

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Dynamic Analysis and Fatigue Assessment of an Existing Railway Steel Bridge

João Diogo de Oliveira Dias Boavida Barroso



A dissertation presented to the Faculty of Engineering of the University of Porto for the degree of Master in Mechanical Engineering

Supervisor: Doctor Pedro Aires Montenegro

Co-Supervisors: Professor José A. Correia, Professor Abílio de Jesus

June, 2019

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Abstract

This dissertation focuses on the study of the dynamic behaviour and fatigue of the Várzeas railway bridge. The development of the bridge model which is currently being the target of several studies is also an objective of this work. This metallic bridge belongs to the Beira Alta Line and it is located across the Várzeas river, near the locality of Luso, in Aveiro district, in Portugal.

The dynamic analyses performed aim to simulate the bridge behaviour assuming the passage of rail vehicles models as a set of moving loads. The modal superposition method is used to characterize the structure response, extracting modal parameters by finite method analysis and combining them with the set of moving load. Some sensitivity analyses related to the number of vibration modes and time increment are also presented.

Lastly, a critical detail is evaluated in terms of fatigue in order to estimate the railway bridge security. The time-history analyses of the nodal stresses are presented and finally, the fatigue damage is calculated applying the Palmgren-Miner rule.

The present formulation is implemented on MATLAB[®], being the structure modeled with a finite element software named ANSYS[®].

Keywords: Railway Metallic Bridges, Dynamic Analysis, Fatigue, S-N Curves, Modal Superposition Method.

Resumo

Com este trabalho pretende-se levar a cabo uma série de análises dinâmicas e abordar a problemática da fadiga da ponte ferroviária das Várzeas. Outro objetivo desta dissertação passa por continuar o desenvolvimento do modelo da ponte, que se encontra atualmente a ser alvo de vários estudos. Esta ponte metálica pertence à Linha da Beira Alta e está localizada sobre a Ribeira das Várzeas, perto da localidade de Luso, no distrito de Aveiro, em Portugal.

As análises dinâmicas realizadas têm o intuito de simular o comportamento da ponte, face à passagem de veículos ferroviários, modelados sob a forma de um conjunto de cargas móveis. Para caracterizar a resposta da estrutura é usado o método da sobreposição modal, sendo as quantidades modais extraídas por uma análise que se baseia no método dos elementos finitos. Algumas análises de sensibilidade relacionadas com o número de modos de vibração e incrementos de tempo são também apresentadas com vista à obtenção de resultados precisos.

Para avaliar a fadiga da ponte, é selecionado um pormenor crítico e levam-se a cabo análises de tensão e, posteriormente, é avaliado o dano por fadiga considerando diferentes cenários de tráfego e aplicando a lei de Palmgren-Miner.

A ponte está modelada no programa de elementos finitos ANSYS® e as respetivas quantidades modais são também extraídas utilizando o mesmo software. Os restantes cálculos são executados no MATLAB®.

Keywords: Pontes Ferroviárias, Análise Dinâmica, Fadiga, Curvas S-N, Método da Sobreposição Modal.

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*"Better to hitch a ride on a dream
than to rot with your thumb in the air."
Brooks Nielsen*

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Chapter 1

Introduction

1.1 Context

The relevance of railways as a mean of transport of passengers and goods has been increasing over the last decades. This is due to some advantages of railways when compared to other types of transport, such as roadway and airway. Those advantages, such as lower costs of transport, lower energy consumption and CO_2 and lower number of incidents and accidents reported emissions, are mostly economic, environmental and safety related (Albuquerque, 2015).

The economic development has been always associated with the construction of railway lines, or with subsequent increase of traffic, vehicle axle loads and speed. In addition, the investment in European transportation maintenance and modernization of the existing infrastructure is increasing. Old railway metallic bridges, in many cases with more than one hundred years, have been required over the years to carry heavier vehicles and endure higher velocities than allowed by the original design. Consequently, the cumulated degradation due to corrosion and fatigue contributes to an increased concern for their safety (Kühn et al., 2008). Hence, there is a need to increase the knowledge and experience in the assessment of existing structures, in particular, structures which are sensitive to fatigue related phenomena have been addressed with more realistic and efficient analysis methodologies.

According to ASCE (1982), about 90% of the failures in steel structures are due to fatigue. In this way, fatigue has emerged as one of the major concerns associated with old metallic railway bridges. Studies developed by Fisher and Roy (2008) showed that the majority of fatigue cracks are caused by the distortion of member cross-sections, local vibration and out-of-plane bending of webs. Nowadays, numerical approaches, using local finite element models, allow one to study the realistic behaviour of structures of this kind with appropriate consideration of vibration-induced distortion and fatigue (Guyer and Laman, 2012). Furthermore, the evaluation of such phenomenon benefits from the implementation of monitoring campaigns, since they reduce the uncertainties associated with the variables of the problem.

In this regard, this work aims the development of a methodology to evaluate of dynamic effects and fatigue of the Várzeas railway bridge.

1.2 Objectives and Scope

This dissertation focuses on the study of the dynamic behaviour and fatigue of the Várzeas railway bridge and this main objective was achieved through a set of intermediate goals:

- Review different methodologies for the dynamic analysis available in the literature;
- Perform dynamic analyses aiming the simulation of the bridge behaviour assuming the passage of rail vehicles models using the moving load model;
- Apply sensitivity analyses to evaluate the influence of the number of vibration modes and time increment in the obtained results;
- Review the methodologies for the fatigue assessment of railway bridges available in the relevant standards and advanced fatigue assessment methodologies available in the literature;
- Evaluate critical details in terms of fatigue damage in order to estimate the railway bridge safety.

1.3 Structure

As a consequence of the objectives described in the previous section, the structure of this work is organized in six chapters including the present one.

In Chapter 2, a literature review regarding the aspects related to the assessment of the train running safety on bridges is presented. Here, an overview of the existing dynamic analysis methods with a special focus on numerical methods. The safety regulations and standards for European railways traffic are approached, consulting mainly the Eurocodes EN1990-A2 (2001) and EN1991-2 (2003). The fatigue in metallic bridges is also introduced and the main focus of it is the presentation of the fatigue assessment of the railway bridges according to the standards. The chapter ends with a brief description of advanced methodologies that are used for the fatigue assessment of structures.

Chapter 3 details the methodology used for the dynamic analyses, in this case, the modal superposition method. Since the moving load method is the numerical method used in this work, it is also approached in this chapter. Then it is described the methodology proposed in this work for the assessment of the train running safety on bridges. An overview of the methodology is presented, along with a brief description of each part that composes it.

Chapter 4 begins with a presentation of the bridge studied and a description of its model. After that, it is presented the definition of the dynamic analysis characteristics, followed by the displacements and accelerations analyses. The chapter ends with a comparison between the results obtained by those analyses and a discussion about the number of vibration modes and time increment that should be used to provide precise results in an efficient way.

In Chapter 5 the fatigue analysis is presented. The chapter begins with the definition of the fatigue analysis characteristics, followed by the stresses obtainment. It is also evaluated the fatigue

damage at the critical details selected caused by a single train passage and traffic mix passage for a year. The chapter ends with a brief conclusion of the results obtained by the analyses performed in this chapter.

Finally, Chapter 6 summaries the main conclusions of each chapter and also suggests some future developments.

Chapter 2

Literature Review

2.1 Introduction

In this chapter, the different dynamic analysis methods that can be used to evaluate the response of the bridge are presented. The different methods are characterized and its advantages and disadvantages are emphasized as well.

The second subject approached is the problem of mechanical fatigue in steel structures, as the more common type of fatigue in mechanical structures. Important to underline that there are other types of fatigue, such as corrosion fatigue, however, those ones are merely introduced.

Bearing in mind the need to build railway bridges with satisfactory performance through its life, there are many standards that those structures must satisfy. Therefore, the criteria presented in the European standards, that the structure needs to fulfill to provide a successful operation are discussed in the present chapter.

2.2 Dynamic Analysis Methods

There are several methods that can be adopted to analyse the dynamic behaviour of railway structures. Each one of them has its advantages and disadvantages. They are:

- Analytical Methods;
- Simplified Methods;
- Empirical Methods;
- Numerical Methods.

In the next sections, a brief description of each method is presented to better understand the range of application of each one.

2.2.1 Analytical Methods

These methods are mostly used for simple structures analysis, since its application in complex structures is usually impracticable. However they can give a first approximation of the dynamic behaviour of single structures that can be, in some cases, extrapolated for other structures. Analytical methods suffered a substantial development thanks to Frýba studies, who proved that it is possible to obtain analytical solutions for classic moving loads problems, considering simply supported beams (Frýba, 1996).

2.2.2 Simplified Methods

Based on analytical solutions, these methods are applied to isostatic bridges, which behaviour is similar to a simply supported beam, and its approximated dynamic response is obtained by the contribution of the first mode of vertical vibration (bending).

Ribeiro (2004) studied two simplified methods: Resonance Excitation Decomposition (RED) and Residual Influence Line (RIL). These methodologies calculate a response based on harmonic series and provide a less exhaustive dynamic analysis comparing with numerical and analytical methods.

2.2.2.1 Resonance Excitation Decomposition

The train passage, defined as a group of moving loads applied on a simply supported beam, is characterized by the following equilibrium equation:

$$EI \frac{\partial^4 y(x,t)}{\partial x^4} + c \frac{\partial y(x,t)}{\partial t} + m \frac{\partial^2 y(x,t)}{\partial t^2} = p(x,t) , \quad (2.1)$$

where EI , c , m represent the bending stiffness, damping and mass per unit length, respectively.

Simplifying the global system, it is assumed that the system can be defined with a single degree of freedom:

$$M^* \ddot{Y}(t) + C^* \dot{Y}(t) + K^* Y(t) = F^*(t) , \quad (2.2)$$

where M^* , C^* , K^* , F^* represent the mass, damping, stiffness and generalized forces of the system, respectively. These parameters are calculated by applying the Virtual Work Principle.

After solving the equations, the next step consists of representing the dynamic excitation by Fourier series. In this way, this method considers that the most relevant contribution for the acceleration calculation is due to the Fourier series term related to the resonance.

In this way, the equation that represents the time-dependent acceleration is obtained, and the last simplification of this method allows the obtainment of a time-independent acceleration, as shown in the next expression:

$$\ddot{y} \leq C_t A \left(\frac{L}{\lambda} \right) G(\lambda) . \quad (2.3)$$

The term C_t , which depends on the length of the isostatic beam (L) and mass per length unit (m), is calculated by:

$$C_t = \frac{4}{mL\pi}. \quad (2.4)$$

The factor $A\left(\frac{L}{\lambda}\right)$, which depends on the length of the isostatic beam and length wave of excitation (λ), is determined by:

$$A\left(\frac{L}{\lambda}\right) = \left| \frac{\cos\left(\frac{\pi L}{\lambda}\right)}{\left(\frac{2L}{\lambda}\right)^2 - 1} \right|. \quad (2.5)$$

The term $G(\lambda)$, which depends on the length wave of excitation, damping coefficient (ξ), load (P_k) and its position (x_l), and number of loads (N), is a factor called train spectrum that represents the excitation caused by the train and resonance response of the bridge and is given by:

$$G(\lambda) = \frac{1}{\xi x_{N-1}} \left[\sqrt{\left(\sum_{k=0}^{N-1} P_k \cos\left(\frac{2\pi x_k}{\lambda}\right) \right)^2 + \left(\sum_{k=0}^{N-1} P_k \sin\left(\frac{2\pi x_k}{\lambda}\right) \right)^2} \right] \left(1 - e^{-2\pi\xi \frac{x_{N-1}}{\lambda}} \right). \quad (2.6)$$

According to the Equation 2.6, the train spectrum only depends on the train characteristics and structural damping coefficient.

The Equation 2.6 assumes that the maximum dynamic response occurs when the whole train is already left the bridge. However, real measurements, made on railway bridges in service, confirm the fact that the most significant dynamic effects do not occur from the moment when the last load has left the structure. In some cases, the maximum accelerations are reached during the passage of the train through the bridge. It was concluded, for this reason, that the application of this methodology does not favour the security side. To avoid this, the concept of subtrain is introduced (Barbero, 2001). The subtrain concept is defined as each of the sets of consecutive loads, that can be formed from the first axis of the composition. Thus, the first subtrain could be constituted with the load F_1 . In the same way, the second subtrain would be determined by the first two axels, and so on (see Figure 2.1).

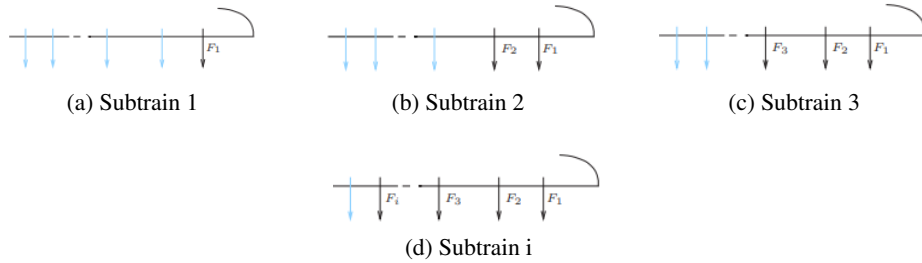


Figure 2.1: Subtrains configuration (Barbero, 2001).

Therefore, it is correct to approximate the dynamic effect produced by a train of loads, as the maximum of the effects produced by all the subtrains related to the train of loads (see Figure 2.2). In this way, the effects produced by all the possible *subtrains* are taking into account and the

accurate spectrum of the train of loads is defined as:

$$G(\lambda) = \max_{i=1, N-1} \frac{1}{\xi x_i} \left[\sqrt{\left(\sum_{k=0}^i P_k \cos\left(\frac{2\pi x_k}{\lambda}\right) \right)^2 + \left(\sum_{k=0}^i P_k \sin\left(\frac{2\pi x_k}{\lambda}\right) \right)^2} \right] \left(1 - e^{-2\pi\xi \frac{x_i}{\lambda}}\right). \quad (2.7)$$

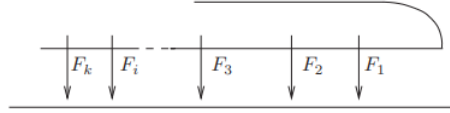


Figure 2.2: Train of loads (Barbero, 2001).

2.2.2.2 Residual Influence Line

The mathematical development of this method is based on the analysis of the vibrations produced by the passage of a single moving load over an isostatic bridge (Barbero, 2001). This method does not take into account the interaction between the vehicle and structure since it is based on a moving load model. RIL considers two main assumptions (Montenegro, 2008):

- The trains are long when compared to the bridge span;
- The maximum dynamic response tends to occur at the instant that the last load of the train leaves the bridge.

The structure response is calculated by the superposition principle effect of each train load, being the individual effect of each load determined by the knowledge of its position in the residual influence line. Thus, the contribution of N loads, represented by P_i , for the displacement at the mid-span (y) is given by the following expression:

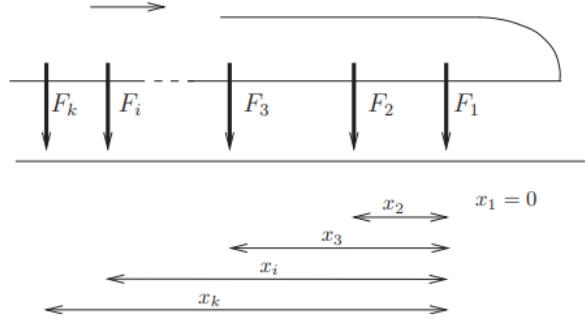
$$y = \sum_{i=0}^{N-1} \left[\frac{-r}{1-r^2} \frac{P_i}{Mw_0^2} \sqrt{e^{-2\xi \frac{\pi}{r}} + 1 + 2\cos\left(\frac{\pi}{r}\right) e^{-\xi \frac{\pi}{r}} \sin(w_0 t_i) e^{-\xi w_0 t_i}} \right], \quad (2.8)$$

where w_0 is the angular natural of the system, M is the system mass; t_i is the temporal delay between the loads and it is obtained by $t_i = \frac{x_i}{v}$, where x_i represents the distance between the axle i and the first axle of the composition (see Figure 2.3) and v represents the loads velocity; the factor r is calculated by:

$$r = \frac{v}{2Ln_0}, \quad (2.9)$$

where L represents the span of the isostatic beam and n_0 is the fundamental frequency.

Considering the instant that the last load leaves the structure and by using the sum of sinusoids method, it is possible to determine the expressions that allow the calculation of the maximum

Figure 2.3: Distances x_k for the determination of $G(\lambda)$ (Barbero, 2001).

displacement (y_{max}) and maximum acceleration ($\ddot{y}_{max}$):

$$y_{max} = C_{disp} A(r) G(\lambda), \quad (2.10)$$

$$\ddot{y}_{max} = C_{accel} A(r) G(\lambda), \quad (2.11)$$

where the constants C_{disp} and C_{accel} are calculated by:

$$C_{disp} = \frac{1}{M w_0^2}, \quad (2.12)$$

$$C_{accel} = \frac{1}{M}, \quad (2.13)$$

where the term $A(r)$ represents the dynamic response factor of the bridge and only depends on its own parameters:

$$A(r) = \frac{-r}{1-r^2} \sqrt{e^{-2\xi \frac{\pi}{r}} + 1 + 2 \cos\left(\frac{\pi}{r}\right) e^{-\xi \frac{\pi}{r}}}, \quad (2.14)$$

and $G(\lambda)$ is a factor that depends on the parameters related to the train. Since the maximum of the dynamic effect may not occur at the instant that the train leaves the bridge, $G(\lambda)$ is considered as the maximum of each *subtrain* corresponding factor (Barbero, 2001):

$$G(\lambda) = \max_{i=1, N-1} \sqrt{\left(\sum_{k=0}^i P_k \cos(2\pi \delta_i) e^{-2\pi \xi \delta_i} \right)^2 + \left(\sum_{k=0}^i P_k \sin(2\pi \delta_i) e^{-2\pi \xi \delta_i} \right)^2}, \quad (2.15)$$

where $\delta_i = \frac{x_1 - x_i}{\lambda}$.

The $G(\lambda)$ factor only depends on the distribution of the loads per axle and damping. For these reasons, it is a characteristic of each train and independent of the mechanical properties of the bridge.

2.2.3 Empirical Methods

The empirical methods are based on the analysis of real bridges models and, in most cases, due to the high cost associated to the time required to the analysis and models construction, the process is unpracticable. The extrapolation of the results for other bridges may also be complicated due to the different characteristics that are intrinsic to each structure.

These type of methodologies analyse statistic data obtained by measurement studies of different railway bridges. From that, it is possible to acquire empirical expressions that estimate amplification factors related to the velocity (φ_V), bending moment at mid-span (φ_M) and vertical acceleration (a) (Frýba, 2001):

$$\varphi_V = 1 + 0.365 \frac{\alpha^2 d}{\vartheta l} \pm 0.579, \quad (2.16)$$

$$\varphi_M = 1 + 0.378 \frac{\alpha^2 d}{\vartheta l} \pm 0.661, \quad (2.17)$$

$$\frac{a}{g} = 1,403 \frac{\alpha^2 d}{\vartheta l} \frac{F}{G} \pm 1,449, \quad (2.18)$$

where l is the bridge span, d is the regular gap between the groups of axles, ϑ is the logarithmic decrement of damping, F is the load per axle, G is the value of the permanent loads, g is the gravitational constant and α represents the speed parameter and it is calculated by the expression:

$$\alpha = \frac{v}{2f_0 l}, \quad (2.19)$$

being f_0 the fundamental frequency of the bridge and v the speed of the train.

These expressions have a confidence degree of 95%. However, their application is only valid for the same type of bridges and trains used in their obtainment.

2.2.4 Numerical Methods

Unlike analytic methods, the numerical methods allow the study of structural systems with any degree of complexity using, for example, the finite element method. However, it is important to enhance that the huge amount of time spent in each dynamic analysis is a relevant limitation of these methods. In spite of the existence of several numerical methods, the most relevant methods to analyse the dynamic behaviour of railway systems are the moving load and the vehicle-structure interaction models.

2.2.4.1 Moving Loads Model

The moving loads model is characterized by a set of loads representative of the static loads of the train axles. That set of loads are separated from each other, according to the train geometry.

The necessary steps related to the moving loads model are presented below:

1. Dynamic Equilibrium Equations

The method begins with the establishment of the dynamic equilibrium equations formulation that expresses the equilibrium of the system. In each point and time instant, there is an equilibrium of loads that is represented by the following equation:

$$f_i(t) + f_a(t) + f_e(t) = f_{ext}(t), \quad (2.20)$$

where $f_i(t)$, $f_a(t)$, $f_e(t)$, are the inertia loads, damping loads, elastic loads, respectively, and $f_{ext}(t)$ the externally applied loads.

The dynamic response of the structure can be obtained by solving the dynamic equation of motion (Equation 2.21), where m , c and k are the mass, damping and stiffness matrices of the structure, respectively; x is the nodal displacement and $f_l(t)$ are the time-dependent moving loads, that represent the action of the train over the structure.

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = f_{ext}(t) + f_l(t) \quad (2.21)$$

2. Dynamic Equilibrium Equations Resolution

There are various procedures to solve the dynamic equation of motion. For example, to deal with linear and non-linear systems, the direct integration methods, such as Newmark method (deeply approached by Clough and Pezzen (1975)), are more indicated. On the other hand, considering only linear models, the modal superposition method is usually the chosen one to solve motion equations due to its computational efficiency.

