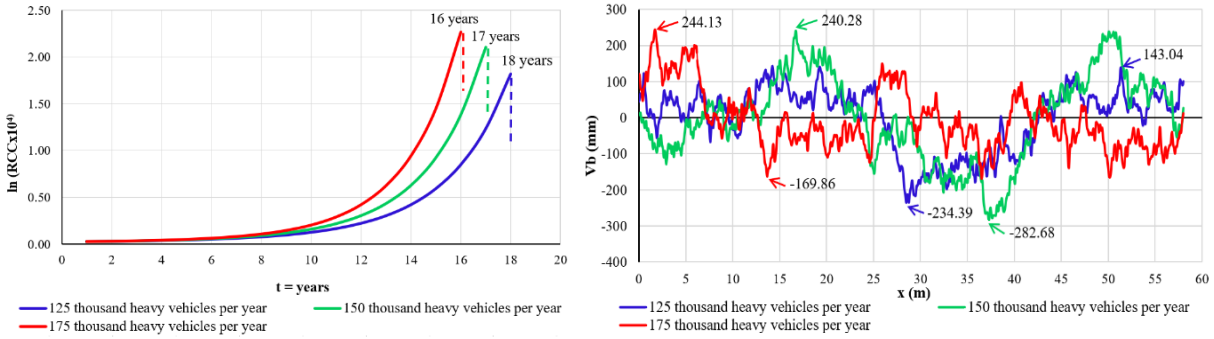
The mathematical formulation that allows the calculation of these two parameters is presented below. Thus, Eq. (6) establishes the expression for determining the harmonic amplitude (V_{bi}). In turn, Eq. (7) is dedicated to the formal definition of the spectral density of irregularities, [$\Phi_{vbvb}(\omega)$]. Where: $\Delta\omega$, represents the discretization interval; W is the ondulability of the track, adopted with a value equal to 2.0 [12]. The pavement's renewal limit has been reached ($RCC_{lim} = 256 \times 10^{-6}$) [13].

$$V_{bi} = \sqrt{2\ \Delta\omega\ \Phi_{vbvb}(\omega_i)} \tag{6}$$

$$\Phi_{vbvb}(\omega_i) = \Phi(\omega_0)\ \left[\frac{\omega}{\omega_0}\right]^{-w} \tag{7}$$

The evolution of a pavement renewal criterion (RCC) over time, as a function of different annual traffic volumes, is illustrated in Fig. 6(a). The curves show an exponential growth of the degradation index, indicating that accumulated damage accelerates over the years. It is observed that a higher traffic volume, such as 175

thousand vehicles per year, advances the need for renewal to 16 years. Conversely, lower volumes, such as 125 thousand vehicles per year, extend the pavement's service life to 18 years, defining the time limit for intervention. So, it was considered a heavy fatigue vehicle equal to a vehicle, HL-93.



a) Pavement limit b) Pavement irregularity sample

Figure 6. Characteristics of the irregularities of the central orthotopic steel box span.

Samples of longitudinal pavement irregularity profiles (V_b) for a 58-meter road length, representing the surface condition under different traffic histories, are presented in Fig. 6(b). The data is non-deterministic in nature, characterizing the random variations of the surface. Visually, the profiles associated with higher traffic volumes (175 thousand and 150 thousand heavy vehicles/year) tend to exhibit greater amplitudes between peaks and valleys, such as the extreme values of +244.13 mm and -282.68 mm, suggesting a more advanced state of degradation.

5 Fatigue standard heavy vehicle HL-93

The three-dimensional numerical model of the HL-93 fatigue design truck [7] was developed using the ANSYS APDL 17.2 [4] software package. The model features 12 degrees of freedom and has three axles, with a longitudinal spacing of 4.30 m between each and a tire spacing of 1.80 m (Fig. 7). A load of 35 kN was applied to the 1st axle (front, with single tires). For both the 2nd axle (intermediate, with dual tires) and the 3rd axle (rear, with dual tires), an applied load of 145 kN was used for each. Where Tab. (1) shows the mechanical properties of the vehicle under structural fatigue HL-93.

Table 1. Mechanical properties of the fatigue heavy vehicle HL-93.

	1° Axle	2° Axle	3º Axle
Unsprung mass (kg)	490	808	653
Front sprung mass (kg)	-	2.612	-
Rear sprung mass (kg)	-	28.077	-
Axle damping (Ns/m)	2.190	7.882	7.182
Tire damping (Ns/m)	2.000	2.000	2.000
Axle suspension (N/m)	242.604	1.903.172	653
Tire stiffness (N/m)	875.082	3.503	3.507.429

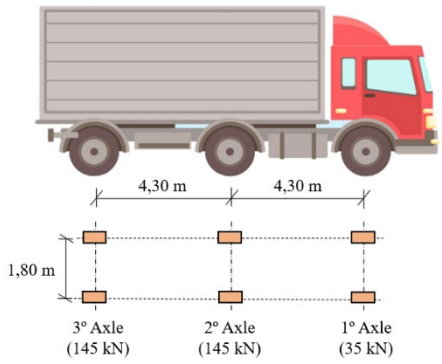


Figure 7. Fatigue heavy vehicle HL-93.

6 Fatigue assessment

The presented methodology describes a hybrid numerical process for bridge fatigue analysis via vehicle-bridge interaction (VBI), integrating the ANSYS [4] and MATLAB [5] software. The first part of the process focuses on modelling and dynamic simulation: in Step 1, finite element models of the bridge and vehicle are developed in ANSYS [4] to extract their matrix properties. Subsequently, in Step 3, MATLAB [5] executes the

interaction simulation, which considers the bridge's modal analysis and road irregularity profiles to obtain the system's complete dynamic response during the vehicle's passage (Fig. 8). This interaction is calculated by the superposition method for the central span structure of the bridge and Newmark to determine the displacements, speed and acceleration of the heavy vehicle, HL-93 on the structure.

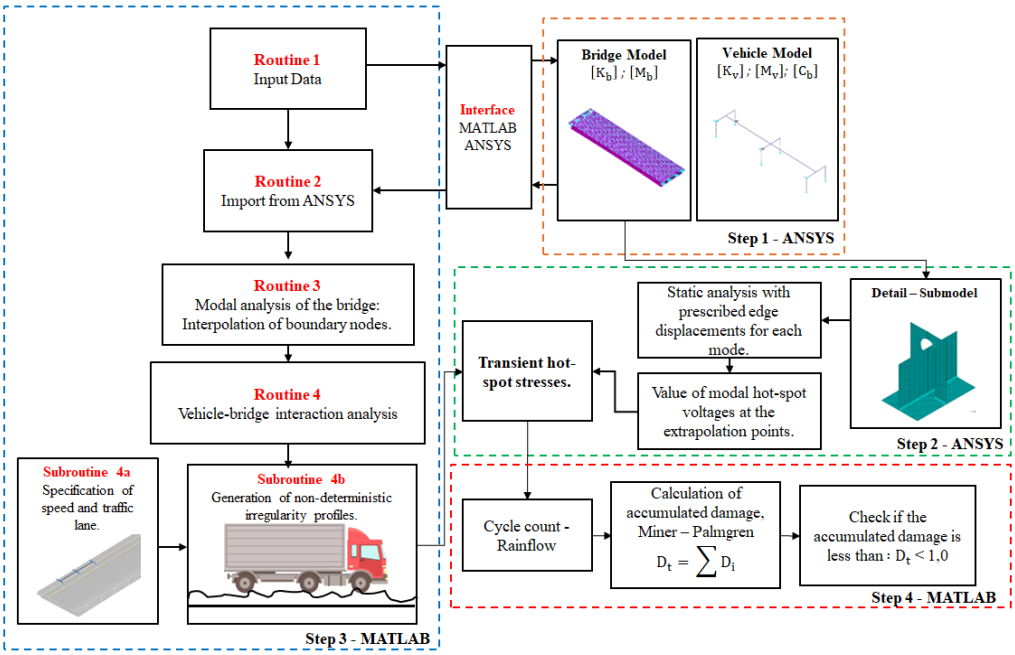
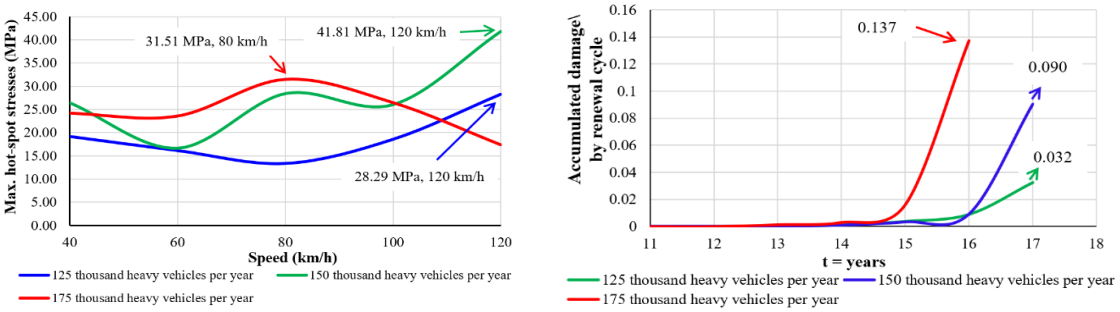