Taking as an example a damped system of two degrees of freedom and according to the Equation 2.21, a system of differential equations of motion can be written as shown in expression 2.22:

$$\begin{bmatrix} m_{11} & m_{12} \\ m_{21} & m_{22} \end{bmatrix} \begin{Bmatrix} \ddot{x}_1(t) \\ \ddot{x}_2(t) \end{Bmatrix} + \begin{bmatrix} c_{11} & c_{12} \\ c_{21} & c_{22} \end{bmatrix} \begin{Bmatrix} \dot{x}_1(t) \\ \dot{x}_2(t) \end{Bmatrix} + \begin{bmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{bmatrix} \begin{Bmatrix} x_1(t) \\ x_2(t) \end{Bmatrix} = \begin{Bmatrix} f_1(t) \\ f_2(t) \end{Bmatrix}, \quad (2.22)$$

or in a compact form:

$$[m]\{\ddot{x}(t)\} + [c]\{\dot{x}(t)\} + [k]\{x(t)\} = \{f(t)\}. \quad (2.23)$$

To a fully characterization of the system motion, it is necessary to establish the initial conditions of displacement $\{x(0)\}$ and velocity $\{\dot{x}(0)\}$ (Rodrigues, 2017):

$$\{x_0\} = \begin{Bmatrix} x_1^0 \\ x_2^0 \end{Bmatrix}, \quad (2.24)$$

$$\{\dot{x}_0\} = \begin{Bmatrix} \dot{x}_1^0 \\ \dot{x}_2^0 \end{Bmatrix}. \quad (2.25)$$

Using the modal superposition method, in order to decouple the differential equations, the initial coordinates are transformed in modal coordinates:

$$\{x(t)\} = [\Phi]\{\eta(t)\}, \quad (2.26)$$

$$\{\ddot{x}(t)\} = [\Phi]\{\ddot{\eta}(t)\}, \quad (2.27)$$

where the matrix $[\Phi]$ represents the transformation modal matrix (time-independent) of the generalized coordinates $\{x(0)\}$ in the modal coordinates $\{\eta(t)\}$. Substituting the transformations done previously (Equations 2.26 and 2.27) in the Equation 2.23:

$$[m][\Phi]\{\ddot{\eta}(t)\} + [c][\Phi]\{\dot{\eta}(t)\} + [k][\Phi]\{\eta(t)\} = \{f(t)\}. \quad (2.28)$$

Considering the orthogonality properties of the modal vectors that structure the modal matrix $[\Phi]$, a linear and independent system of equations can be reached (Equation 2.29):

$$[I][m]\{\ddot{\eta}(t)\} + [\Phi]^T[c][\Phi]\{\dot{\eta}(t)\} + [\Omega^2][k]\{\eta(t)\} = \{N(t)\}, \quad (2.29)$$

where $[I]$ and $[\Omega^2]$ represent, respectively, the identity matrix and a diagonalized matrix, which elements are the squares of the natural frequencies. In its turn, the matrix $[\Phi]^T[c][\Phi]$ is not, necessarily, a diagonalized matrix, and represents the projection of the damping matrix in the modal basis.

After these steps, it is possible to study each mode independently, which simplifies the resolution of the dynamic equation.

3. Modal Displacements Calculation

In the modal basis, the system of Equations 2.29 can be solved by analytical or numerical methods, according to the solicitation type. Important to refer that the response can be determined by using the Duhamel integral (Clough and Pezien, 1975) or a direct integration procedure of each equation in the modal basis.

Once determined the response $\eta_i(t)$ ($i=1,2$ for the example of a system with two degrees of freedom) in the modal or natural basis, the calculation of the response $\{x(t)\}$, in the generalized coordinates, is given by Equation 2.30. Thus, the response, for the example of a damped system with two degrees of freedom and a proportional damping matrix, is determined by solving the last step of Equation 2.30.

The response is given by a superposition of the vibration mode-shape vectors $\{\phi\}_i$ of the non-damped system, multiplied by the relative damped responses in modal coordinates $\{\eta\}_i$. This procedure ensures the modal superposition technique credible.

$$\begin{aligned}
\{x(t)\} &= [\Phi]\{\eta(t)\} \\
&= [\{\phi\}_1 | \{\phi\}_2] \begin{Bmatrix} \eta_1(t) \\ \eta_2(t) \end{Bmatrix} \\
&= \{\phi\}_1 \eta_1(t) + \{\phi\}_2 \eta_2(t) \\
&= \sum_{n=1}^2 \{\phi\}_i \eta_i(t).
\end{aligned} \tag{2.30}$$

2.2.4.2 Vehicle-Structure Interaction Methods

The methodology, considering the interaction between the vehicle and the structure, is a more realistic analysis when compared to the methods previously discussed. The accuracy of the results is due to the fact that not only the bridge is modeled, but also the vehicle, allowing to study the interaction between the two subsystems. These methods are particularly important for studying passengers comfort or train running safety, since in these cases it is necessary to address the response of the vehicle. Hence, the train is no longer represented by moving loads of constant value but rather by point masses, bodies and springs which represent wheels, bogies and coaches (Goicolea et al., 2008), as illustrated in Figure 2.4.

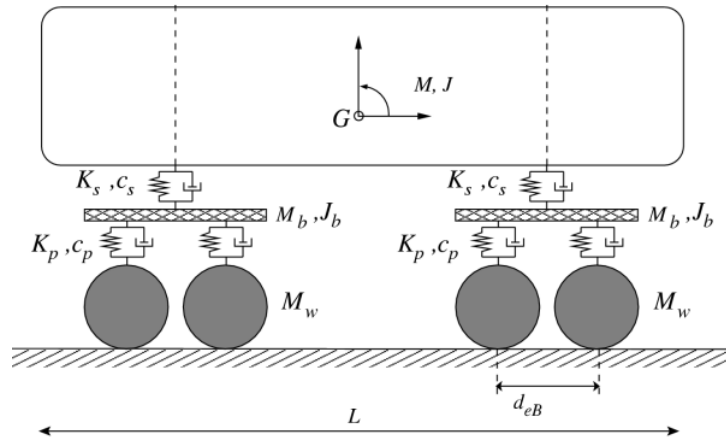


Figure 2.4: Vehicle-structure interaction model (Goicolea et al., 2008).

Therefore, the train can be considered as a vehicle formed by the following elements:

- Vehicle body whose mass, rotational inertia and total length are represented by M , J and L , respectively;
- Bogies whose mass and rotational inertia are represented by M_b and J_b , respectively;

- Primary suspension simulated by springs whose stiffness and damping are represented by K_p and C_p , respectively;
- Secondary suspension simulated by springs whose stiffness and damping are represented by K_s and C_s , respectively;
- Wheels distanced by d_{eB} whose unsprung mass is represented by M_w ;

There are different vehicle-structure interaction methodologies, such as: iterative method (vertical interaction), the direct method (vertical interaction) and the methods considering the wheel and rail geometries (lateral interaction).

The iterative method, studied by Delgado and Santos (1997) and Yang and Yau (1997), considers two different and independent subsystems (vehicle and structure). This method establishes the equilibrium forces acting on the contact interface and uses an iterative procedure to impose the constraint equations. These equations relate the displacements of the contact nodes of the vehicle with the corresponding displacements of the structure (Montenegro, 2015), and allows to study the vehicle-structure interaction in the vertical direction. The equilibrium equations are written splitting the equations related to the train (t) and the equations related to the structure (s):

$$\begin{bmatrix} m_s & 0 \\ 0 & m_t \end{bmatrix} \begin{Bmatrix} \ddot{u}_s(t) \\ \ddot{u}_t(t) \end{Bmatrix} + \begin{bmatrix} c_s & 0 \\ 0 & c_t \end{bmatrix} \begin{Bmatrix} \dot{u}_s(t) \\ \dot{u}_t(t) \end{Bmatrix} + \begin{bmatrix} k_s & 0 \\ 0 & k_t \end{bmatrix} \begin{Bmatrix} u_s(t) \\ u_t(t) \end{Bmatrix} = \begin{Bmatrix} f_s(t) \\ f_t(t) \end{Bmatrix}, \quad (2.31)$$

where m , c and k represent the mass, damping and stiffness matrices, respectively; $\ddot{u}$, $\dot{u}$, u represent the nodal accelerations, nodal velocities and nodal displacements, respectively; f represents the load vector.

This iterative process aims the compatibilization of the two subsystems (Calçada, 1995). Each time step involves the following operations at each iteration:

1. As said before, the moving loads correspond to the action that the train wheelsets apply over the structure. Each moving load $F_s(t)$ is calculated by the subsequent equation:

$$F_s(t) = F_{sta} + F_{dyn}^{i-1}(t), \quad (2.32)$$

where F_{sta} is the static load of the wheelset and $F_{dyn}^{i-1}(t)$ is the dynamic component of the interaction force in the previous interaction i . This latter component is assumed null at the initial instant and in the first interaction, at each time step, this component is equal to that is calculated in the previous time step. By solving the system of Equations 2.31, related to the structure, it is possible to compute the nodal displacements $u_s^i(t)$.

2. At the same time, the displacements $u_t^i(t)$, which correspond to the displacements u_s^{i-1} , are previously calculated. Next, by solving the system of Equations 2.31, related to the vehicle, for each train axle, it is possible to calculate the support reaction forces $F_t^i(t)$, which correspond to the group of interaction forces $F_{dyn}^i(t)$ that will be applied to the structure, in the next iteration. The interaction subsystems schemes are presented in Figure 2.5.

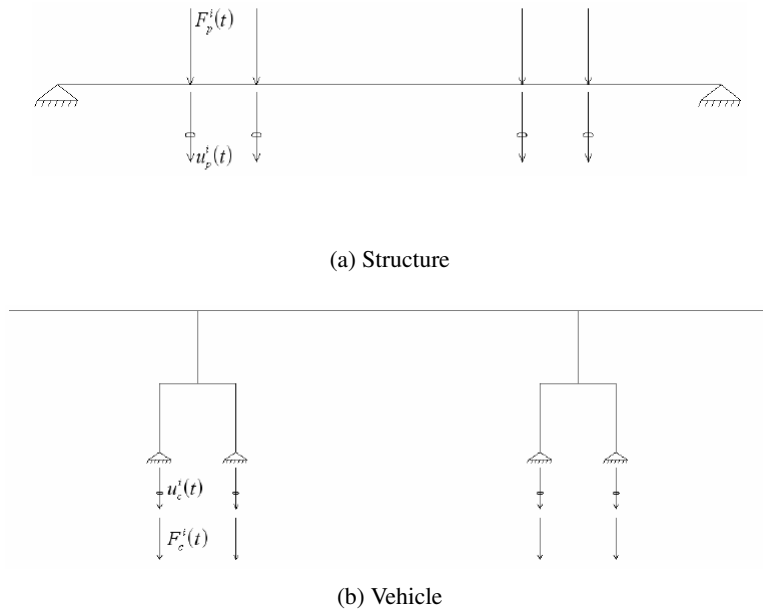


Figure 2.5: Interaction schemes (Calçada, 1995).

3. At the end of each iteration, a convergence criteria is used that takes the dynamic components of the interaction forces, of the current and the next iterations, into account. For this case, the convergence criteria can be defined by the following ratio:

$$\frac{F_{dyn}^i(t) - F_{dyn}^{i-1}(t)}{F_{dyn}^{i-1}(t)}. \quad (2.33)$$

If the ratio referred is inferior or equal to an established tolerance ε , it is considered that the structural subsystems are compatibilized. Consequently, the procedure may advance to the next time step otherwise, the iterative process continues. A synthesis of the numerical methodology approached is presented in Table 2.1.

Table 2.1: Numerical methodology synthesis - interaction vehicle-structure (adapted from Calçada (1995)).

	Structure	Vehicle
Load	$F_s^i(t) = F_{sta} + F_{dyn}^{i-1}(t)$	$u_t^i(t) = u_s^{i-1}(t)$
Result	$u^i(t) = u_s^i(t)$	$F_{dyn}^i(t) = F_t^i(t)$
Convergence Criteria	$\frac{F_{dyn}^i(t) - F_{dyn}^{i-1}(t)}{F_{dyn}^{i-1}(t)}$	If $< \varepsilon \rightarrow$ new time step If $< \varepsilon \rightarrow$ new iteration

Neves et al. (2012) proposed a new method for the dynamic analysis of the vertical vehicle–structure interaction named direct method, that consists in the discretization of the subsystems, with various types of finite elements, with any degree of complexity. The equilibrium equations, of both subsystems, are complemented with additional constraint equations to guarantee the contact between them. It is relevant to refer that the equations of motion and the constraint equations may be solved directly, as a single system. An optimized block factorization algorithm is used to solve the single system of linear equations, expressed as:

$$\begin{bmatrix} \bar{K}_{FF} & \bar{D}_{FX} \\ \bar{K}_{XF} & 0 \end{bmatrix} \begin{Bmatrix} u_F^{t+\Delta t} \\ X^{t+\Delta t} \end{Bmatrix} = \begin{Bmatrix} \bar{F}_F \\ \bar{r} \end{Bmatrix}, \quad (2.34)$$

where $\bar{K}_{FF}$ is the effective stiffness matrix of the system; $\bar{D}_{FX}$ is the transformation matrix that relates the contact forces, in the local coordinate system, with the nodal forces in the global coordinate system; $\bar{K}_{XF}$ is the transformation matrix that makes the correlation between the nodal displacements of the structure in the global coordinate system, with the displacements of the auxiliary points defined in the local coordinate system; $u_F^{t+\Delta t}$ and $X^{t+\Delta t}$ are the nodal displacements and contact forces, respectively, at each time step; $\bar{F}_F$ is the load vector and $\bar{r}$ represents the irregularities in the contact surface.

Later, Neves et al. (2014) reformulated the method in order to allow the separation between the subsystems. To achieve that, the elements in contact are detected by a contact search algorithm. Thus, when occurs contact - only frictionless contact - the constraints are imposed. The constraint equations are purely geometrical and relate the displacements of the contact node with the displacements of the corresponding target element. Studying the contact between two bodies is necessary to establish that conventionally, one of them has a contact surface, and the other a target surface (see Figure 2.6). Being the nature of contact nonlinear, an incremental formulation based on the Newton method is adopted to solve the equation of motion of the vehicle-structure system:

$$\begin{bmatrix} \bar{K}_{FF} & \bar{D}_{FX} \\ \bar{K}_{XF} & 0 \end{bmatrix} \begin{Bmatrix} \Delta u_F^{i+1} \\ \Delta X^{i+1} \end{Bmatrix} = \begin{Bmatrix} \psi(u_F^{t+\Delta t,i}, X^{t+\Delta t,i}) \\ \bar{g} \end{Bmatrix}, \quad (2.35)$$

where Δu_F^{i+1} and ΔX^{i+1} are the incremental nodal displacements and the contact forces, respectively, at each iteration; ψ is the residual force vector, which depends on the nodal displacements and contact forces, calculated in the previous Newton iteration. The iterative scheme continues until the condition:

$$\frac{\|\psi(u_F^{t+\Delta t,i+1}, X^{t+\Delta t,i+1})\|}{\|P_F^{t+\Delta t}\|} \leq \varepsilon, \quad (2.36)$$

is fulfilled. $P_F^{t+\Delta t}$ represents the vector of the externally applied loads at the current time step and ε is a specified tolerance. The accuracy and computational efficiency of the proposed method are demonstrated in a numerical example presented in Neves et al. (2014).

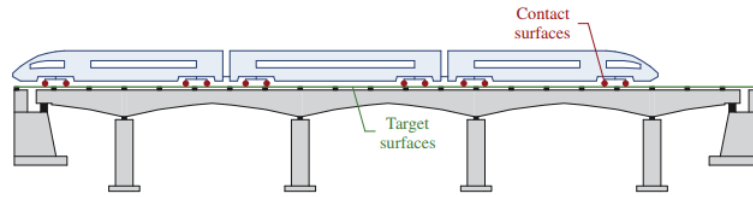
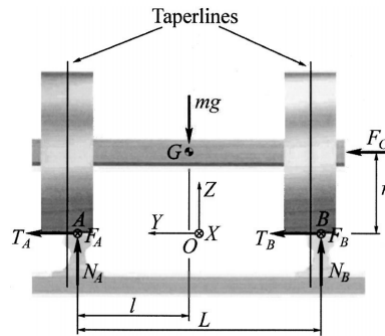


Figure 2.6: Contact pair concept (Neves et al., 2014).

On the other hand, the methods considering the wheel and rail geometries are methodologies that can also be used to study the vehicle-structure interaction. In wheel-rail contact problems, an efficient definition of the geometric characteristics of the surfaces near the contact points is required. The normal and tangential forces (see Figure 2.7) depend on the geometries of the wheel and rail profiles, and in extreme conditions, such as strong lateral winds or earthquakes, the dynamic behaviour of the system is significantly affected.

Figure 2.7: External forces applied to the wheelset (N_i , T_i and F_i represent the normal, lateral and longitudinal forces, respectively, in contact point i) (Shabana et al., 2001).

Besides these geometric concerns, it is also necessary to use a fully nonlinear formulation. The formulations which take into account the geometry of wheel and rail profiles are the most accurate to deal with railway dynamics and allows the evaluation of the vehicle-structure interaction system not only in the vertical direction, but also in the lateral one.

These formulations can be classified in two different approaches, that are distinguished by the different computation of the normal contact forces. In 2001, Shabana et al. (2001) proposed his first approach to the problem - Constraint Contact Formulation. In this study, the kinematic contact constraint conditions are formulated in terms of the normal and tangents to the contact surfaces. By imposing these constraints using, for example, the Lagrangian Formulation, it is possible to eliminate one degree of relative motion between wheel and rail and thus the normal force can be calculated as a constraint force. Contrary to the previous approach, the Elastic Contact Formulation, studied by Antolín (2013), assumes the penetration between the wheel and rail. In this case, there is no elimination of any degree of freedom and the normal force is defined as a function of that penetration, using the normal contact theories, such as the Hertz theory (Hertz, 1882) or the Piotrowski and Chollet theory (Piotrowski and Chollet, 2005).

The algorithm, used to obtain the contact point position, is also a parameter that distinguishes some formulations. In this subject, there are two different approaches: Offline Contact Search and Online Contact Search. The first one implies an analysis of the geometry of the surfaces, that is previously performed, and the location of the contact points is precalculated and stored in a lookup table. In the latter formulation, the location of contact points is determined during the dynamic simulation using iterative procedures, at every time step. These two formulations are properly approached by Sugiyama et al. (2009).

2.3 Safety Regulations and Standards for European Railways Traffic

2.3.1 Introduction

In general, the safety regulations and standards are focused on structural safety, both in terms of ultimate limit states and service limit states. For ordinary operating conditions or extreme conditions, such as earthquakes or strong crosswinds, some recommendations regarding the running safety of the railway vehicles have been proposed so far.

Regardless the existence of distinct standards established in different countries, as United States of America and Japan, the present section only summarizes the main criteria regarding the stability and running safety of trains on railway viaducts, defined in the standards from Europe.

In Europe, the main criteria regarding the stability of the track and, consequently, the stability of railway vehicles, are presented in EN1990-A2 (2001) and in EN1991-2 (2003). The verifications defined by these standards are related to the control of deformations (see Section 2.3.3), and to control the vibrations on bridges. (see Section 2.3.4).

Before the presentation of the standards, it is presented a brief description of how to approach the dynamic effects in bridges according to Eurocode EN1991-2 (2003).

2.3.2 Dynamic Effects According to the Eurocodes

2.3.2.1 Dynamic Factor - Φ (Φ_1 , Φ_2) for Load Models

According to Eurocode EN1991-2 (2003), the dynamic factor takes into account the dynamic amplification of stresses and vibration effects in the structure, but does not take into account of resonance effects. For such cases, where there is a risk that resonance or excessive vibration of the bridge may occur (with a possibility of excessive deck accelerations leading to ballast instability etc. and excessive deflections and stresses etc.), a dynamic analysis shall be carried out to analyse the impact and resonance effects.

The definition of the dynamic factor is base on the following aspects EN1991-2 (2003):

- The dynamic factor Φ , which enhances the static load effects under LM71, SW/0 and SW/2 shall be taken as either Φ_2 or Φ_3 .
- The dynamic factor Φ is taken as either Φ_2 or Φ_3 according to the quality of track maintenance as follows:

- for carefully maintained track,

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \quad \text{with } 1.00 \leq \Phi_2 \leq 1.67, \quad (2.37)$$

- for track with standard maintenance,

$$\Phi_3 = \frac{2.16}{\sqrt{L_\Phi} - 0.2} + 0.73 \quad \text{with } 1.00 \leq \Phi_3 \leq 2.0, \quad (2.38)$$

where L_Φ represents the “determinant” length (associated with Φ) in meters defined in Eurocode EN1991-2 (2003);

- If no dynamic factor is specified, Φ_3 shall be used.
- The dynamic factor Φ shall not be used with:
 - the loading due to real trains;
 - the loading due to fatigue trains;
 - the load model High-Speed Train Load Model (HSLM);
 - the load model “unloaded train”.

2.3.2.2 Dynamic Factors $1 + \varphi$ for Real Trains

For real trains, Annex C of the Eurocode EN1991-2 (2003) provides the determination of the dynamic factors for real trains:

- To take account of dynamic effects resulting from the movement of actual service trains at speed, the forces and moments calculated from the specified static loads shall be multiplied by a factor appropriate to the Maximum Permitted Vehicle Speed.
- The dynamic factors $1 + \varphi$ are also used for fatigue damage calculations.
- The static load due to a Real Train at v (m/s) shall be multiplied by:
 - for carefully maintained track,

$$1 + \varphi = 1 + \varphi' + 0.5\varphi'', \quad (2.39)$$

- for track with standard maintenance,

$$1 + \varphi = 1 + \varphi' + \varphi'', \quad (2.40)$$

with:

$$\varphi' = \begin{cases} \frac{K}{K - K + K^4} & \text{for } K < 0.76 \\ 1.325 & \text{for } K \geq 0.76 \end{cases}, \quad (2.41)$$

where, $K = \frac{v}{2L_{\Phi}n_0}$ and,

$$\varphi'' = \frac{\alpha}{100} \left[56 e^{-\left(\frac{L_{\Phi}}{10}\right)^2} + 50 \left(\frac{L_{\Phi}n_0}{80} - 1 \right) e^{-\left(\frac{L_{\Phi}}{20}\right)^2} \right], \varphi'' \geq 0, \quad (2.42)$$

being:

$$\alpha = \begin{cases} \frac{v}{22} & \text{if } v \leq 22 \text{ m/s} \\ 1 & \text{if } v > 22 \text{ m/s} \end{cases}. \quad (2.43)$$

In the equations presented above, v represents the Maximum Permitted Vehicle Speed (m/s), n_0 represents the first natural bending frequency of the bridge loaded by permanent actions (Hz), L_{Φ} is the determinant length (m) and α is a coefficient for speed.

- The values of $\varphi' + \varphi''$ shall be determined using upper and lower limiting values of n_0 , unless it is being made for an individual bridge of known first natural frequency.

The upper limit of n_0 is given by:

$$n_0 = 94.76 L_{\Phi}^{-0.749}, \quad (2.44)$$

and the lower limit is given by:

$$n_0 = \begin{cases} \frac{80}{L_{\Phi}} & \text{for } 4 \text{ m} \leq L_{\Phi} \leq 20 \text{ m} \\ 23.58 L_{\Phi}^{-0.592} & \text{for } 20 \text{ m} < L_{\Phi} \leq 100 \text{ m} \end{cases}. \quad (2.45)$$

Thus, for the ultimate limit state the dynamic factor $1 + \varphi$ is calculated according Annex C of the Eurocode EN1991-2 (2003). In addition, the dynamic factor $1 + \varphi$ is also calculated for the fatigue assessment of railway structures. However, in this case, its determination is provided by Annex D of the Eurocode EN1991-2 (2003).

2.3.2.3 Requirement for a Static or Dynamic Analysis

To conclude this section, a flow chart is presented in Figure 2.8 and it introduces the requirements for determining whether a static or dynamic analysis is required when designing a railway bridge.