Figure 8. Flowchart of the computer application, Vehicle-Bridge Interaction (VBI).

The second part focuses on stress analysis and fatigue damage assessment. In Step 2, a submodeling technique in ANSYS [4] is used to accurately determine the modal hot-spot stresses at critical structural details. These stresses are combined with the dynamic response from the VBI simulation to generate the transient stress time history. Finally, in Step 4, this time series is processed in MATLAB [5] using the Rainflow counting algorithm and the Palmgren-Miner [14] rule to quantify the accumulated fatigue damage, which is then compared against the design criterion ($D_t < 1,0$).

The relationship between vehicle speed and the maximum hot-spot stress generated in the structure, detailed in Fig. (9), reveals a complex influence of vehicle dynamics. It is observed that the magnitude of the stresses does not follow a linear trend, varying significantly with speed for each traffic scenario. For an annual traffic flow of 175 thousand heavy vehicles/year, for instance, the maximum stress of 31.51 MPa is reached at the critical speed of 80 km/h. Conversely, for other traffic volumes, the peak stresses occur at different speeds, such as 41.81 MPa at 120 km/h for 150 thousand heavy vehicles/year, demonstrating the structural response's sensitivity to dynamic parameters.



a) Maximum hot-spot stresses. b) Accumulated damage\pavement renewal cycle.

Figure 9. Maximum hot-spot stresses and accumulated damage.

Regarding the progression of accumulated fatigue damage over the service life, caused by the annual passage of the standard HL-93 fatigue vehicle [7], the exhibited behaviour is markedly non-linear, with the damage remaining at very low levels for a long period before accelerating exponentially in the final years, which is

characteristic of fatigue degradation processes. It becomes evident that a higher traffic volume, such as 175 thousand trucks per year, accelerates damage accumulation, reaching a damage index of 0.137 in 16 years and anticipating the need for pavement renewal compared to the lower traffic scenarios.

7 Conclusions

This study assessed the structure of the Honestino Guimarães bridge steel central span through an advanced Vehicle-Bridge Interaction (VBI) methodology that integrates the ANSYS [4] and MATLAB [5] software. The analysis investigated the dynamic behaviour and fatigue of the central span considering traffic scenarios of up to 175 thousand heavy vehicles annually and the effect of progressive pavement deterioration. This detailed approach is crucial for preserving the architectural legacy of Oscar Niemeyer and ensuring the structure's safety.

The quantitative results demonstrated a complex, non-linear relationship between speed and stress. For the flow of 175 thousand heavy vehicles/year (HL-93), for example, the maximum stress of 31.51 MPa was reached at the critical speed of 80 km/h. The fatigue damage analysis revealed that this same traffic volume accelerates degradation, reaching a damage index of 0.137 in 16 years, while lower volumes extend the service life to up to 18 years. This accelerated damage accumulation is a critical indicator for planning interventions. Therefore, the ability to predict that an increase in traffic could advance the need for rehabilitation by years allows for proactive planning, optimizing resources, and ensuring the longevity of the bridge's main steel span.

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