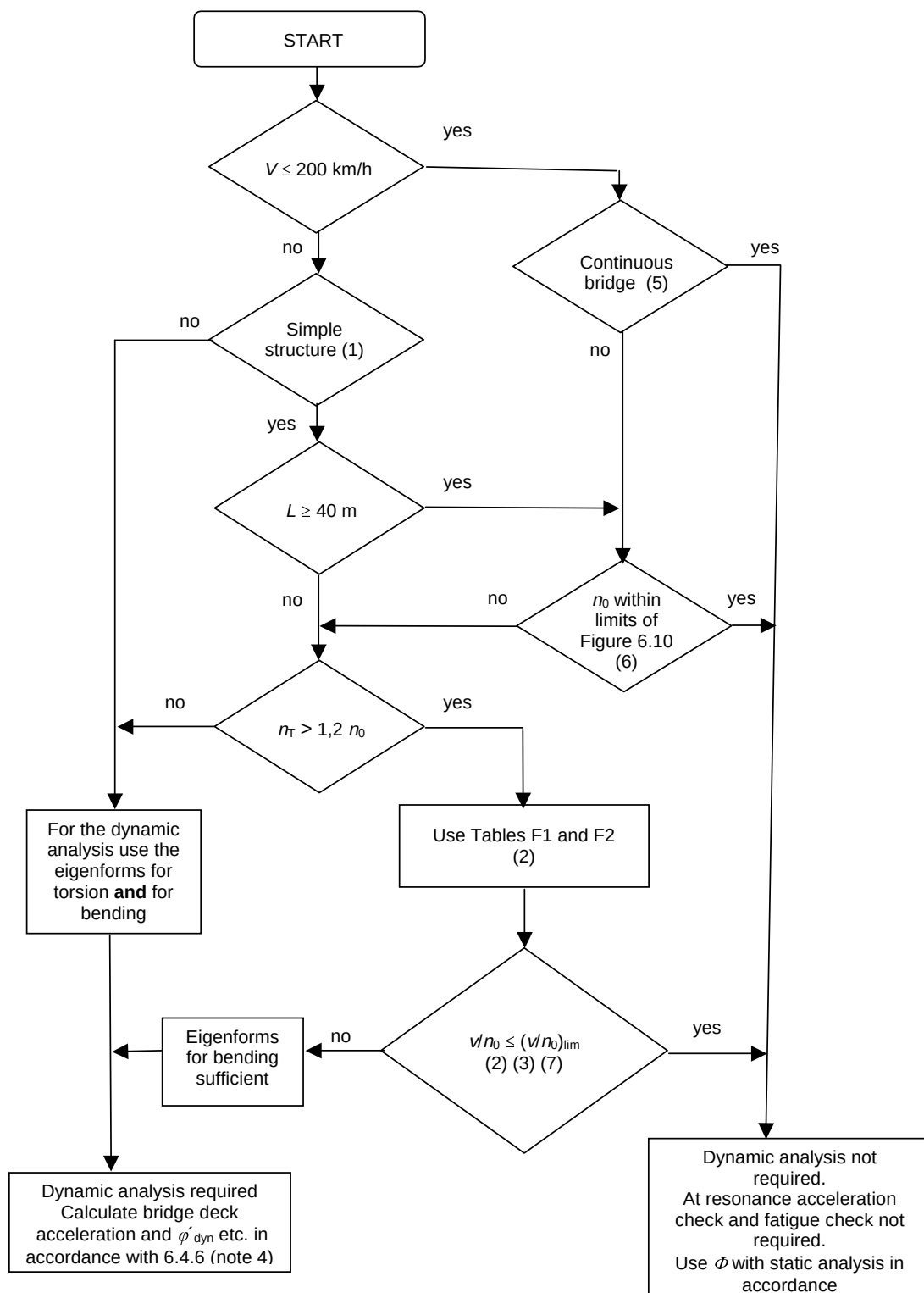


Figure 2.8: Flow chart for determining whether a dynamic analysis is required (EN1991-2, 2003).

2.3.3 Criteria Regarding the Bridge Deformation Control

2.3.3.1 Vertical Deflection of the Deck

According to EN1990-A2 (2001), the maximum total vertical deflection (δ_v) measured along the track, due to the characteristic values of the vertical traffic load models LM71 and SW/0, as appropriate, defined in EN1991-2 (2003), cannot exceed $\frac{L}{600}$, where L is the span length (see Figure 2.9).

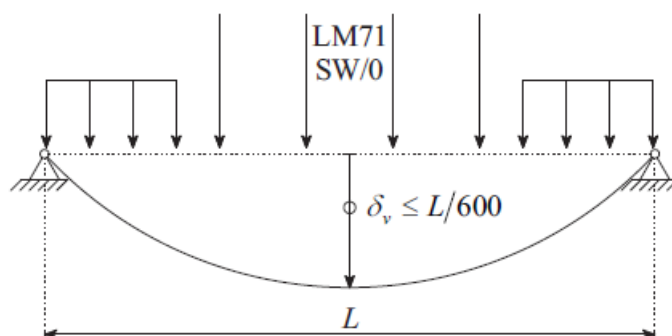


Figure 2.9: Vertical deflection of the deck (δ_v) (Montenegro, 2015).

2.3.3.2 Transverse deflection of the deck

According to EN1990-A2 (2001), the transversal deflection of the deck (δ_h) has to be limited to ensure that the angular variation and horizontal radius of curvature satisfy the limits specified in Table 2.2. The angular variations refer to the transversal rotations at the end of the deck (θ_h) or to the relative transversal rotations between two adjacent spans ($\theta_{h1} + \theta_{h2}$), as described in Figure 2.10. This condition has to be checked for characteristic combinations of: load model LM71 and SW/0, as appropriate, multiplied by the dynamic factor, wind loads, nosing force, centrifugal forces in accordance with EN1991-2 (2003) and the effect of transverse differential temperature across the bridge.

Table 2.2: Design limit values of angular variation and radius of curvature (adapted form EN1990-A2 (2001)).

Speed V (km/h)	Maximum Angular Variation (rad)	Maximum radius of curvature (m)	
		Single span	Multi-span
$V \leq 120$	0.0035	1700	3500
$120 < V \leq 200$	0.0020	6000	9500
$V > 200$	0.0015	14000	17500

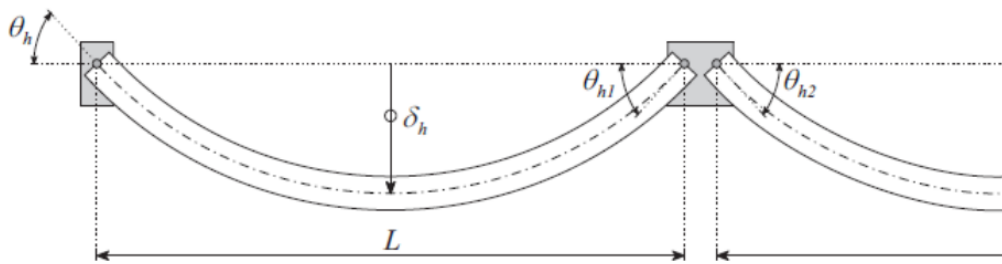


Figure 2.10: Transverse deflection of the deck (δ_h) and angular variations at the deck ends (θ_h) - plan view (Montenegro, 2015).

2.3.3.3 Deck Twist

The deck twist criterion defined in EN1990-A2 (2001) aims to minimize the risk of train derailment. The twist of the bridge deck is calculated taking into account the characteristic values of the load model LM71, as well as the load models SW/0 or SW/2, as appropriate, and the High-Speed Load Models including centrifugal effects, as defined in EN1991-2 (2003).

The maximum twist t (see Figure 2.11) of a track gauge of 1435 mm, measured over a length of 3 m, should not exceed the values represented in Table 2.3. The total twist due to any twist which may be present in the track when the bridge is unloaded, and that due to the total deformation of the bridge, may not exceed 7.5 mm/ 3 m.

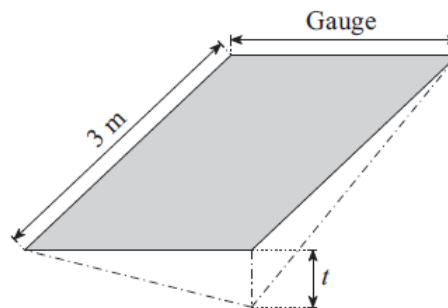


Figure 2.11: Definition of deck twist t (Montenegro, 2015).

Table 2.3: Design limit values of deck twist (EN1990-A2, 2001).

Speed V (km/h)	Maximum twist (mm/3m)
$V \leq 120$	$t \leq 4.5$
$120 < V \leq 200$	$t \leq 3.0$
$V > 200$	$t \leq 1.5$

2.3.3.4 Vertical Displacement of the Upper Surface at the End of the Deck

This requirement is intended to avoid destabilizing the track, to limit uplift forces on the rail fastening systems and to limit additional rail stresses. According to EN1991-2 (2003) (see Figure 2.12) , the vertical displacement of the upper surface deck relative to the adjacent construction (δ_v) due to variable actions, may not exceed:

- 3 mm for a maximum line speed at the site of up to 160 km/h.
- 2 mm for a maximum line speed at the site over 160 km/h.

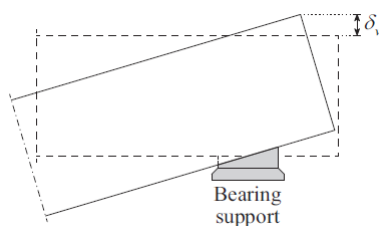


Figure 2.12: Vertical displacement of the upper surface of the deck (δ_v) (Montenegro, 2015).

2.3.3.5 Longitudinal Displacement of the Upper Surface at the End of the Deck

The longitudinal displacement of the upper surface at the end of the deck has to be limited to minimize disturbance to track ballast and adjacent track formation. EN1991-2 (2003) limits the longitudinal displacement of the deck (δ_b) relative to the adjacent abutment or relative to two consecutive decks. The limit is 5 mm for continuous welded rails without expansion devices and 30 mm for rails with expansion devices.

For vertical traffic loading defined by the load model LM71 and SW/0, as appropriate, the longitudinal displacement δ_h (see Figure 2.13) of the upper surface at the end of the deck may not exceed 8 mm, if the combined behaviour of structure and track is considered in the numerical model, otherwise the limit is 10 mm.

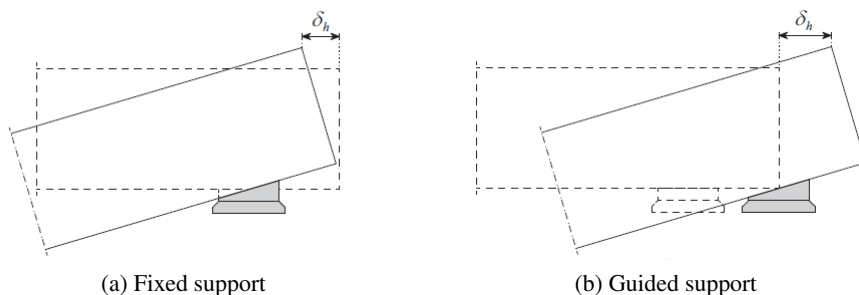


Figure 2.13: Longitudinal displacement of the upper surface of the deck δ_h (Montenegro, 2015).

2.3.4 Criteria Regarding the Bridge Vibration Control

2.3.4.1 Vertical Acceleration of the Deck

To ensure traffic safety, the verification of maximum vertical peak deck acceleration, due to the rail traffic loads, should be regarded as a traffic safety requirement at the serviceability limit state, for the prevention of track instability. Therefore, according to EN1990-A2 (2001), the maximum vertical acceleration allowed of the bridge deck should not exceed 3.5 m/s^2 , on ballasted tracks, and 5 m/s^2 , on slab tracks. The acceleration is calculated by a dynamic analysis, with real high-speed train models and with the load models HSLM, defined in EN1991-2 (2003), considering only one loaded track. In the calculations, only the contributions of the mode shapes with frequencies up to 30 Hz or to 1.5 times the frequency of the first mode of vibration of the element being analysed, including at least the first three modes, should be taken into account.

2.3.4.2 Lateral Acceleration of the Deck

According to EN1990-A2 (2001), to avoid the occurrence of resonance between the lateral motion of the vehicle and the bridge, the fundamental frequency of lateral vibration of a span should not be less than 1.2 Hz.

2.4 Fatigue in Metallic Bridges

2.4.1 Introduction

Fatigue life estimation of metal historical bridges is a key issue for managing cost-effective decisions, regarding rehabilitation or replacement of existing infrastructure. The increasing volume of traffic and axle weight of trains mean that the current loads are much higher than those envisaged when the bridge was designed (Pipinato et al., 2012).

In Europe, railway bridges are mainly metal made and have been designed and constructed during the last century. Naturally, it is important to underline that the fatigue damage is more severe in railway bridges than in road bridges, due to the magnitude of the acting dynamic loadings during their operating life, being particularly relevant when freight traffic is common. Such load cases, mainly due to the traffic action, are not constant through the lifetime, giving origin to a variable stress spectra history with multiple applied load cycles.

Bearing in mind the concerns exposed and, as said before, there are issues as maintenance, assessment, rehabilitation and strengthening of existing bridges that assume significant importance. Further, regarding the local structural details where fatigue problems are expected, metallic bridges can be classified into riveted bridges, welded and bolted bridges.

Since mechanical fatigue is the more common type of fatigue in railway bridges, this work mainly focuses on that problem. Nevertheless, it is important to recall that there are other types

of fatigue that are undesirable, such as corrosion fatigue. Corrosion fatigue is a high rate degrading process which occurs in environmentally adverse conditions, such as those affecting offshore structures (ABS, 2014).

Several authors, as Furukawa and Murakami (1999), studied this problem and assume fatigue as the main cause of structural failures. During their operating life, many bridges suffered from a loss of structural integrity or complete structural collapse. The comprehension of the reasons for such accidents has been accompanying the understanding of the fatigue phenomenon.

2.4.2 Fatigue Phenomenon

The fatigue corresponds to a limit state of the metallic material from which it loses the capacity to resist the applied actions, even if they induce the material to a state of low tension, when compared to its yielding strength. This sudden loss of strength is due to the fact that the metal does not have an ideal linear elastic behavior. It is important to refer that in each loading and unloading cycle occur deformations that are not fully recovered. Hence, during the time that the material is in service, it accumulates an history of deformations. This accumulated damage results in microcracking and consequent propagation until the structural element completely loses its resistant capacity reaching rupture.

The fatigue problem consists on the initiation, stable propagation and, ultimately, unstable propagation of cracks, in structural details when subjected to cyclic loading. Therefore, this is a problem whose dimensional scale spans several orders of magnitude. It usually starts at the material crystals level (order of nanometres) and it may extend to the dimension of the structural elements (order of meters). The initiation process includes, as mentioned before, the appearance of microcracks. After that, the cracks propagation is stable and the cracks remain invisible to the naked eye. In the last phase, the crack starts to grow in a unstable way until the total failure (Schijve, 2001). To clarify these different phases, each one of them is approached below.

1. Crack Initiation (Physical)

The fatigue crack initiation process is the result of cyclic slip in bands when a certain metallic specimen is submitted to cyclic loading, which leads to local stress amplitudes below the yield stress. At such low stress level, it is developed a micro-plasticity in a small number of material grains. Naturally, this process tends to occur at the material surface because there is only supporting material on one side, which means that the plastic deformation in such area is less constrained than in the interior grains.

2. Stable Crack Propagation

The process of microcrack propagation is controlled by the acting local stress range magnitude, which is directly related to the stress concentration factor, K_t . The macrocrack growth initiates when the crack orientation is perpendicular to the loading direction. The size of the microcrack at the transition from the initiation period to the crack growth period will be significantly different for distinct types of materials (Schijve, 2001). Hence, the

propagation is no longer controlled by the local stress magnitude, being the parameter that defines the stable crack propagation the stress intensity factor, K . That propagation occurs slowly, until the maximum stress intensity factor, K_{max} , reaches the material toughness, K_C , or until another rupture mechanism manifests. The striation formed by the successive loading cycles is a characteristic phenomenon that occurs in this phase of the fatigue life.

3. Unstable Crack Propagation

When a fatigue crack in a load-carrying member is allowed to progress, the final collapse certainly will take place at some stage of crack growth (disregarding other types of limit states which may have been reached before final failure). Depending on the toughness of the material, temperature, loading rate, plate thickness and constraints, such critical event may occur essentially by three mechanisms (brittle fracture, ductile fracture and plastic collapse) (Almar-Naess, 1985).

If the fracture is ductile, this phenomenon happens due to cavitation and coalescence of internal micropores, from particles on suspension at the material matrix. Naturally, a rupture of this type consists in the development of a significant plasticity region which is responsible for an important energy absorption. A high plastic deformation is allowed and it may be a warning for a possible global or local structural failure. Regarding the plastic collapse, the energy dissipation is even higher being such failure mode characterized by extreme ductile characteristics. On the other hand, the fracture mechanism observed on the brittle fractures is cleavage. The fracture surface presents plane steps and facets. The energy absorbed during this process is much lower than that absorbed on a ductile rupture, and that is the reason why the brittle fractures may progress in an unstable and sudden way (Albuquerque, 2015).

2.4.3 Examples of Fatigue Damage in Metallic Bridges

A serious accident caused by fatigue occurred in 2007, in Minneapolis, state of Minnesota (USA). A bridge, named *I – 35W* (see Figure 2.14), was opened to the traffic in 1967 and, according to studies developed in 2004, the average daily traffic was around 141000 vehicles, being 5640 of them heavyweight vehicles. These numbers proved a significant increase in traffic in the bridge, since in 1976 it was only used by an average of 60600 vehicles (NTSB, 2008). The accident happened in the afternoon on October 1, in 2007. Videos recorded during the catastrophe show the separation of the main route and the rest of the structure.

Before finding the true cause of the collapse, a team of experts considered distinct possible theories. Between them it was considered the corrosion of the gusset plates at certain nodes; the pre-existing fractures in the tray of the truss, or even in the access bay; the effects of temperature and the supposed movement of the pillars. All these theories were rejected when, after an exhaustive investigation, the team concluded that the cause of the collapse was the inappropriate load capacity of a specific gusset. These plates had half of the thickness of what was needed and there were possible factors that led to this issue (NTSB, 2008):

- insufficient quality control procedures by design firms;
- insufficient procedures for project review and approval by project owners;
- lack of recommendations regarding the loadings during maintenance and repair work;
- exclusion of gusset plates when the load capacities of existing bridges were evaluated, according to the recommendations;
- lack of recommendations regarding the inspection of the gusset sheets condition;
- inadequate use of technologies for an accurate assessment of the gusset plates condition in trays of truss bridges.



Figure 2.14: Bridge I-35W, Minneapolis, Minnesota, USA, 2006 (photographed by I. Elkman).

A replacement bridge, named I-35W Saint Anthony Falls Bridge, was built and it was opened to the public on September 18, 2008.

The partial failure of the Sgt. Aubrey Cosens VC Memorial Bridge that occurred on January 14, 2003 was another serious accident. This steel arch bridge is located in Latchford and spans the Montreal River (see Figure 2.15). As a Southbound tractor-trailer crossed the bridge, the concrete deck deflected approximately 2 meters at the northwest corner. The bridge was immediately closed to traffic. This failure was caused by the fatigue-induced fracture of three steel hanger rods on the northwest side of the bridge. The failure of these rods can be attributed to a combination of factors (Bagnariol, 2003):

- the original design did not consider that the pins in the hangers could seize and cause bending fatigue stresses in the rods. The bending fatigue stresses led to the eventual fracture of the rods;
- the threaded portion of the rods was damaged during construction 40 years ago;

- the quality of the steel does not meet current standards for ductility in cold temperatures and chemical composition;
- the critical parts of the hanger rods were hidden from inspection since they were inside the arch.

The hanger rods transferred the vertical load to the arch and the longitudinal movements of the deck induced their bending due to a deficient operation of the superior ball joint (Akesson, 2008). This bending caused a non-uniform distribution of forces on the superior ball joint, which contributed to its deterioration. On the connection near the arch, the hanger rods stiffness increased substantially. The deformations induced by the secondary bendings (in longitudinal direction) were low, however in the long term, these deformations caused serious problems, as the cracks initiation. These type of cracks would be fundamental to the hanger rods collapse.



Figure 2.15: Northwest corner of Sgt. Aubrey Cosens V.C. Memorial Bridge (photographed by Cameron Bevers).

Metallurgical analysis of the corrosion product on the “fast fracture” face of hanger 1 (at the northwest) indicated that there was a high probability that it had fractured 5 to 7 years prior to January 14, 2003. This hanger rod would have fallen, but after the drop by about 75 mm, it was held by the beam next to the bridge deck. The threaded part of hanger 1 showed signs of abrasion and wear caused by rubbing against the steel sleeve (up and down motion), as traffic crossed the bridge. Prior to final fracture, a portion of the hanger rod would have been cracked as a result of bending fatigue stresses. Once hanger 1 had failed, the load was transferred to the neighboring hangers and to the abutment by the steel stringers, floor beams and concrete deck system. Continued loading resulted in fatigue cracks developing in hangers 2 and 3 and in the floor beam to stringer connection plate. Hanger 2 failed 1 to 3 years prior to the accident and, consequently, hanger 3 and adjacent hangers were forced to carry the entire additional load from failed hangers 1 and 2. However, hanger 3 was incapable of carrying this additional load and failed. This led to the partial collapse of the deck on January 14, 2003 (Bagnariol, 2003).

The bridge was finally repaired and opened to traffic in 2005. That time, it was decided to use cables instead of hangers.

2.4.4 Fatigue Assessment Procedure for Existing Steel Bridges

For the fatigue assessment a step-by-step procedure is proposed. This procedure is based on a general procedure developed by the Joint Committee for Structural Safety (JCSS, 2001), which has been enhanced with focus on existing steel bridges exposed to fatigue loading. If the assessment by this procedure proves a sufficient remaining fatigue life in one phase, the subsequent phases can be disregarded. The following phases for the assessment are proposed (Kühn, 2013):

- **Phase I - Preliminary evaluation:** The aim of the preliminary evaluation is to remove existing doubts about the safety of the structure using fairly simple methods and identify critical parts or members in the structure. At this stage, the assessment is carried out by the engineer alone. In order to identify critical members (fatigue critical construction details), it is necessary to carry out an intensive study of the available documents, along with a visual inspection of the structure. Sometimes, the available documents are incomplete and it may also be determined that there were little (if any) fatigue calculations done at the time of design. In all cases, even when there is significant documentation available, it is important to perform an inspection of the bridge. For this phase, preliminary evaluation should be conducted as for design of a new structure, that is by using current codes and recommendations and making conservative assumptions where information is lacking or doubtful. In this manner, critical construction details can be identified. Calculations such as this are quick and give a good estimation of the safety level of each detail on the bridge;
- **Phase II - Detailed investigation:** The aim of the detailed investigation is to update the information obtained in Phase I by carrying out refined assessments only for those members for which adequate safety was not confirmed by the preliminary evaluation. At this stage, in addition to the engineer, a specialised firm, agency or individual experts may be consulted. Once the critical construction details are known, e.g. the lower f_{at} values, a calculation of the remaining fatigue life can be made. This calculation normally takes the form of a damage accumulation calculation;
- **Phase III - Expert investigation:** For problems with large consequences in terms of risks or of costs related to a decision, a team of experts should be called in order to carefully check the conclusions and proposals reached in Phase II. Discussions and further assessments using specific tools can also be carried out to help in reaching decisions;
- **Phase IV - Remedial measures:** If none of the assessments, described in Phases II and III, provide ample justification for leaving bridge in service “as is” (i.e. continuing with standard monitoring), then a suitable remedial measures must be implemented. The possibilities include strengthening, repair, intensification of monitoring, reduction of traffic loads or volume, and for the worst cases: demolition of the structure.

In Figure 2.16, it is proposed a stepwise procedure that aims on the identification of the best strategy for the optimal investment of life cycle costs.

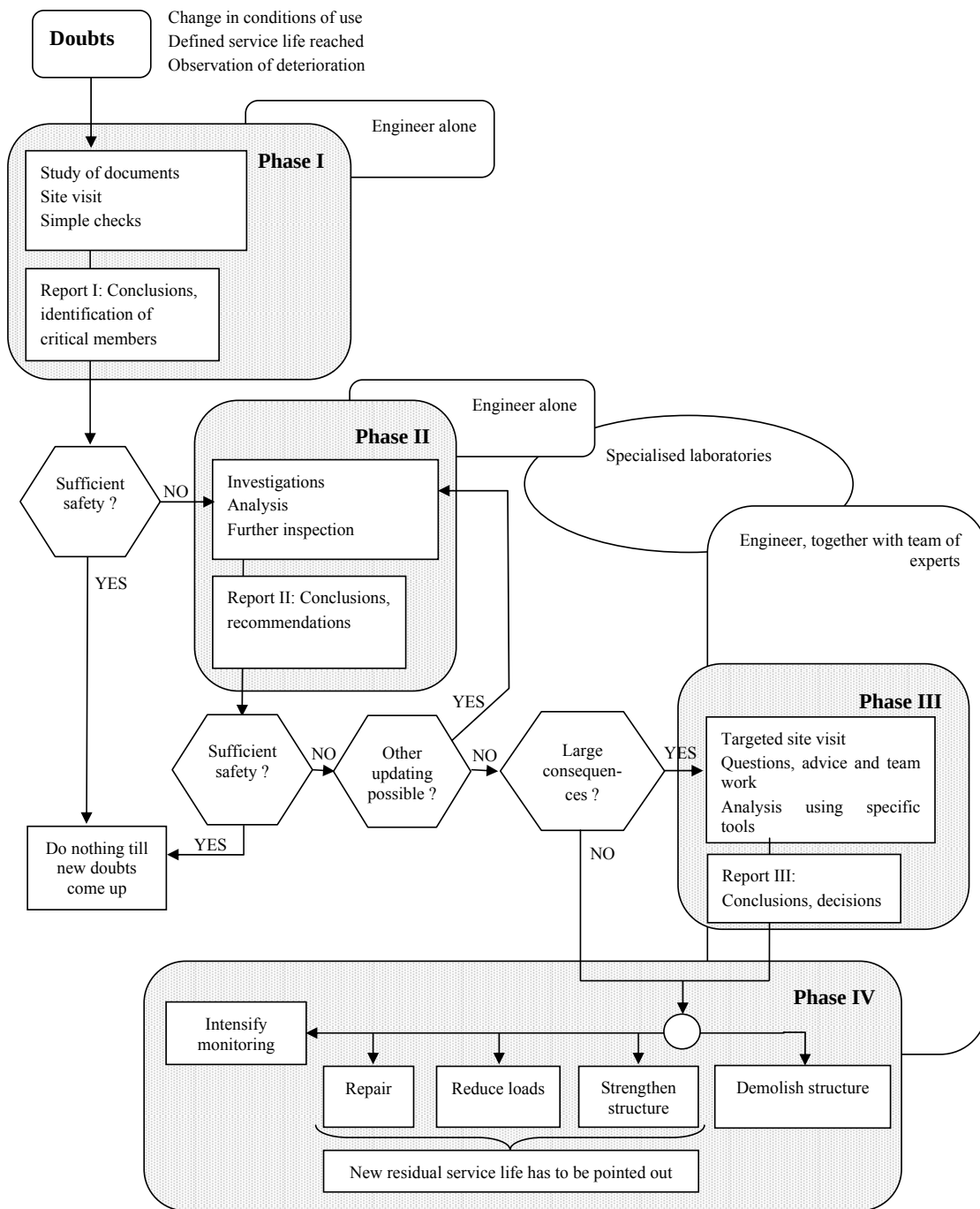


Figure 2.16: Stepwise procedure for fatigue assessment (Kühn et al. (2008)).

2.4.5 Code Procedures for fatigue Assessment of Railway Bridges

2.4.5.1 Introduction

The fatigue assessment usually involves two fundamental stages: fatigue actions and fatigue resistance of structural details. The knowledge of the actions is important to simulate the stress histories at locations prone to fatigue. Both fatigue actions and fatigue resistance data usually

shows various sources of uncertainties. The load information is normally provided by the responsible authorities or is based in existing standards and monitoring activities. Concerning the fatigue resistance assessment, usually it is defined in the form of experimental S-N curves, as shown in Section 2.4.5.4.

In this work, the standard procedures studied are described in Eurocodes that result from an effort of compiling and combining the knowledge on structural design, in European territory. Thus, the most relevant Eurocodes that provide the fatigue analysis of structural components, as the reinforced concrete and composite steel-concrete bridges, are presented bellow:

- Eurocode 0: EN 1990 – Basis of structural design. Annex A2 – Bridges;
- Eurocode 1: EN 1991 – Actions in structures. Part 2 – Traffic loads in bridges;
- Eurocode 2: EN 1992 – Design of concrete structures. Part 2 – Concrete bridges, Design rules and Part 2 - Annex NN – Damage equivalent stresses for fatigue assessment;
- Eurocode 3: EN 1993 – Design of steel structures. Part 1.9 - Fatigue and Part 2 – Steel Bridges;
- Eurocode 4: EN 1994 – Design of concrete-steel composite structures. Part 2 – Composite bridges.

2.4.5.2 Standard Fatigue Loads and Load Cases

EN1991-2 (2003) establish that the fatigue assessment should be performed, using one of the standard traffic scenarios described there. These traffic scenarios correspond to different combinations of 12 standard fatigue trains described in Table 2.4. The load scheme relative to fatigue train type 1 is presented in Figure 2.17 for exemplification, while the others train models schemes may be found in EN1991-2 (2003).

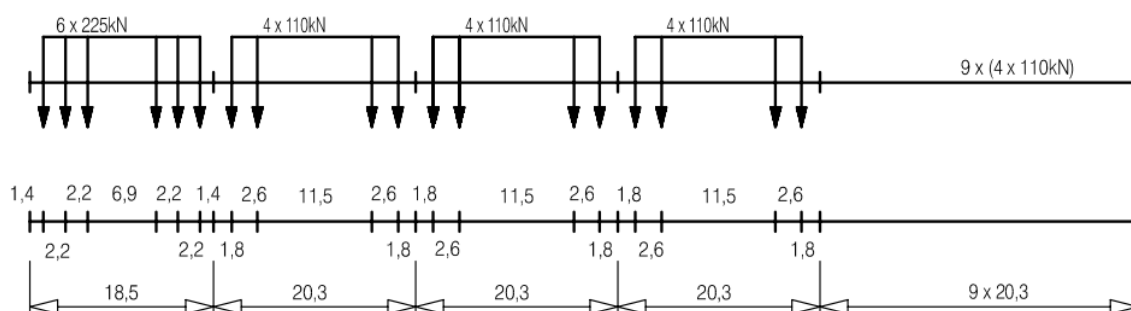


Figure 2.17: Example of a fatigue train model: Type 1 - Locomotive-hauled passenger train (EN1991-2, 2003).

The scenario selected must be in agreement with the type of traffic forecasted for the railway bridge studied: standard traffic mix with axles ≤ 225 kN (see Table 2.5), heavy traffic mix with axles = 225 kN (see Table 2.6) and light traffic mix with axles ≤ 225 kN (see Table 2.7).

Table 2.4: Standard fatigue trains (adapted from EN1991-2 (2003)).

Train	Train type	Speed (km/h)	Length (m)	Load (kN/m)	Max. axle load (kN)	Mass of train (kN)
1	Locomotive-hauled passenger	200	262.10	25.30	225	6630
2	Locomotive-hauled passenger	160	281.10	18.90	225	5300
3	High speed passenger	250	385.52	24.40	200	9400
4	High speed passenger	250	237.60	21.50	170	5100
5	Locomotive-hauled freight	80	270.30	80.00	225	21600
6	Locomotive-hauled freight	100	333.10	43.00	225	14310
7	Locomotive-hauled freight train	120	196.50	52.70	225	10350
8	Locomotive-hauled freight	100	212.50	48.70	225	10350
9	Suburban multiple unit	120	134.80	22.00	130	2960
10	Underground	120	129.60	27.80	150	3600
11	Locomotive-hauled freight	120	198.50	57.20	250	11350
12	Locomotive-hauled freight	100	212.50	53.40	250	11350

Table 2.5: Standard traffic mix (adapted from EN1991-2 (2003)).

Train	Train type	Number of trains/day	Mass of train (kN)	Traffic volume (10 ⁶ t/year)
1	Locomotive-hauled passenger	12	6630	2.90
2	Locomotive-hauled passenger	12	5300	2.32
3	High speed passenger	5	9400	1.72
4	High speed passenger	5	5100	0.93
5	Locomotive-hauled freight	7	21600	5.52
6	Locomotive-hauled freight	12	14310	6.27
7	Locomotive-hauled freight	8	10350	3.02
8	Locomotive-hauled freight	6	10350	2.27
Total		67		24.95

Table 2.6: Heavy traffic mix (adapted from EN1991-2 (2003)).

Train	Train type	Number of trains/day	Mass of train (kN)	Traffic volume (10^6 t/year)
5	Locomotive-hauled freight	6	21600	4.73
6	Locomotive-hauled freight	13	14310	6.79
11	Locomotive-hauled freight	16	11350	6.63
12	Locomotive-hauled freight	16	11350	6.63
Total		51		24.78

Table 2.7: Light traffic mix (adapted from EN1991-2 (2003)).

Train	Train type	Number of trains/day	Mass of train (kN)	Traffic volume (10^6 t/year)
1	Locomotive-hauled passenger	10	6630	2.4
2	Locomotive-hauled passenger	5	5300	1.0
5	Locomotive-hauled freight	2	21600	1.4
9	Suburban multiple unit	190	2960	20.5
Total		207		25.3

According to EN1991-2 (2003) and EN1993-2 (2006), when the standard traffic mixes adequately reproduce the real traffic and no dynamic analysis is required, the fatigue analysis may be performed using the equivalent constant amplitude stress range method. Thus, the loading to consider is the load model LM71, presented in Figure 2.18, including the dynamic factor, Φ , but excluding the load classification factor, α .

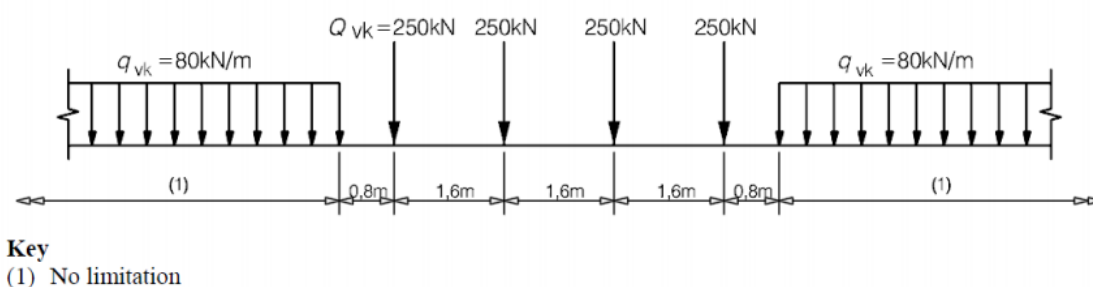


Figure 2.18: LM71 load model (EN1991-2, 2003).

In situations when the real traffic can not be represented by the standard traffic scenarios previously presented, an alternative traffic scenario should be specified, e.g. a traffic mix composed of real trains.

Regarding the loading direction, the loads to be considered are the vertical and centrifugal forces. Nosing and longitudinal forces may be neglected. When dynamic analyses are required, there are additional requirements for fatigue assessment.

2.4.5.3 Determination of Actuating Stresses

The methodologies proposed by EN1993-1-9 (2004) for fatigue assessment are based mainly on nominal stress but also on structural stress. This standard defines three different acting stresses:

- Nominal stress - It is calculated in accordance with the elastic theory, in the parent material or in a weld adjacent to a potential crack location excluding local concentration effects (see Figure 2.19);

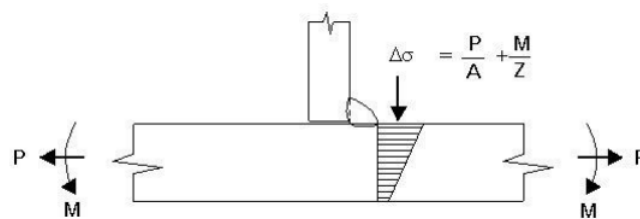


Figure 2.19: Nominal stress example (ESDEP, 1996).

- Geometrical stress - The maximum principal stress in the parent material adjacent to the weld toe, taking into account stress concentration effects due to the overall geometry of a particular constructional detail (see Figure 2.20);

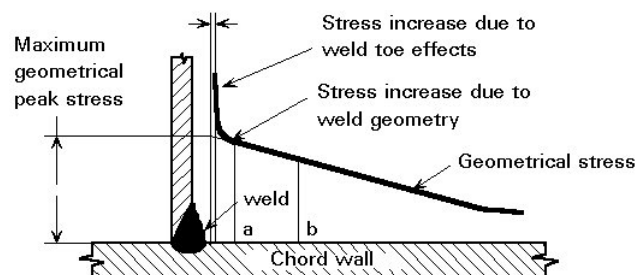


Figure 2.20: Geometrical stress (ESDEP, 1996).

- Modified nominal stress - is based on the value of the nominal stress multiplied by an appropriate stress concentration factor, k_f , in order to consider a geometric discontinuity that has not been taken into account, in the classification of a particular structural detail (see Figure 2.21).

The computation of the mentioned three type of stresses should be performed considering a linear elastic material behaviour. The adoption of refined finite element meshes, which has been possible due to the progressive development of the computational efficiency, has been allowing to increase the accuracy in the determination of the local stress history (Horas, 2017).

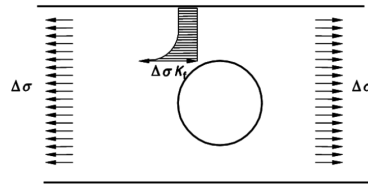


Figure 2.21: Modified nominal stress example (Albuquerque, 2015).

2.4.5.4 Fatigue Strength

To evaluate the fatigue strength of steel connections, the EN1993-1-9 (2004) provides guidelines for the assessment of the fatigue phenomenon in steel structures. According to EN1993-1-9 (2004), the fatigue strength of structural steel components is given by series of S-N curves, $\log(\Delta\sigma_r)$ vs. $\log(N)$ and $\log(\Delta\tau_r)$ vs. $\log(N)$, categorised by three characteristic stress ranges:

- $\Delta\sigma_D$ - Constant amplitude fatigue limit: defined as the fatigue strength for direct stresses evaluated at 5×10^6 constant amplitude loading cycles. When variable amplitude loading occurs, no damage will be observed if all the stress cycles have a range lower than this value;
- $\Delta\sigma_L$ or $\Delta\tau_L$ - Cut-off limit: is the value of direct or shear stress range, respectively, below which cycles do not contribute to the damage accumulation of fatigue damage;
- $\Delta\sigma_C$ or $\Delta\tau_C$ - Detail category: is the value that identifies each S-N curve and can be understood as the fatigue strength at 2×10^6 constant amplitude loading cycles.

The S-N curves presented in EN1993-1-9 (2004) are applicable to compute the fatigue considering nominal and structural stresses (see Figure 2.22). This Eurocode also presents safety factors that must be applied.

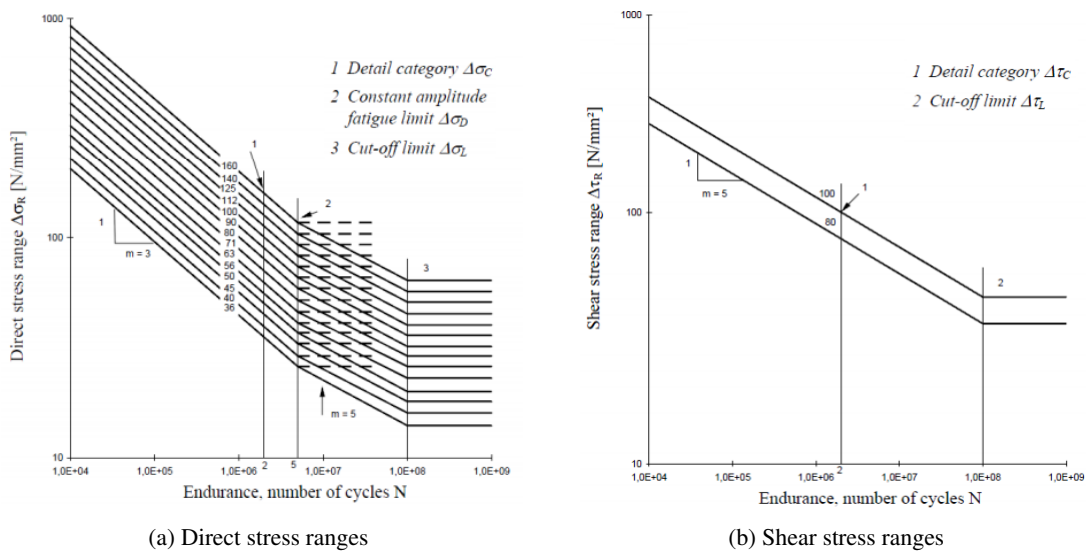


Figure 2.22: S-N curves for different stress ranges presented in EN1993-1-9 (2004) .

2.4.5.5 Assessment Methods According to the Standards

As previously mentioned, the Eurocodes propose two different methodologies to analyse the fatigue strength of a structural member: the equivalent constant amplitude stress range method and the linear damage accumulation method.

The Equivalent Constant Amplitude Stress Range Method may be applied when no dynamic analysis is required if some requirements are fulfilled, and simplifications are possible to be considered. In other words, the bridge should fit the requisites that allow avoiding the dynamic analysis. Furthermore, the real traffic must be adequately reproduced by the standard traffic mix scenarios presented in Section 2.4.5.2.

According to EN1993-2 (2006), the equivalent constant amplitude stress, $\Delta\sigma_{E_2}$ $\Delta\tau_{E_2}$, is associated with 2×10^6 cycles:

$$\Delta\sigma_{E_2} = \lambda \Phi_2 \Delta\sigma_{71}, \quad (2.46)$$

$$\Delta\tau_{E_2} = \lambda \Phi_2 \Delta\tau_{71}. \quad (2.47)$$

where, λ is the damage equivalence factor, $\Delta\sigma_{71}$ and $\Delta\tau_{71}$ represent the maximum stresses amplitudes due to the load model LM71 and Φ_2 is the respective dynamic amplification factor.

The damage equivalence factor, λ , for railway bridges with a span up to 100 m, should be determined according to the following expression (EN1993-2, 2006):

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \leq 1.40 \quad (2.48)$$

where, λ_1 represents the factor which considers the type of traffic and which depends on the length of the influence line of the detail under analysis, λ_2 represents factor that accounts for different volumes of traffic (traffic intensities), λ_3 represents the factor that accounts for the design life of the structure and λ_4 represents the factor accounting for simultaneous traffic, in bridges that have more than one track.

For each partial factor, EN1993-2 (2006) propose a procedure to compute them. The fatigue assessment, according to the equivalent constant amplitude stress range, consists in the verification of the following conditions (see Figure 2.23):

$$\gamma_{F_f} \Delta\sigma_{E_2} \leq \frac{\Delta\sigma_C}{\gamma_{M_f}}, \quad (2.49)$$

$$\gamma_{F_f} \Delta\tau_{E_2} \leq \frac{\Delta\tau_C}{\gamma_{M_f}}. \quad (2.50)$$

where, γ_{F_f} is the partial factor for equivalent constant amplitude stress range, associated with fatigue loading, being equal to 1 if no other value is indicated by the national annex.

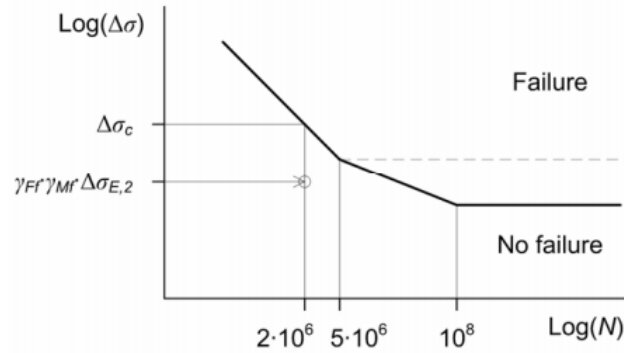


Figure 2.23: Preliminary evaluation of fatigue safety level (Kühn et al., 2008).

When there is a simultaneous action of direct and shear stresses, $\Delta\sigma_{E2}$ and $\Delta\tau_{E2}$, the following condition should be satisfied:

$$\left(\frac{\gamma_{Ff} \Delta\sigma_{E2}}{\Delta\sigma_C / \gamma_{Ff}}\right)^3 + \left(\frac{\gamma_{Ff} \Delta\tau_{E2}}{\Delta\tau_C / \gamma_{Ff}}\right)^5 \leq 1. \quad (2.51)$$

On the other hand, the Linear Damage Accumulation Method is the most comprehensive method for the fatigue analysis present in the Eurocodes. It can be used in most of the cases, but its applicability may be conditioned by the type of details existing in the structure to be analysed, since the range of details covered by the Eurocodes is limited. Due to the fact that it requires to obtain the response at the critical detail, the method may be time consuming (Albuquerque, 2015).

The accumulation of the fatigue damage is evaluated using the Palmgren-Miner rule applied to the several stress amplitude ranges, $\Delta\sigma_i$, computed from the stress history (EN1993-1-9, 2004):

$$D = \sum \frac{n_i}{N_i} \leq 1 \quad (2.52)$$

where, D represents the accumulated fatigue damage considering a certain cyclic loading, n_i represents the number of load cycles at $\Delta\sigma_i$ and N_i represents the fatigue life expressed in cycles of the detail submitted to the same stress range.

2.4.6 Advanced Methodologies for the Fatigue Assessment of Structures

The fatigue analysis in Mechanical Engineering is usually based on S-N curves and in the concept of linear damage accumulation. The Eurocodes, presented in Section 2.4.5, as most of the standards, use this methodology, presenting categories of details and the corresponding fatigue strength curves and has the advantage of having a easy implementation and application. However this methodology has several limitations related to the correct consideration of the local characteristics of the geometry, loading and materials. In order to solve these constraints, several advanced approaches more have been proposed which tried to take into account the local characteristics of geometry, loading and materials. These approaches are based on:

- **Stress-life Methods:**

- Nominal Stress Method

This approach is based on extensive tests of welded joints and connections. Weld joints are classified by type, loading and shape and these stresses of interest can be calculated using linear elastic structural mechanics for simple structural components or finite element method for relatively complicated structures.

- Hot Spot Stress Method

The structural hot spot stresses include nominal stresses and stresses from structural discontinuities and the presence of attachments, but do not include stresses due to the presence of welds. The structural hot spot stress determination according to the International Institute of Welding (IIW, 2016) uses reference points and extrapolation to the weld toe at the hot spot in consideration.

- Notch Stress Method

In contrast to the nominal stress and the hot spot stress approaches, this method is focused on the fatigue crack initiation life assessment at the root of a notch. The simplest assessment procedure, from those based on notch stress, consists in assuring infinite life to fatigue, which means that the crack initiation must be avoided.

- **Notch Strain Method**

The notch strain approach allows assessing the fatigue life up to the fatigue crack initiation. It uses a history of elasto-plastic strains at the root of the notch and compares it with the strain S-N curve of the material, obtained by testing several un-notched specimens. The local approaches, recognizing the localized nature of the fatigue damage to assess the fatigue life, develop the correlation of the local response amplitude (stress or strain) with endurance index to failure (number of cycles) (Liu et al., 2018). The elastic and plastic strain amplitude may be deeply described by the Basquin law (Basquin, 1910) and the Manson-Coffin law (Manson, 1965).

- **Fracture Mechanics and Crack Propagation Laws.**

Griffith (1921) had the idea that the discrepancy between the theoretical and real values were explained by the presence of flaws, even if they were small. Basing on results obtained in experiences with glass, Griffith proved that if the inexistence of fissures is guaranteed, the strength observed was close to the theoretical strength. The difficulties were in explaining how the presence of cracks justifies the lack of fracture strength. Griffith introduced an artificial flaw in his experimental specimens and it was in the form of a surface crack which was much larger than other flaws in a specimen. With this experience, Griffith concluded that the product of the square root of the flaw length a , and the stress at fracture σ_f was nearly constant, which is expressed by the equation:

$$\sigma_f \sqrt{a} = C . \quad (2.53)$$

Later, Irwin suggested a modification in the theory proposed by Griffith, that was ignored by the engineering community until the early 1950. The modifications proposed by Irwin appear to be in accordance with the actual structural materials theory. The level of energy needed to cause fracture is orders of magnitude higher, than the corresponding surface energy proposed by Griffith. This is justified by the fact that there are always inelastic deformations around the crack tip that would make the use of linear elastic theory at the crack tip, highly unrealistic. As the applied load increases, the plastic zone increases in size, until the crack grows and elastic strain energy is released as a crack grows. The plastic loading and unloading cycle near the crack tip leads to the dissipation of energy as heat. Hence, a dissipative term has to be added to the energy balance relation devised by Griffith for brittle materials. In physical terms, additional energy is needed for crack growth in ductile materials when compared to brittle materials (Ribeiro et al., 2011).

Approaching Irwin's theory is important to understand that the stored elastic strain energy, which is released as a crack grows, represents the thermodynamic driving force for fracture and the dissipated energy, which according to Orowan (1949) includes plastic dissipation and the surface energy, represents the dissipated energy provides the thermodynamic resistance to fracture. The total energy dissipated is represented by G , as follows:

$$G = 2\gamma + G_p, \quad (2.54)$$

where γ represents the surface energy and G_p represents the plastic dissipation energy of crack growth. Later, Orwan and Irwin clarified that, since the linear elasticity methods can be performed, the fracture resistance is given by the value of G (Branco et al., 1999):

$$\sigma_f \sqrt{a} = \sqrt{\frac{EG}{\pi}}. \quad (2.55)$$

In 1958, Irwin proposed the concept of the stress-intensity factor, given by the following expression:

$$K = \sigma \sqrt{\pi a}, \quad (2.56)$$

where, K is the stress intensity factor, σ is an applied stress and a is the crack length. If K reaches a certain critical value depending on the "fracture toughness" of the material, the collapse occurs. The critical fracture toughness value can be defined in terms of the stress intensity factor, K , as follows:

$$K_{IC} = Y \sigma \sqrt{\pi a}, \quad (2.57)$$

where Y is a geometrical factor that depends on both specimen and crack geometry.

This concept, where the stress intensity factor, K , is used to predict the stress state "stress intensity", near the tip of a crack caused by a remote load or residual stresses. When this stress state becomes critical, the crack grows and the material fails (Ribeiro et al., 2011).

Westergaard developed the analytical technique to obtain stresses and strains in cracked materials. The problem handled by Westergaard consists in the determination of the stress state at the far end of a crack with a length of $2a$ on an infinite plate loaded by the stress σ , as represented in Figure 2.24.

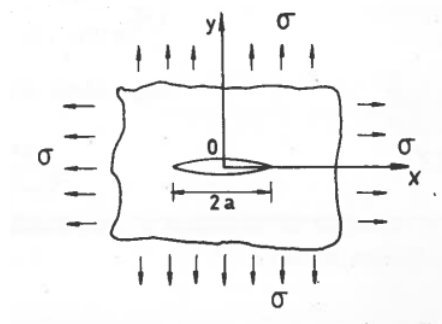


Figure 2.24: Plate with a central crack (Branco et al., 1999).

These methodologies presented have their own scope of application with some degree of overlapping among them.

Chapter 3

Dynamic Analysis and Fatigue Assessment Methodologies

3.1 Introduction

This chapter purpose is to describe the methodologies used to perform the dynamic and fatigue analyses. The calculations were performed not only in ANSYS® (2018), but also in MATLAB® (2017).

Section 3.2 aims to expose the methodology used to perform the dynamic analysis of the bridge. The methodology is based on moving load models, in which the dynamic response of the bridge is obtained through the modal superposition method. At the end of this section, a flow chart is presented to better understand the calculation process.

In Section 3.3, the S-N curves used in this work are presented and the partial safety factor is described. The section ends with the description of the methodology used to analyse the fatigue strength.

3.2 Methodology for Performing the Dynamic Analysis

To obtain the response of the structure, several routines were developed in ANSYS® in order to extract modal quantities (modal displacements) which are not load-dependent (see Section 3.2.1). Later on, the modal coordinates are calculated using MATLAB® taking into account the specific loading type. With the modal coordinates obtained, it is possible to evaluate the final response of the structure applying the modal superposition method. All the routines used in the dynamic analysis methodology were developed by Horas et al. (2018). The following sub-sections aim to explain in more detail the whole process for calculating the dynamic response of the structure.

3.2.1 Modal Quantities Evaluation

The procedure used in the present thesis allows the evaluation of any type of modal quantity, namely modal displacements, modal internal forces, modal stresses, etc. Table 3.1 presents the

functions developed in ANSYS® in order to obtain the aforementioned quantities.

Table 3.1: Modal quantities evaluation - input functions and outputs.

Function	Aim	Input	Output
<i>Func_Modal_Analysis</i>	Performing the modal analysis	- Number of vibration modes	- Modal frequencies - Modal displacements in the nodes belonging to the elements that define the load path (rails)
<i>Func_Extract_Modal_Displ</i>	Extraction of the modal displacements	- Node selection	- Modal displacements in the selected node
<i>Func_Extract_Modal_Forces</i>	Extraction of the modal forces	- Element selection	- Modal forces in the selected element

3.2.1.1 Modal Analysis

The first function that has to be executed in order to evaluate the modal quantities is the function "Func_Modal_Analysis", which purpose is to perform the modal analysis. Before its use, it is necessary to select in the routine the number of vibration modes, n , to be considered. The outputs of this function consist of the modal frequencies of the n selected modes and the modal displacements of the nodes, that belong to the elements that define the load path.

3.2.1.2 Modal Quantities Extraction

With the modal analysis performed, the next step consists of evaluating the specific modal quantity that is intended to analyse, using the function "Func_Extract_Modal_Displ", to obtain modal displacements, or the "Func_Modal_Forces", in order to evaluate the modal internal forces. In these functions, it is necessary to define the nodes or elements whose response is intended to be evaluated. Regarding the internal forces, the routine automatically extracts the axial force and bending moments of the respective element.

After the modal quantities extraction, the next step is the evaluation of the response of the structure, using the modal superposition method and it is presented in Section 3.2.2.

3.2.2 Evaluation of the Response of the Structure

The functions performed in MATLAB® that calculate the response of the structure, in terms of displacements, accelerations and forces, are presented in Table 3.2.

Table 3.2: Response of the structure evaluation - input functions and outputs.

Function	Aim	Input	Output
<i>Func_Solve_Syst_Eqs</i>	Resolution of the system of decoupled equations	- Time increment - Damping ratio - Traffic data - Modal frequencies - Modal displacement of the load path (rails).	- Modal coordinates
<i>Func_Modal_Superpos</i>	Application of the modal superposition method	- Modal displacements in the selected node - Modal forces in the selected element - Modal coordinates	- Nodal displacements - Nodal accelerations - Nodal forces

3.2.2.1 Resolution of the System of Decoupled Equations

The dynamic analysis performed in this work is based on moving load models, previously introduced in Chapter 2. This method does not allow the evaluation of the vehicle response since it is not explicitly modeled. However, and although it can lead to small imprecisions in the structure response (Albuquerque, 2008), its simple implementation makes it widely used in bridge dynamics.

When moving loads pass over a certain structure, they originate its dynamic response. If the loads are known, the dynamic response can be simulated numerically using a finite element model of the structure. The positions of the moving loads, at a certain time step t , may be computed based on their initial position and on their velocity. Once each load position is defined, it is converted into equivalent vertical nodal forces, applied to the nodes of the structure (Albuquerque, 2015). Figure 3.1 represents the degradation model of the loads, and the Equation 3.1 represents the shape function that determines the load component N at node j , for load k , at time instant t , with a spacing between nodes of a .

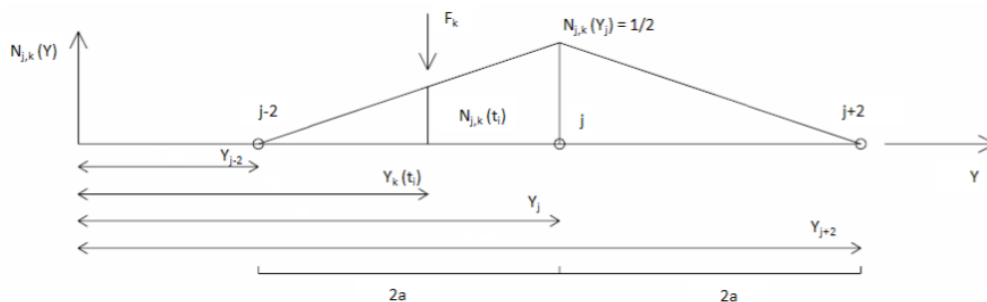


Figure 3.1: Shape function for the loads distribution (adapted from EN1991-2 (2003)).

$$N_{j,k}(t) = \begin{cases} \frac{1}{2} \times \frac{Y_k(t) - Y_{j-2}}{2a} & , Y_{j-2} \leq Y_k(t) \leq Y_j \\ \frac{1}{2} - \frac{1}{2} \times \frac{Y_k(t) - Y_j}{2a} & , Y_j \leq Y_k(t) \leq Y_{j+2} \\ 0 & , Y_k(t) \notin [Y_{j-2}; Y_{j+2}] \end{cases} \quad (3.1)$$

It is possible to calculate the total load at each node j at time instant t , and finally, to construct the load matrix for the n nodes, considering a number of analysis steps of m^v (Ferreira, 2017).

$$F_j(t_i) = \sum_k N_{j,k}(t_i) F_k \quad (3.2)$$

$$\text{Load Matrix} = \begin{bmatrix} 0 & F_1(0) & \dots & F_n(0) \\ \vdots & \vdots & \ddots & \vdots \\ t_i & F_1(t_i) & \dots & F_n(t_i) \\ \vdots & \vdots & \ddots & \vdots \\ t_{m^v} & F_1(t_{m^v}) & \dots & F_n(t_{m^v}) \end{bmatrix} \quad (3.3)$$

Thus, the load information at each node is obtained, for each time instant, for a certain train velocity. It is important to refer that the position of the moving loads and the corresponding nodal forces need to be updated at each time step. Two common numerical methods employed to perform the dynamic analysis are the direct integration and the modal superposition methods (Clough and Pezien, 1975).

Since the moving load model is characterized by a set of constant loads corresponding to the static loads of the train axles, these set of loads has to be separated from each other, according to the train geometry. The dynamic response of a structural system with N degrees of freedom, at a certain time-instant, may be evaluated considering the system of Equations 3.4:

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = f(t) , \quad (3.4)$$

where m , c and k are, respectively, the matrices of mass, damping and stiffness, each with N dimension. $f(t)$ is the nodal forces vector defined by $N \times 1$ terms. The terms $x(t)$, $\dot{x}(t)$ and $\ddot{x}(t)$ are correspondingly, the vectors of displacement, velocity and acceleration which represent the dynamic response of the structural system, also composed of $N \times 1$ terms.

When a direct time-integration algorithm is considered, the system of Equations 3.4 is solved for all time-steps being each instant of time $t + 1$ dependent of the structural response at t . This means that the structural analysis is indivisible and it has to be performed at once, considering one single numerical model and computing process (Horas et al., 2019). Since the structural system is globally elastic and has invariant properties along time, the modal superposition concepts may be considered and the system of Equations 3.4 is converted into N decoupled equations (Clough and Pezien, 1975):

$$\ddot{Y}_j(t) + 2w_j \xi_j \dot{Y}_j(t) + w_j^2 Y_j(t) = f_j(t) , \quad (3.5)$$

where, for a certain time instant, t , and for the j^{th} vibration mode, Y_j , $\dot{Y}_j$ and $\ddot{Y}_j$ are the modal coordinates for each of the N degrees of freedom related to, respectively, displacements, velocities and accelerations. Also, w_j is the j^{th} natural frequency and ξ_j the j^{th} damping coefficient, both modal properties invariant with time. Regarding the loading side, f_j is the nodal force vector being each term associated with each degree of freedom. In which concerns the modal parameters represented on the first-term of Equation 3.5, they are not load-dependent and only need to be computed once. When the superposition of modal quantities is adopted, such feature allows that in railway bridges analyses, for each time instant, t , only the definition of f_j is required, which in most of the times can be done using simple and linear shape functions.

The resolution of the system of decoupled equations was implemented using the Newmark direct integration method (see function "Func_Solve_Syst_Eqs" in Table 3.2). The following six inputs are needed to solve the decoupled equations presented in Equation 3.5: time increment, damping ratio, modal frequencies (see Table 3.1), modal displacements of the nodes that define the load path (see Table 3.1), load model of the train and train speed. By solving the system of decoupled equations, it is possible to obtain the modal coordinates, velocities and accelerations in the selected nodes.

With the modal coordinates determined by the procedure presented in the present section and with the modal quantities obtained by the process approached in Section 3.2.1, the final response of the structure in terms of nodal displacements, velocities and accelerations or in terms of internal forces or stresses can be calculated by modal superposition, as explained in the next sub-section.

3.2.2.2 Modal Superposition

The efficiency of the modal superposition technique arises not only from the decoupling of the system of equations, but also from the fact that the total number of modes of vibration needed to accurately reproduce the dynamic response of the flexible structure, J , is usually much lower than the total number of degrees of freedom ($J \ll N$). Then, the modal superposition principles may be used to efficiently compute any type of quantity ψ , such as displacements, velocities, accelerations, forces and stresses through the following equation (Horas et al., 2018):

$$\psi(t) = \sum_j \psi_j Y_j(t) , \quad (3.6)$$

where ψ_j is the respective modal quantity, taking into account the modal shape of the j^{th} vibration mode.

Finally, the MATLAB® function "Func_Modal_Superpos" aims to compute the final response of the structure $\psi(t)$ using the modal superposition principle expressed in Equation 3.6. The inputs of this function are the modal quantities ψ_j computed through the modal analysis (see Table 3.1) and the modal coordinates Y_j calculated through the resolution of the decoupled system of equations (see Table 3.2).

In this way, the response of the structure in terms of displacements, accelerations and forces is reached.

3.2.3 Procedure Synthesis

Figure 3.2 shows a flow chart that summarizes the previously described steps to analyse the structural response through the modal superposition method.

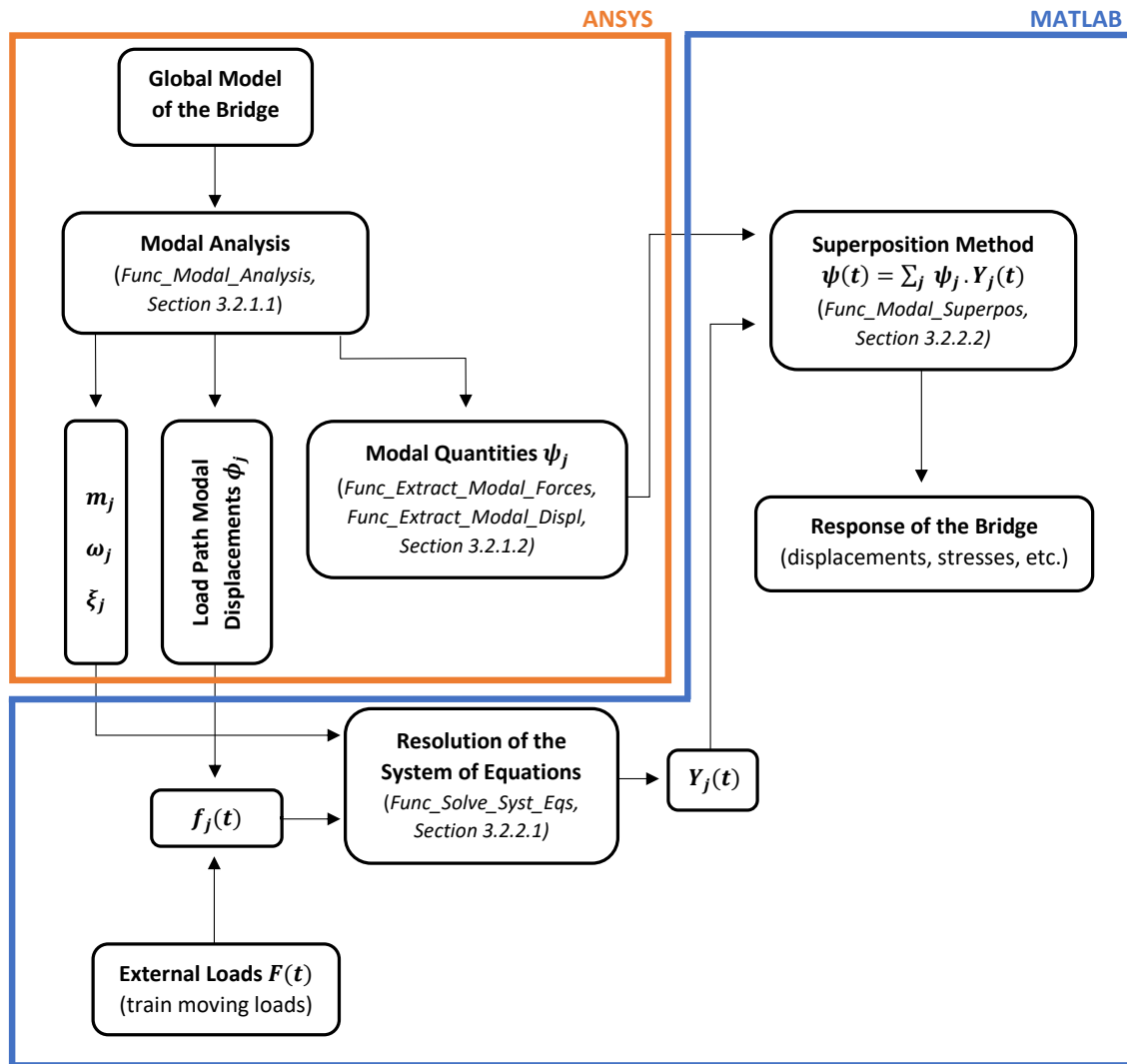


Figure 3.2: Procedure to evaluate the structural response through the modal superposition method.

3.3 Fatigue Analysis Methodology

The fatigue analysis methodology used in this work is presented in the present section. Initially, the S-N curves used to represent the fatigue resistance under nominal stress intervals are presented (see Section 3.3.1). The partial safety factor is approached in Section 3.3.2 and lastly, in Section 3.3.3, it is presented the fatigue damage evaluation based on the accumulated damage method, defined by the Palmgren-Miner rule presented in EN1993-1-9 (2004).

3.3.1 S-N Curves for Direct Stress Ranges

The S-N curves for direct stress ranges, $\Delta\sigma_i$, may present two different configurations. EN1993-1-9 (2004) provides fourteen S-N curves for direct stresses, corresponding to fourteen different fatigue strength levels (see Figure 3.3).

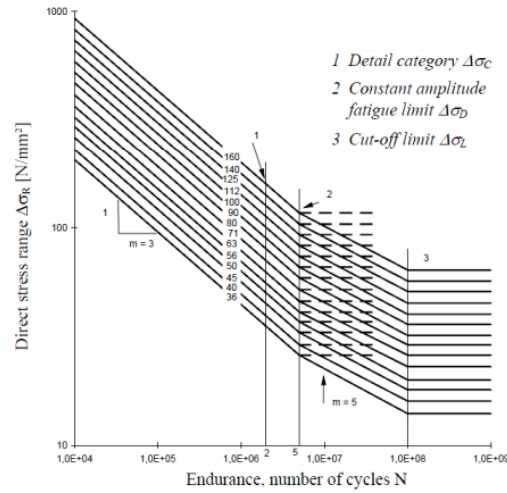


Figure 3.3: S-N curves for direct stress ranges (adapted from EN1993-1-9 (2004)).

If the actuating direct stress is composed by constant amplitude cycles, the curve is characterized by a single slope, $m = 3$, until 5×10^6 cycles. After this point, according to the definition of constant amplitude fatigue limit, $\Delta\sigma_D$, it is considered that no damage will occur for cycles which do not exceed the constant amplitude fatigue limit. That curve configuration is also applicable when variable amplitude intervals are present but do not exceed $\Delta\sigma_D$. For domains in which direct stress is composed by constant amplitude cycles, fatigue strength can be obtained by the following expression:

$$\Delta\sigma_R^m N_R = \Delta\sigma_C^m 2 \times 10^6 \quad \text{with } m = 3 \text{ for } N \leq 5 \times 10^6, \quad (3.7)$$

where $\Delta\sigma_R$ represents the fatigue strength of the structural member, which is reached after N acting cycles. The constant amplitude limit, $\Delta\sigma_D$, is expressed by:

$$\Delta\sigma_D = \left(\frac{2}{5}\right)^{1/3} \Delta\sigma_C = 0.737 \Delta\sigma_C. \quad (3.8)$$

If the direct stresses are composed by variable amplitude cycles and, at least one of those cycles has a stress range higher than the constant amplitude fatigue limit, $\Delta\sigma_D$, then the S-N curve has two linear trends, with slopes of 3 and 5. For these domains, the fatigue strength is given by the following expressions:

$$\Delta\sigma_R^m N_R = \Delta\sigma_C^m 2 \times 10^6 \quad \text{with } m = 3 \text{ for } N \leq 5 \times 10^6, \quad (3.9)$$

$$\Delta\sigma_R^m N_R = \Delta\sigma_C^m 2 \times 10^6 \quad \text{with } m = 5 \text{ for } 5 \times 10^6 \leq N \leq 10^8. \quad (3.10)$$

The cut-off limit $\Delta\sigma_L$ can be calculated by the expression:

$$\Delta\sigma_L = \left(\frac{5}{100}\right)^{1/5} \Delta\sigma_D = 0.549\Delta\sigma_D. \quad (3.11)$$

When some structural details do not fit in any of the fourteen S-N curves presented above, the details are classified one category below the one they could be associated to, considering a conservative analysis. Those details are identified with the $\Delta\sigma_C^*$ designation. Nevertheless, an alternative assessment can be performed, assuming a higher detail category but imposing, at the same time, that the value of $\Delta\sigma_D$ is the fatigue strength for 10^7 cycles, as illustrated in Figure 3.4.

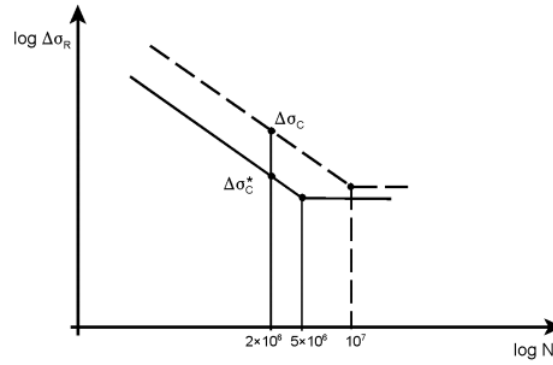


Figure 3.4: Alternative strength $\Delta\sigma_C$ for details with special classification $\Delta\sigma_C^*$ (EN1993-1-9 (2004)).

The fatigue strength of some types of structural details is especially dependent on dimensional parameters such as the thickness of their components. Hence, the Eurocode EN1993-1-9 (2004) foresees a reduction on fatigue strength, by means of a coefficient, k_s :

$$\Delta\sigma_{C,red} = k_s \Delta\sigma_C. \quad (3.12)$$

3.3.2 Partial Safety Factor

The value of the partial safety factor for fatigue strength, γ_{Mf} , depends on two different factors (Albuquerque, 2015):

- Accessibility of the detail during inspection and maintenance procedures;
- Consequences of the collapse of the structural detail for the global stability of the structure.

According to EN1993-1-9 (2004), fatigue assessment shall be performed considering two different approaches: the safe life method and the damage tolerant method. The first aims to ensure a safe performance of the structure for a certain period of time without fatigue cracks. The damage tolerant method is based on the principle that, if an element of the structure fails, due to the lack of fatigue-resistant capacity, the remaining elements must be capable of maintaining the structural integrity and operability until further strengthening operations. Depending on the concept of fatigue design adopted, the safety partial factor related to fatigue damage is presented in Table 3.3.

Table 3.3: Partial safety factor for fatigue strength γ_{Mf} (adapted from EN1993-1-9 (2004)).

Assessment method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1.00	1.15
Safe life	1.15	1.35

3.3.3 Linear Damage Accumulation Method - Palmgren-Miner Rule

The methodology used in this work to analyse the fatigue strength of a structural member is the Linear Damage Accumulation Method. As mentioned in Chapter 2, the accumulation of the fatigue damage is evaluated using the Palmgren-Miner rule applied to the several stress amplitude ranges, $\Delta\sigma_i$, computed from the stress history, as shown in Figure 3.5.

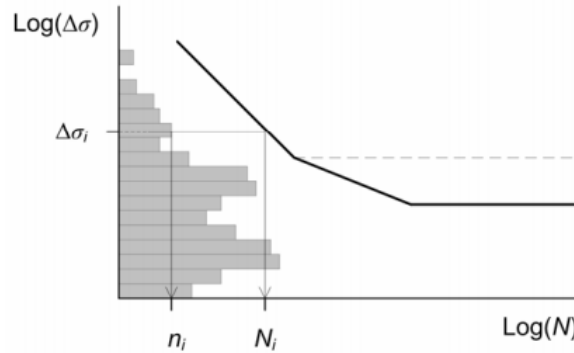


Figure 3.5: Damage accumulation calculation (Kühn et al., 2008).

According to the Annex A of the EN1993-1-9 (2004), the evaluation of the accumulated linear fatigue damage is carried out by performing the following steps:

1. Definition of the loading events, after characterization of the traffic scenarios - Figure 3.6a;
2. Determination of the stress histories at the details - Figure 3.6b;
3. Stress cycles counting using an appropriate methodology, such as the rainflow method - Figure 3.6c;
4. Calculation of the number of cycles, n_i , associated with each stress interval range, $\Delta\sigma_i$ - Figure 3.6d;
5. Identification of the S-N curve that better represents the fatigue strength of the detail and computation of the fatigue strength in cycles, N_i , associated to each stress interval range, $\Delta\sigma_i$ - Figure 3.6e;
6. Damage computation using the Palmgren-Miner rule - Figure 3.6f.

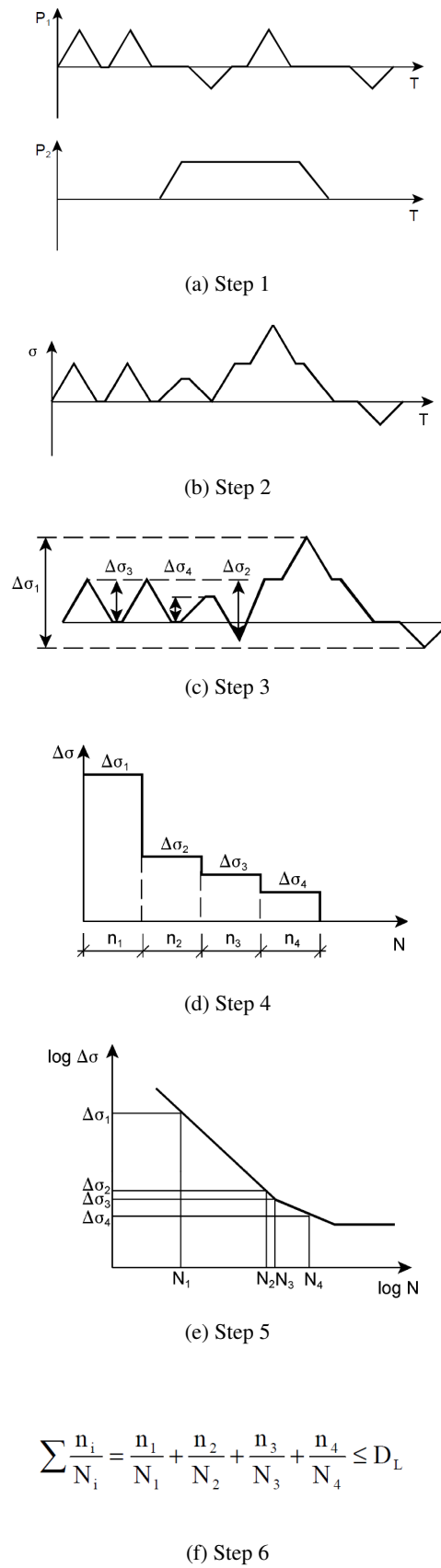


Figure 3.6: Workflow of the linear damage accumulation method (EN1993-1-9, 2004).

Chapter 4

Dynamic Analysis of the Várzeas Railway Bridge

4.1 Introduction

In this work, as already mentioned, the dynamic analysis of the Várzes Railway bridge is performed using a numerical methodology, based on the moving loads model. Initially, before the dynamic analysis presentation, the studied bridge is introduced in Section 4.2. Still, in this section, the bridge model developed in previous investigations on the finite element software ANSYS® (2018) is presented. Its technical drawings were developed in AUTOCAD® (2018).

Section 4.3 is dedicated to the dynamic analysis presentation. Some considerations are made before starting the analysis itself, defining aspects such as the variables to be extracted from the analysis, the train speeds, the number of vibration modes to be considered, the damping to be assigned to the model and the time increment required to obtain accurate results. Once these parameters become defined, the response of the bridge in terms of displacements and accelerations are presented for two different nodes (control points).

The chapter ends with a comparison between the results obtained by those analyses. It is also discussed the number of vibration modes and the value of the time increment that should be adopted, in order to clarify how the results obtained could be as accurate as possible.

4.2 The Várzeas Railway Bridge

4.2.1 Bridge Description

The bridge was constructed as part of a project to replace existing bridges in the “Beira Alta” route, carried out during the decade of 50. This metallic bridge was built in 1958 by the German House Fried Krupp and it is situated in the Beira Alta railway line, across the Várzeas river, near the town of Luso in Portugal (see Figures 4.1 and 4.2).



Figure 4.1: West view of Várzeas railway bridge (Infraestruturas, 2018).



Figure 4.2: Photo of the interior of the 3D truss of the deck (Infraestruturas, 2018).

This main bridge is composed by five continuous metallic spans of $50.5 + 60 \times 3 + 50.5$ meters, as detected in Figure 4.3, and a sixth secondary simple supported span with 20 meters length in the West side. The bridge is supported by four metallic truss columns, one masonry abutment in the East extremity and one masonry column in the West side. The simply supported span is supported in the latter column in the West masonry abutment.

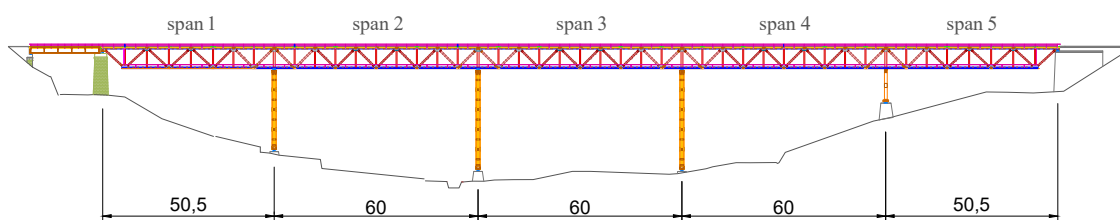


Figure 4.3: Plan view of the Várzeas railway bridge (in meters).

The deck structural solution consists of a three-dimensional truss composed by two main vertical Warren type trusses (VT) connected, whose lower chords are connected to each other through a horizontal bracing plane composed by transversal bars and crosses connecting them. In the upper side, the connection is made with "I" shape cross-girders with, approximately, 700 millimeters height, in which the two longitudinal girders that support the ballastless track system lay on.

Figure 4.4 shows schematic drawings of the typical cross-section and the vertical main truss.

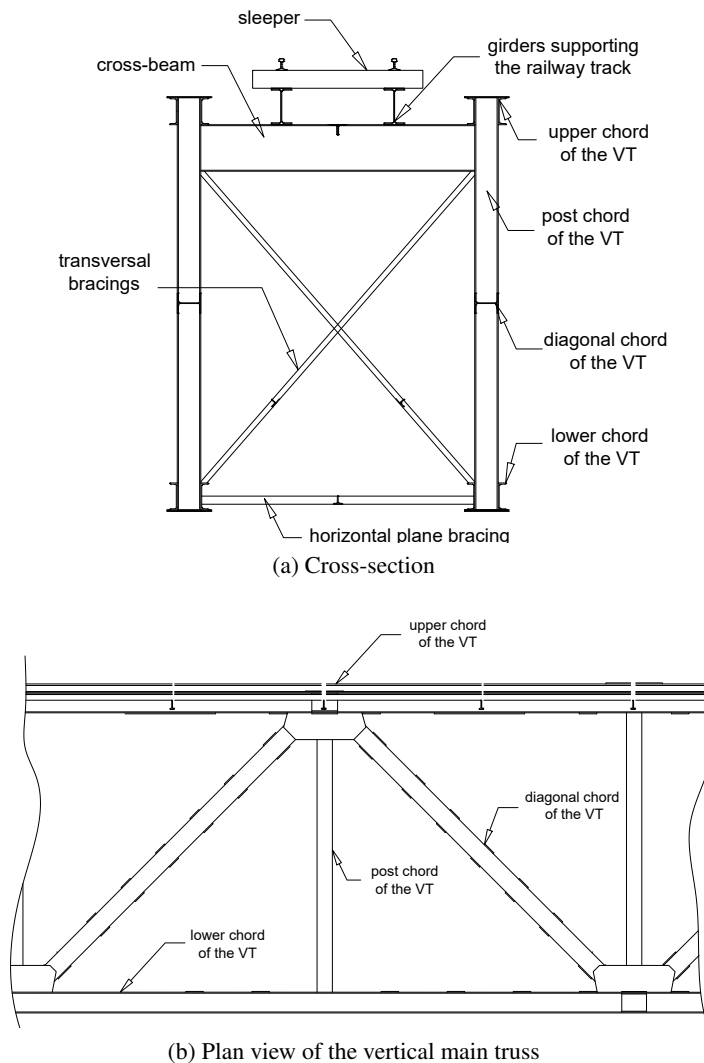


Figure 4.4: Schematic representation.

4.2.2 Bridge Modelling

The model of the Varzeas bridges, represented in Figure 4.5, has been developed by Horas (2017) in the scope of the author's PhD thesis with the objective of developing advanced methodologies for assessing the fatigue damage in structural details. The model was developed using beam elements to model the bars (cross-beams, longitudinal girders, diagonals, etc) and mass

point elements to model non-structural components. Moreover, the track, including pads, fasteners, sleepers and rails, are also modelled using spring-dampers for the first two components and beam elements for the latter.

The track properties, namely pads/fasteners stiffness and damping, have been adopted from the literature.

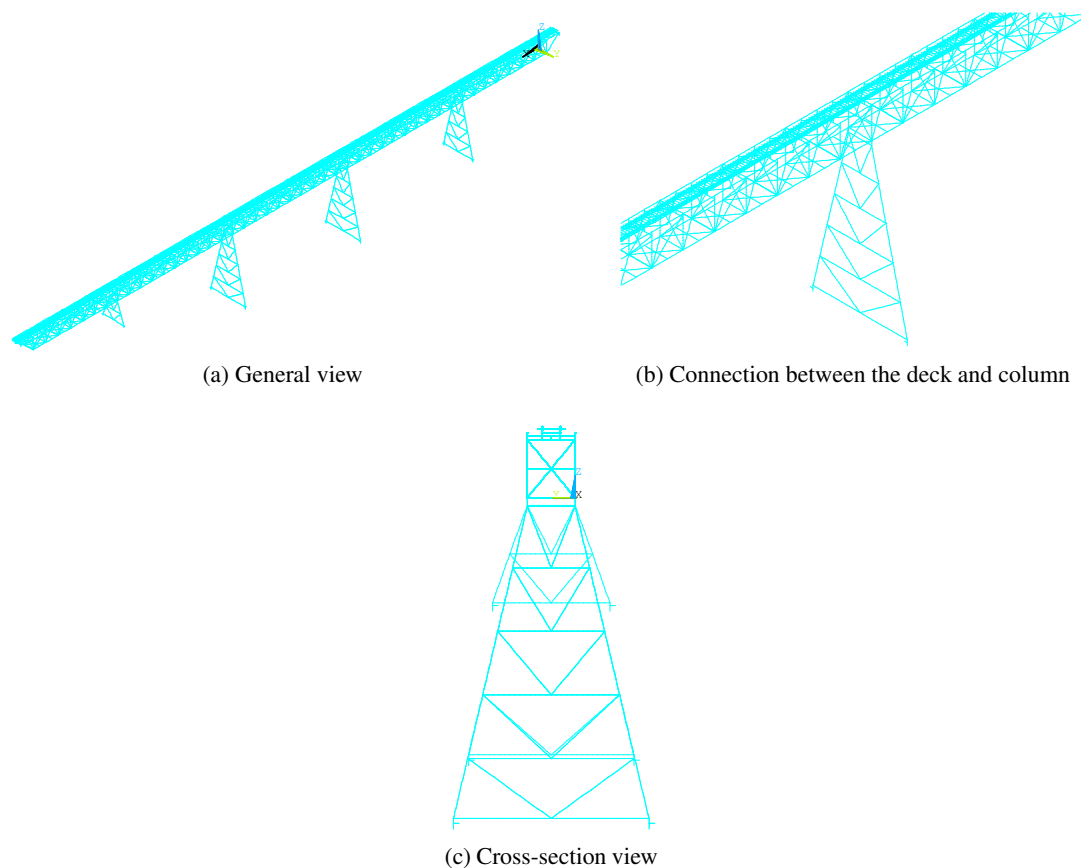


Figure 4.5: Numerical model of the Várzeas railway bridge.

4.3 Bases for the Dynamic Analysis

4.3.1 Variables Extracted

The relevant parameters extracted from this dynamic analysis are presented below:

- Displacements analysis in the centre of the cross-beam at mid-span of span 3;
- Displacements analysis in the lower chord of the vertical main truss at mid-span of span 3;
- Accelerations analysis in the previously mentioned control points.

In Figure 4.6, it is presented the location of the control points in the cross-section of the bridge.

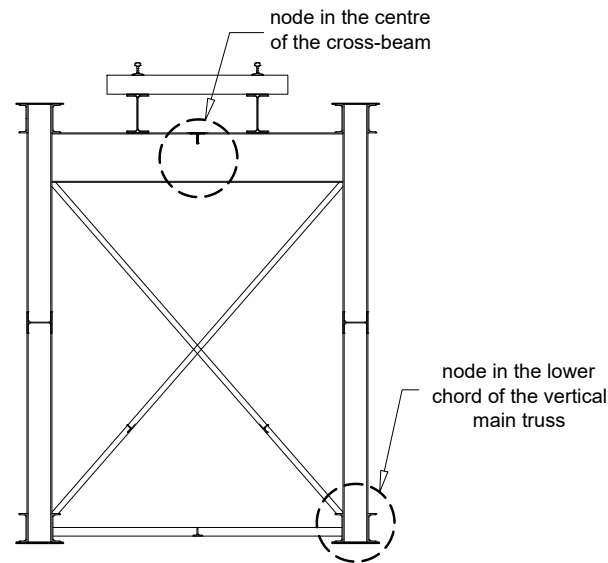


Figure 4.6: Location of the control points studied .

4.3.2 Range of Speeds

EN1991-2 (2003) establishes that the dynamic analysis must be performed for train speeds between 40 m/s (144 km/h) and 20% higher than the maximum track speed. The bridge studied in this work is not a high-speed railway bridge and, consequently, it was defined that the analysis would be performed for a medium range of speeds - between 5 km/h and 140 km/h with a speed interval of 5 km/h.

4.3.3 Number of Vibration Modes

A sensitivity analysis was performed in order to evaluate the influence of the number of modes adopted on the dynamic response of the bridge. Regarding the analysis of displacements and accelerations in the control points, it was considered vibration modes of the model with vibration frequencies up to the values of:

- 30 Hz, equivalent to 320 vibration modes;
- 60 Hz equivalent to 1150 vibration modes;
- 100 Hz equivalent to 2150 vibration modes;
- 200 Hz equivalent to 4000 vibration modes.

4.3.4 Damping Coefficient

The peak response of a structure at traffic speeds, corresponding to resonant loading, is highly dependent on damping. According to EN1991-2 (2003), the values of damping that should be used in the dynamic analysis are presented in Table 4.1.

Table 4.1: Values of damping to be assumed for design purposes (adapted from EN1991-2 (2003)).

Bridge Type	ζ Lower limit of percentage of critical damping (%)	
	Span $L < 20$ m	Span $L \geq 20$ m
Steel and composite	$\zeta = 0,5 + 0,125 (20 - L)$	$\zeta = 0.5$
Prestressed concrete	$\zeta = 1,0 + 0,07 (20 - L)$	$\zeta = 1.0$
Filler beam and reinforced concrete	$\zeta = 1,5 + 0,07 (20 - L)$	$\zeta = 1.5$

Since the studied bridge is a steel bridge, and according to Table 4.1, the value of damping adopted in the calculation is 0.5%.

4.3.5 Time Increment

According to ERRID21/RP9 (2001), there are three ways to define this parameter, being its values presented in Table 4.2, as a function of the maximum vibration frequency adopted, f_{max} , and, consequently, the number of vibration modes, n . In the present study, the span length of the Várzeas bridge, L , is 60 m and the maximum train speed, v_{max} , considered was 38.9 m/s (140 km/h).

Table 4.2: Time increments recommended (adapted from ERRID21/RP9 (2001)).

Maximum Frequencies (Hz)	30	60	100	200
Number of Vibration Modes (n)	320	1150	2150	4000
$\Delta t = \frac{1}{8f_{max}}$ (ms)	4.2	2.1	1.3	0.6
$\Delta t = \frac{L}{4nv_{max}}$ (ms)	1.2	0.3	0.2	0.1
$\Delta t = \frac{1}{20f_{max}}$ (ms)	4.2	0.8	0.5	0.3

The values presented in Table 4.2 served as a starting point to adopt a reasonable time increment. In Sections 4.4 and 4.5, the definition of the number of vibration modes was performed assuming a time increment of 2 ms.

4.4 Displacements Analysis

Despite not being a high-speed track, the displacements analysis of the control points was performed considering a traffic composed only by the Alfa Pendular train. The graphs generated present the maximum nodal displacement calculated for each train velocity, considering different approaches to the number of vibration modes of the model and to the time increments used in this analysis.

4.4.1 Analysis of the Control Point at the Cross-beam

Observing the graph represented in Figures 4.7 and 4.8, it can be noticed that the maximum nodal displacement calculated is 10.18 mm for a train speed of 120 km/h (60 Hz). It is also possible to conclude that the analyses, considering modes with frequencies up to 60, 100 and 200 Hz, provide similar values of maximum nodal displacements. Those values represent a difference around 1.3% when compared to the values obtained by the analysis assuming modes with frequencies up to 30 Hz. Further, the analysis assuming modes with frequencies up to 100 Hz represents a fair commitment between the results precision and the computational effort needed for the calculations performed.

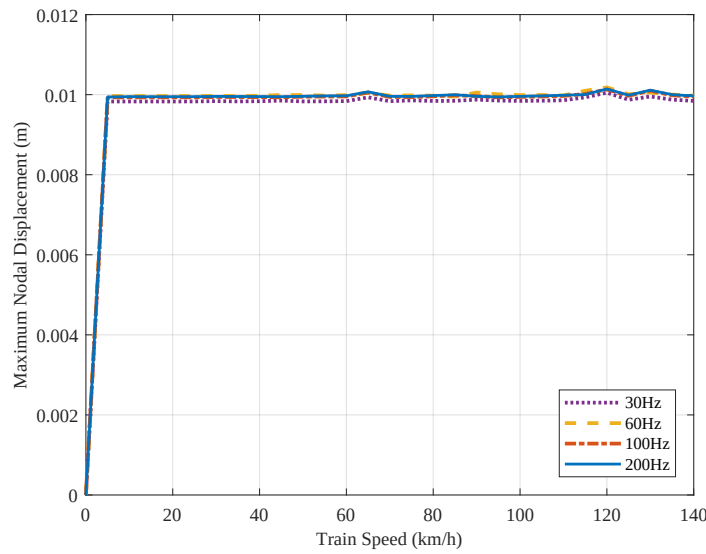


Figure 4.7: Maximum displacements of the node at the cross-beam ($\Delta t = 2ms$) - Zoom out.

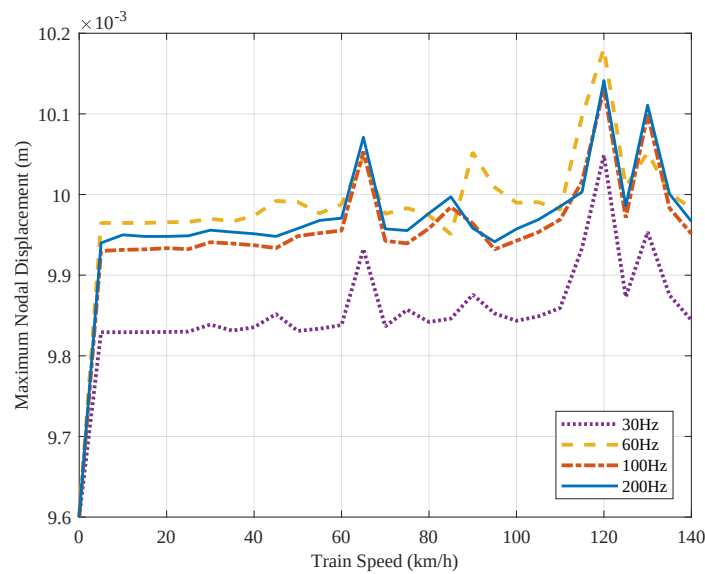


Figure 4.8: Maximum displacements of the node at the cross-beam ($\Delta t = 2ms$) - Zoom in.

Figure 4.9 highlights the similarities between the nodal displacements obtained, considering modes with different frequencies for two specific train speeds.

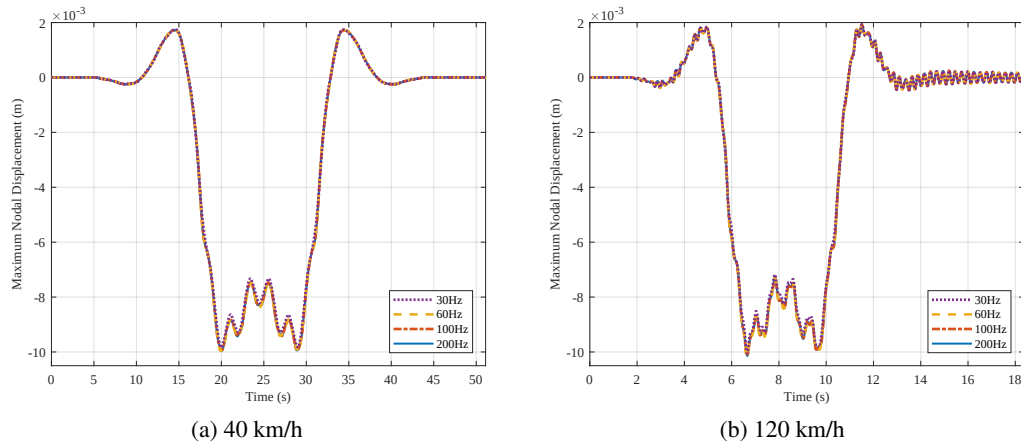


Figure 4.9: Displacements time-history of the node at the cross-beam ($\Delta t = 2ms$).

On the other hand, Figures 4.10 and 4.11 is present the graph obtained by an analysis that aims to clarify which time increment provides the best performance efficiency, both in terms of precision and calculation time of the results. In these calculations, it was already adopted an analysis assuming modes with frequencies up to 100 Hz. It is easy to verify that the trendlines, obtained by the analyses using different time increments, are almost overlapped which expresses a high similarity between the values calculated by each analysis. This fact is also evidenced by the graphs presented in Figure 4.12.

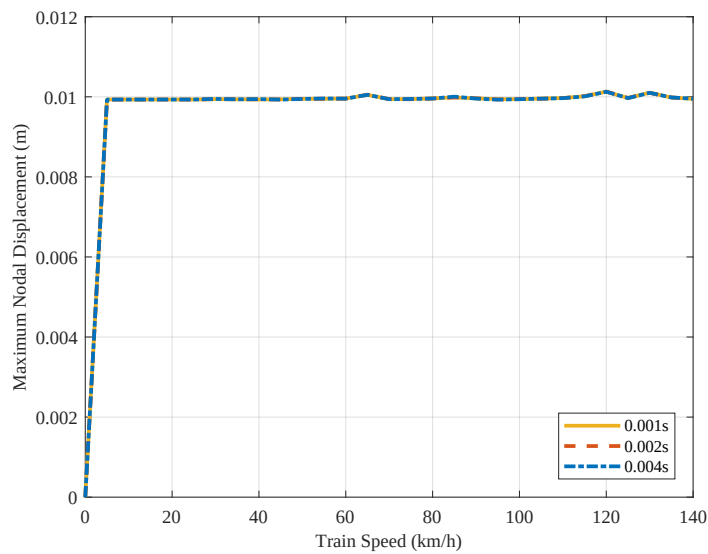


Figure 4.10: Maximum displacements of the node at the cross-beam ($f = 100Hz$) - Zoom out.

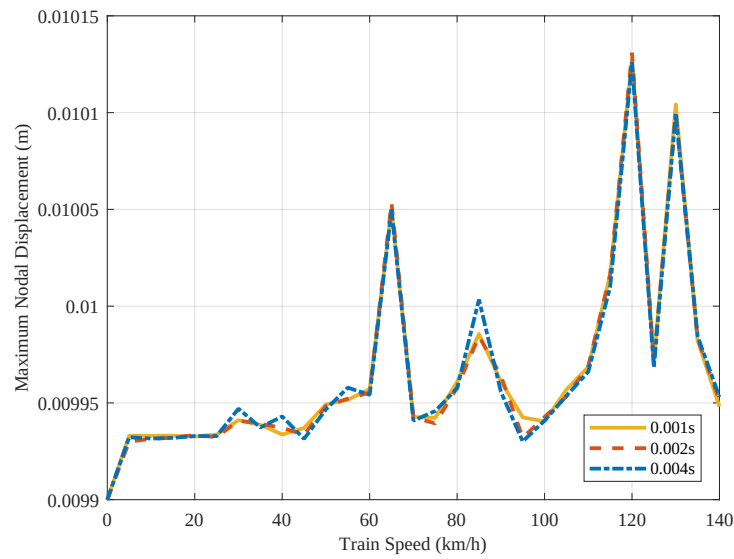


Figure 4.11: Maximum displacements of the node at the cross-beam ($f = 100\text{Hz}$) - Zoom in.

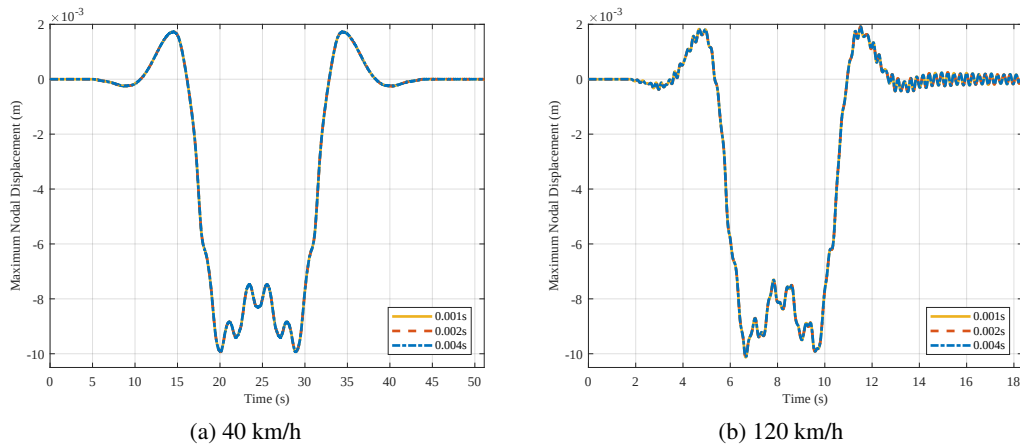


Figure 4.12: Displacements time-history of the node at the cross-beam ($f = 100\text{Hz}$).

4.4.2 Analysis of the Control Point at the Vertical Main Truss

Interpreting the graph represented in Figures 4.13 and 4.14, it is possible to perceive that the maximum nodal displacements occur for train speeds of 120 and 125 km/h, as it was verified for the analysis of the node located at the cross-beam (see Figures 4.7 and 4.8). For the node located at the main truss, the maximum nodal displacement obtained is 9.50 mm and it happens when the speed of the train is 120 km/h (30 Hz). Furthermore, the trendlines obtained by the analyses considering modes with frequencies up to 60, 100 and 200 Hz, provide similar values of maximum nodal displacements and represent a difference of 1.4% when compared to the trendline obtained by the analysis assuming modes with frequencies up to 30 Hz.

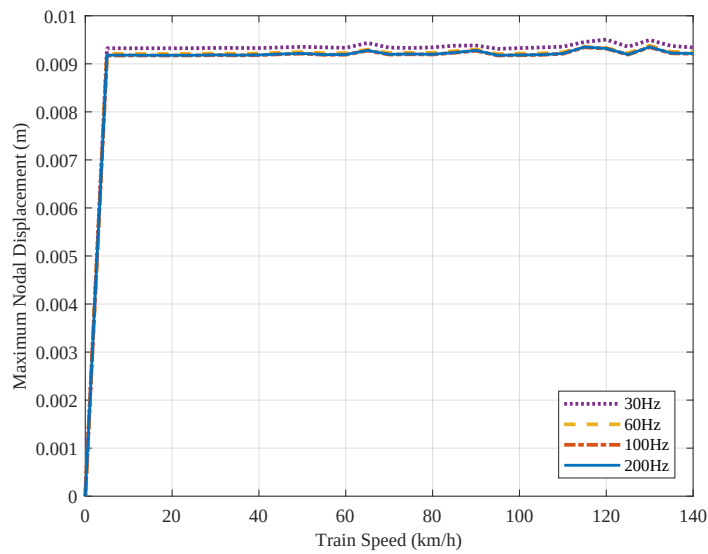


Figure 4.13: Maximum displacements of the node at the vertical main truss ($\Delta t = 2ms$) - Zoom out.

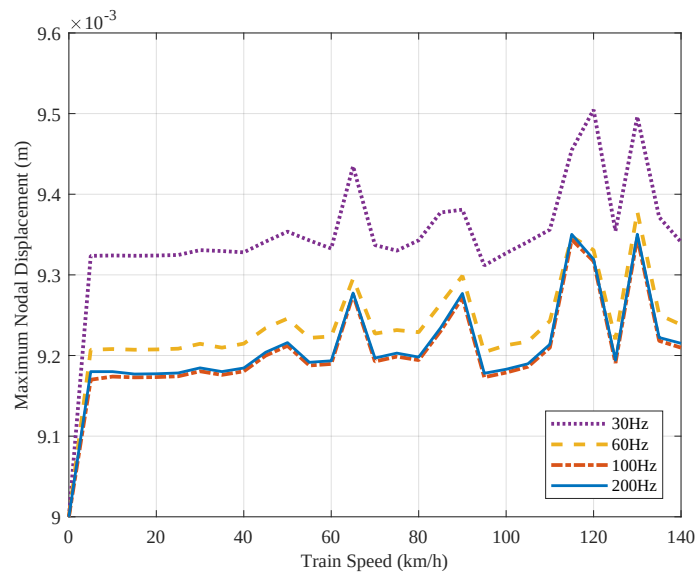


Figure 4.14: Maximum displacements of the node at the vertical main truss ($\Delta t = 2ms$) - Zoom in.

Analysing the time-history of the nodal displacements for two different train speeds, it is verified that the results considering different frequencies are almost equals (see Figure 4.15).

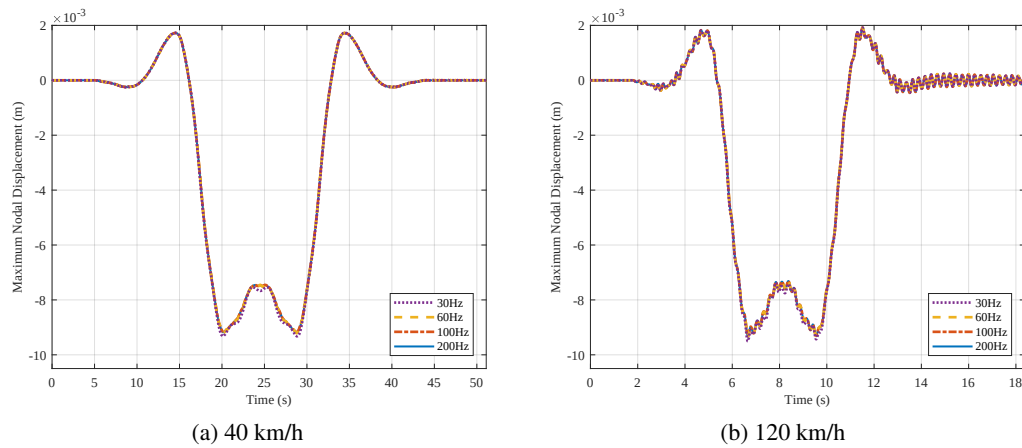


Figure 4.15: Displacements time-history of the node at the vertical main truss ($\Delta t = 2ms$).

To evaluate the time increment, the procedure was the same used for the node located at the cross-beam and it was considered an analysis assuming modes with frequencies up to 100 Hz. Paying attention to the graph represented in Figures 4.16 and 4.17, it is visible that the trend-lines are almost overlapped again, which means that the results were similar using different time increments.

Comparing the results for different train speeds, the similarity between the analyses assuming different time increments is evidenced by the graphs represented in Figure 4.18.

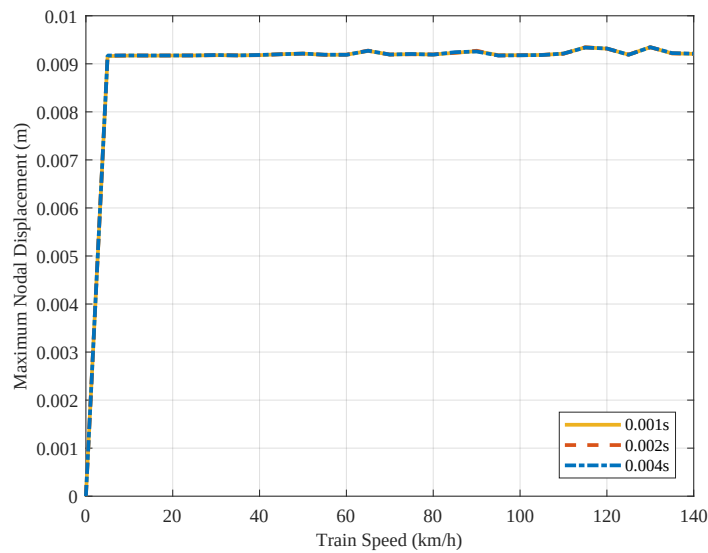


Figure 4.16: Maximum displacements of the node at the vertical main truss ($f = 100Hz$) - Zoom out.

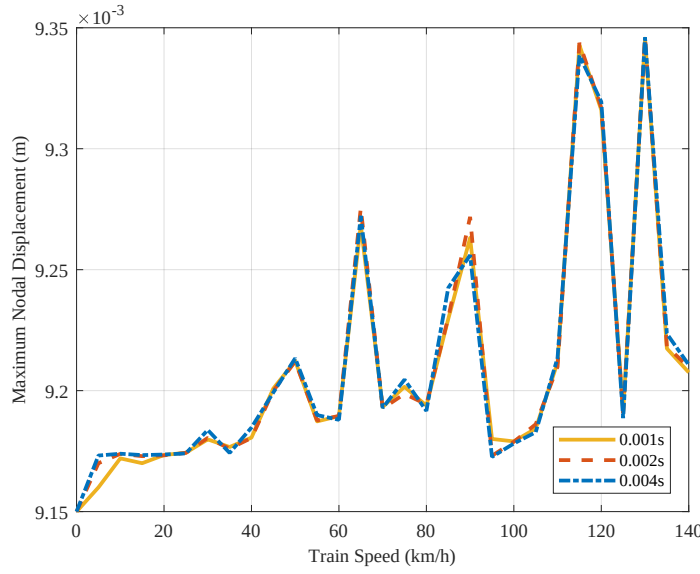


Figure 4.17: Maximum displacements of the node at the vertical main truss ($f = 100\text{Hz}$) - Zoom in.

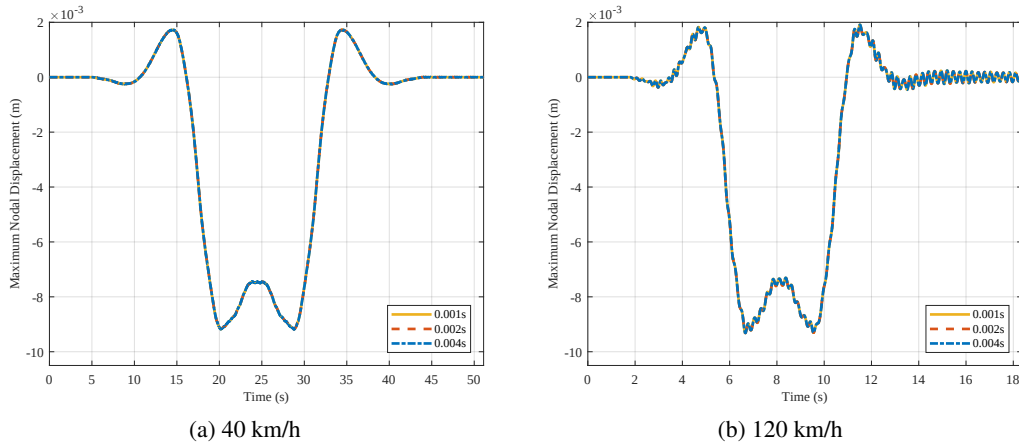


Figure 4.18: Displacements time-history of the node at the vertical main truss ($f = 100\text{Hz}$).

4.5 Accelerations Analysis

This analysis was performed considering the same traffic assumptions taken in the analysis of the displacements. The graphs that were generated present the maximum nodal acceleration calculated for each train velocity, considering different approaches to the number of vibration modes of the model and to the time increments used in the analysis.

4.5.1 Analysis of the Control Point at the Cross-beam

The graph presented in Figure 4.19 shows that increasing the number of vibration modes of the model, the accelerations trendlines raise significantly but do not tend to converge as expected. Due to the characteristics of the bridge, the number of vibration modes adopted highly influences

this analysis results, once that the contribution of the local vibration modes to the accelerations field is substantial. Moreover, this node is located at the cross-beam, which is a segment where this phenomenon hugely affects the dynamic analysis.

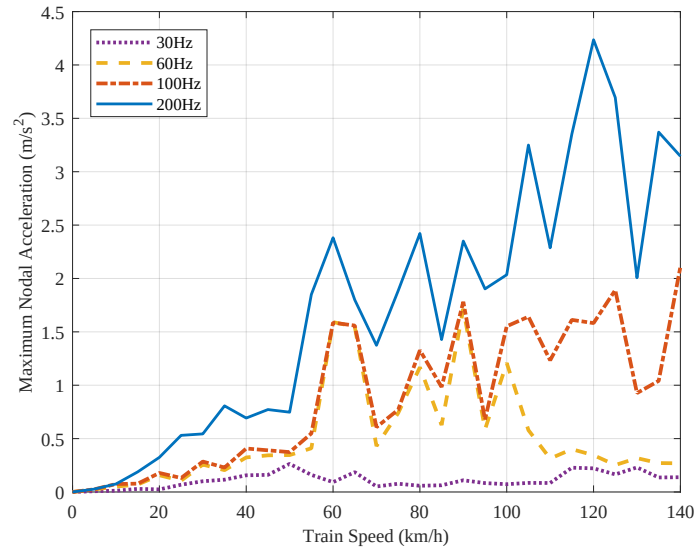


Figure 4.19: Maximum accelerations of the node at the cross-beam ($\Delta t = 2ms$).

In Figure 4.20, it is presented a time-history analysis of the nodal accelerations for two different train speeds. It is demonstrated again that for higher train speeds, the nodal accelerations tend to increase and the results obtained by the analyses considering modes with different frequencies tend to diverge more.

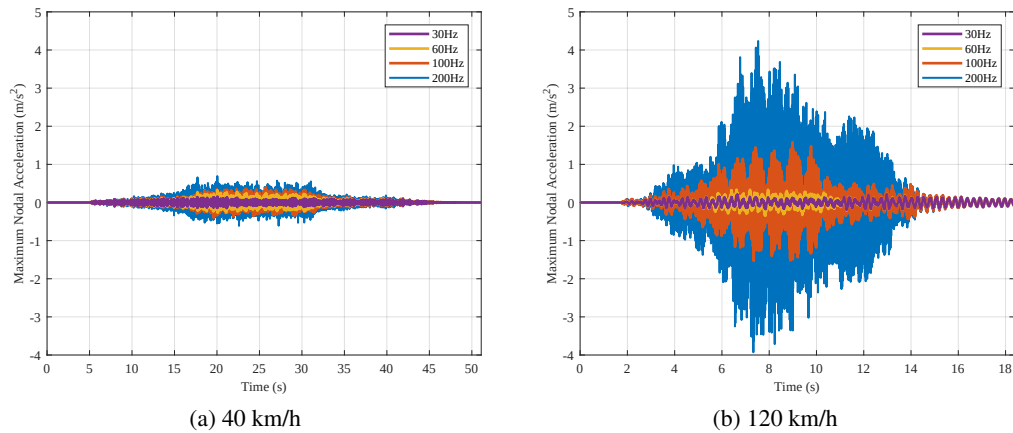


Figure 4.20: Accelerations time-history of the node at the cross-beam ($\Delta t = 2ms$).

As a consequence of the phenomenon previously referred and by analysing the graph presented in Figure 4.21, it is concluded that even with an analysis considering a time increment of 1 ms was not possible to obtain a convergence of results. Nevertheless, it is important to underline that this issue only occurs for the analysis of the accelerations.

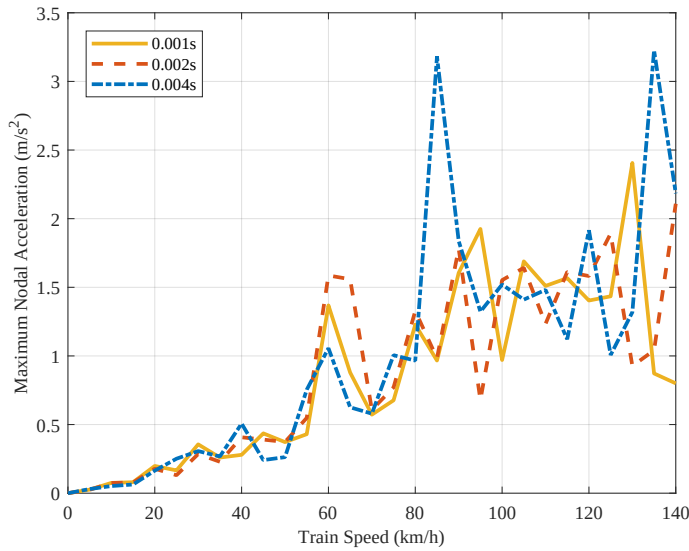


Figure 4.21: Maximum accelerations of the node at the cross-beam ($f = 100Hz$).

Figure 4.22 presents the time-history analysis of the nodal acceleration for two different train speeds.

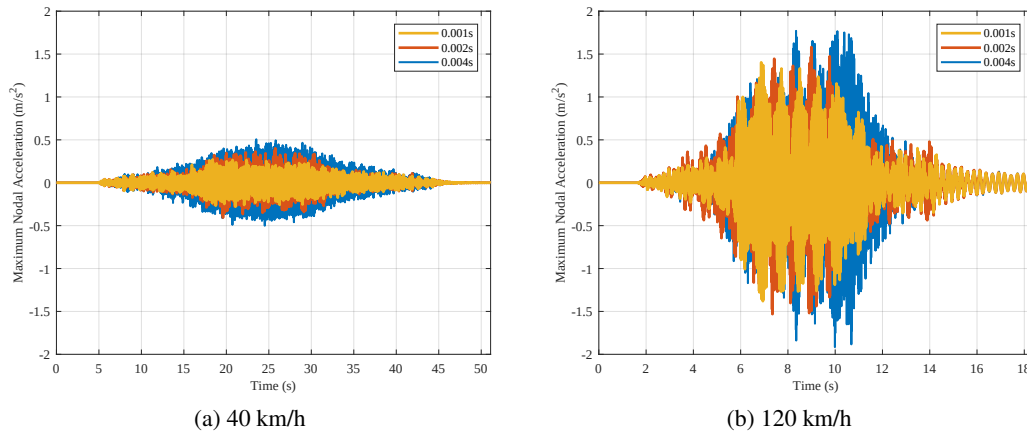


Figure 4.22: Accelerations time-history of the node at the cross-beam ($f = 100Hz$).

4.5.2 Analysis of the Control Point at the Vertical Main Truss

Contrarily to the graph presented in Figure 4.19 relative to the node located at the cross-beam, the results obtained for the node located at the main truss reveal that when considering modes with frequencies up to 60, 100 and 200 Hz, the trendlines converge until a train velocity of 100 km/h (see Figure 4.23). From 100 to 140 km/h, the analyses assuming modes with frequencies up 100 and 200 Hz provide similar results. Despite the fact that this node is located in a less unfavourable segment when compared to the node located at the cross-beam, there is also a contribution of the local vibration modes to the dynamic response, which explains the non-uniformed graph.

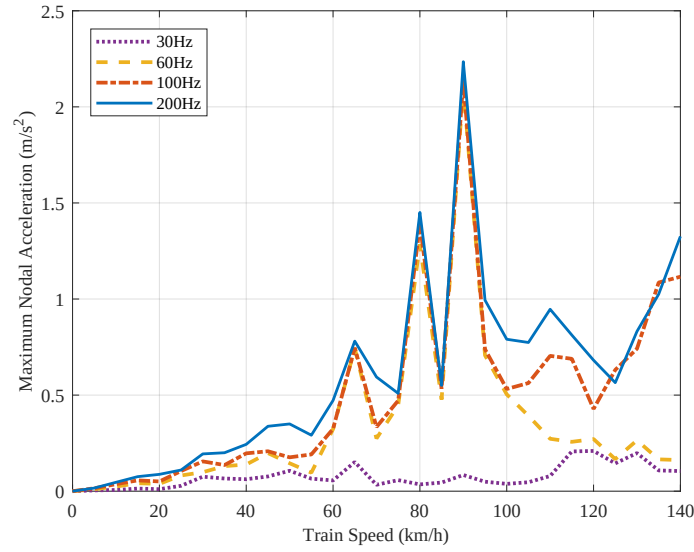


Figure 4.23: Maximum accelerations of the node at the vertical main truss ($\Delta t = 2ms$).

Bearing in mind the analyses presented in Figure 4.23, two time-history of nodal accelerations analysis relative to two different train speeds were performed are presented in Figure 4.24. It is visible that for a train speed of 110 km/h the different modal frequencies analyses process results with a higher discrepancy. Hence, as already seen for the node located at the cross-beam, this happens as a result of the high influence that the number of vibration modes adopted has in accelerations analysis. There is a significant contribution of the local vibration modes and therefore, the choice of the number of vibration modes must be cautious.

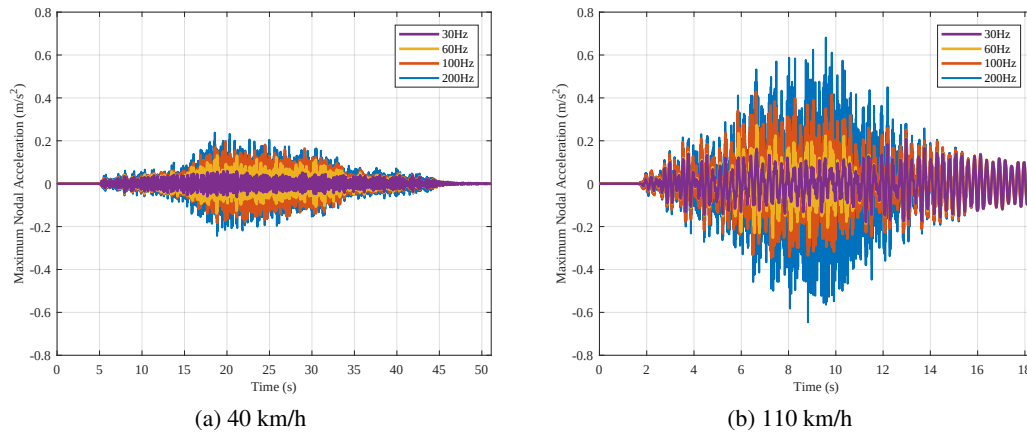


Figure 4.24: Accelerations time-history of the node at the vertical main truss ($\Delta t = 2ms$).

Understanding now the graph presented in Figure 4.25, it can be concluded that, as already seen for the analyses which involve the node located at the cross-beam, the time increments adopted do not provide a convergence of results.

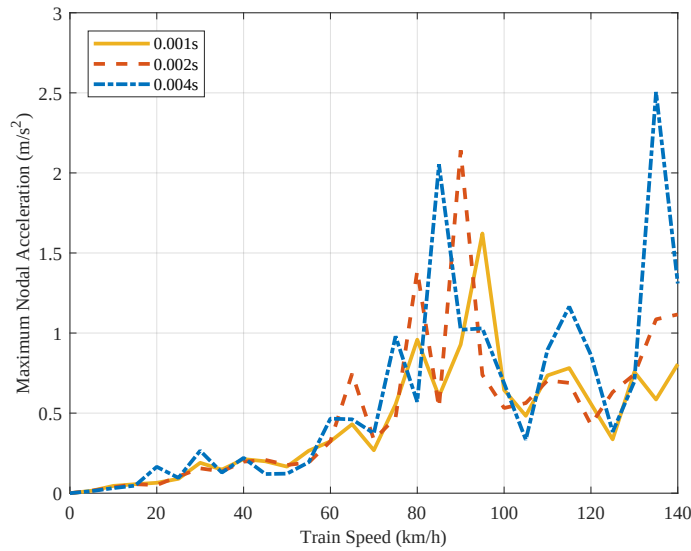


Figure 4.25: Maximum accelerations of the node at the vertical main truss ($f = 100\text{Hz}$).

In Figure 4.26 is presented the time-history analysis of the nodal accelerations for two different train speeds.

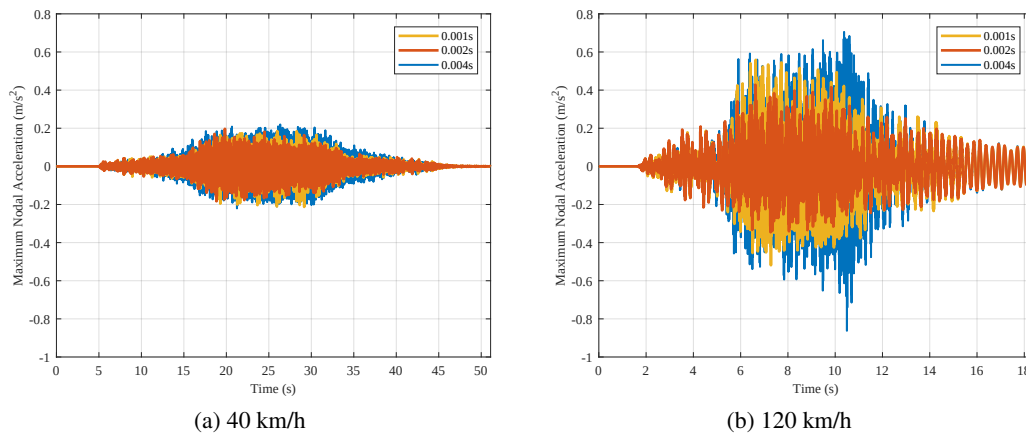


Figure 4.26: Accelerations time-history node at the vertical main truss ($f = 100\text{Hz}$).

4.6 Concluding Remarks

Evaluating the displacements obtained for critical points, it can be concluded that an analysis considering a frequency of 100 Hz and adopting a time increment of 2 ms provides accurate results and minimizes the time spent in calculations.

Differently, for the accelerations analysis, it was not possible to reach that commitment between precision and computational effort. This occurs since there is a high contribution of the local vibration modes to the dynamic response and therefore, it is concluded that the number of vibration modes adopted has a huge influence on the results. It is also important to underline that

this phenomenon has more incidence in the cross-beam than in the main truss, as confirmed by the results obtained in by the analyses of the different nodes.

Considering the expression that allows the obtainment of the acceleration:

$$\ddot{x} = -\omega^2 \cos(\omega t) , \quad (4.1)$$

where ω is frequency and t represents the time, it is easy to understand that the acceleration field highly depends on the frequency. This fact can explain the increasing of the maximum nodal accelerations when using greater frequencies (see graph presented in Figure 4.19). As already mentioned, the time increments used for the accelerations analyses demonstrated not to be reasonable and then, it should be used lower ranges of Δt .

The high accelerations obtained are not alarming, since the displacements remain within a low range, as presented by the analyses performed in this chapter. Further, for the range of train speeds analysed in this work (0 - 140 km/h), the accelerations are always below the acceleration limit imposed by the EN1991-2 (2003) for ballastless track bridges of 5 m/s^2 .

Chapter 5

Fatigue Analysis

5.1 Introduction

In this chapter, the fatigue analysis of the Varzeas Bridge is carried out using the methodologies described in Chapter 3.

The purpose of Section 5.2 is to clarify some assumptions and characteristics that were study before the analysis, defining aspects such as the nodes that were relevant to this analysis, the number of vibration modes to be considered, the damping to be assigned to the model and the time increment required to obtain accurate results. In Section 5.3, the stresses are evaluated for the studied critical details, while in Section 5.4 the fatigue damage caused by the passage of a single train and by a traffic mix for a year is calculated for those same details.

Finally, some conclusions regarding the present study are drawn at the end of the chapter.

5.2 Bases for the Fatigue Analysis

5.2.1 Variables Extracted

The relevant parameters extracted from this fatigue analysis are presented below:

- Stress analysis at the critical detail corresponding to the connection between the upper cross-beam and the upper chord of the vertical trusses at the mid-span of span 3;
- Stress analysis at the same critical detail described above but over the support between span 2 and 3;
- Fatigue damage analysis in the previously mentioned critical details, according to the accumulated damage method proposed in EN1993-1-9 (2004).

In Figure 5.1, the location of the critical details evaluated is clarified.

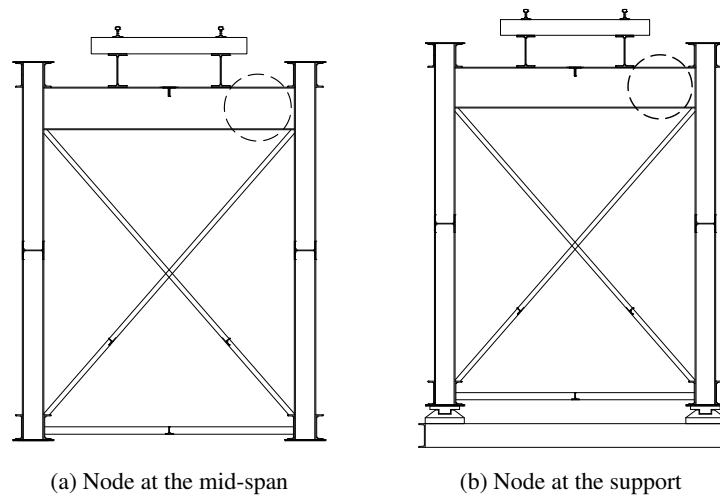


Figure 5.1: Location of the critical details evaluated.

5.2.2 Range of Speeds

It was defined that the fatigue analysis would be performed for the same speed range adopted for the dynamic analysis described in Chapter 4 - between 5 and 140 km/h with a speed interval of 5 km/h.

5.2.3 Number of Vibration Modes

As stated in Section 4.6, the displacements analyses appear to be accurate when considering the contribution of modes with frequencies up to 100 Hz (2150 vibration modes). In the present analysis, it was considered the contribution of the vibration modes with frequencies up to:

- 30 Hz, equivalent to 320 vibration modes;
- 60 Hz, equivalent to 1150 vibration modes;
- 100 Hz, equivalent to 2150 vibration modes.

5.2.4 Damping Coefficient

As mentioned in the previous chapter, and since this bridge is a steel bridge, the adopted damping coefficient for all modes according to Eurocode EN1991-2 (2003) is 0.5%.

5.2.5 Time Increment

In the previous chapter, it was demonstrated that a time increment of 2 ms provides accurate performances of the displacements analysis. Therefore, in fatigue analysis, 2 ms was also the time increment adopted.

5.3 Stress Evaluation

5.3.1 Loading Model

Based on the methodology described in Chapter 3, and already used in Chapter 4 for the nodal displacements and the nodal accelerations extraction, it is possible to obtain the nodal forces. From here, and as will be explained in this chapter, it is possible to evaluate the stresses at the critical details studied. The traffic selected for this analysis was only composed by the Locomotive-hauled freight train (type 5) and it is described in Figure 5.2.

$$\Sigma Q = 21600\text{kN} \quad V = 80\text{km/h} \quad L = 270,30\text{m} \quad q = 80,0\text{kN/m'}$$

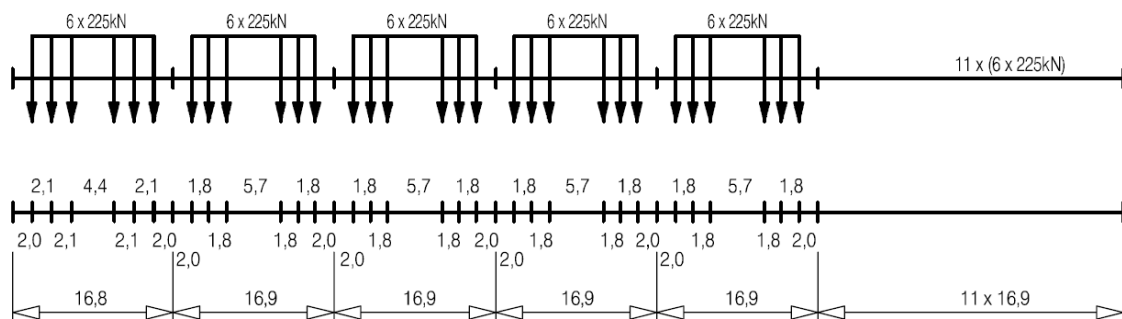


Figure 5.2: Locomotive-hauled freight train - Type 5 (adapted from EN1991-2 (2003)).

5.3.2 Stress Calculation

First of all, it was necessary to characterize the beam sections where the selected nodes are located. As previously mentioned in Section 5.2.1, the critical detailed analysed in the present work consists in the connection between the cross-beam and the upper chord of the vertical trusses. Figure 5.3 shows the cross-section of the cross-beam for which the stresses have been analysed in order to later evaluate the fatigue damage caused by the passage of trains. The relevant parameters that are required for computing the stresses are presented in Table 5.1.

Table 5.1: Section geometrical properties.

Area $A \text{ (m}^2\text{)}$	Distance $y_{\max} \text{ (m)}$	Distance $z_{\max} \text{ (m)}$	Inertia Moment $I_{yy} \text{ (m}^4\text{)}$	Inertia Moment $I_{zz} \text{ (m}^4\text{)}$
0.026738	0.1485	0.344	0.002188	0.123×10^{-3}

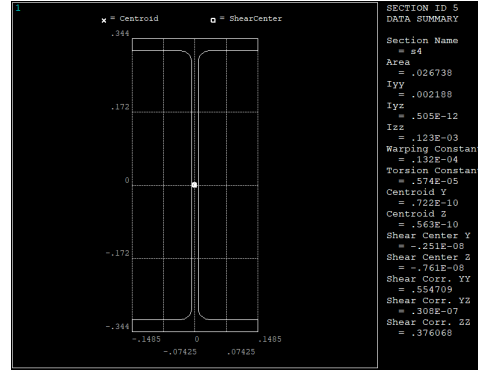


Figure 5.3: Beam section I.

Calculating the section modulus:

$$W_y = \frac{I_y}{z_{max}} = \frac{0.002188}{0.344} = 0,00636 m^3, \quad (5.1)$$

$$W_z = \frac{I_z}{y_{max}} = \frac{0.123 \times 10^{-3}}{0.1485} = 0,00083 m^3, \quad (5.2)$$

and considering the internal forces, already extracted from ANSYS® - axial force, N , and moments about y and z axes, M_y and M_z , respectively - it is possible to obtain the stresses at both nodes, using the following expression:

$$\sigma = \frac{N}{A} + \frac{M_y}{W_y} + \frac{M_z}{W_z}. \quad (5.3)$$

5.3.3 Analysis of the Critical Detail at the Mid-span

The maximum stresses obtained in the critical detail at the mid-span, as a function of the train speed and number of modes that contribute to the response, are plotted in Figure 5.4. The maximum stress verified was 47 MPa for an analysis considering modes with frequencies up to 60 Hz, when the train speed is 80 km/h. The graph demonstrates that the nodal stresses remain almost constant when the train speed is increased. The trendlines obtained by the analyses considering modes with different frequencies show a substantial difference. The analyses considering modes with frequencies up to 60 and 100 Hz represent a stresses difference of approximately 22% and the analysis assuming frequencies up to 30 Hz is the one that provides the most distinct values of nodal stresses. These differences are highlighted in Figure 5.5, where the time-history of the nodal stresses for a train speed of 130 km/h is plotted for the different mode contributions.

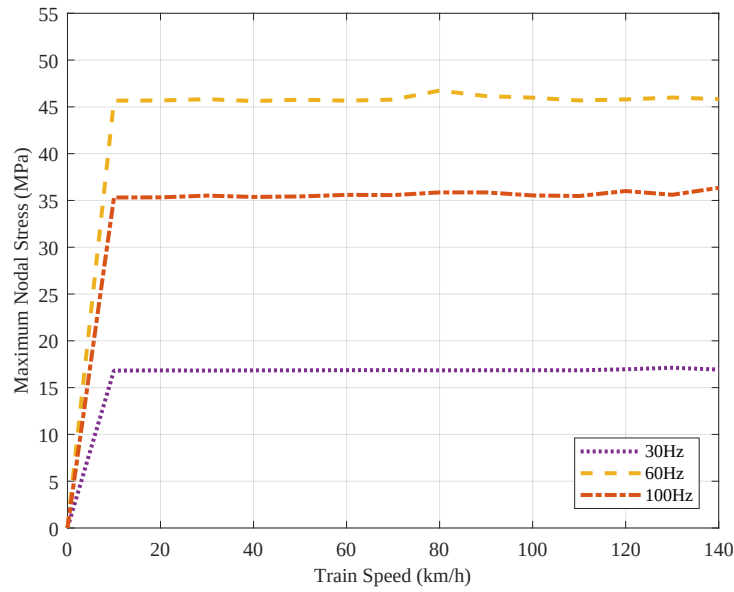


Figure 5.4: Maximum stresses of the node at the mid-span ($\Delta t = 2ms$).

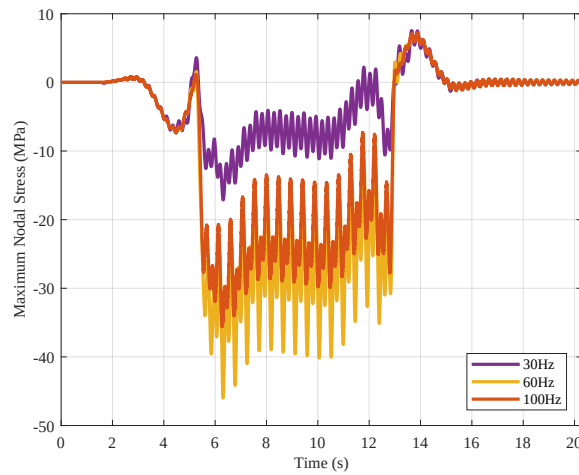


Figure 5.5: Stresses time-history of the node at the mid-span for a train speed of 130 km/h ($\Delta t = 2ms$).

5.3.4 Analysis of the Critical Detail at the Support

The graph presented in Figure 5.6 is relative to the node located over the bridge support. The maximum stress obtained by the analysis is 50 MPa and again, for the scenario where the contributions of modes up to 60 Hz are taken into account. Despite the fact that the stresses calculated in these analyses are greater than the ones obtained for the mid-span node, both scenarios are similar. The speed increasing does not affect the nodal stresses values and, consequently, the trendlines remain almost horizontal. Figure 5.7 shows the time-histories of the stresses obtained in the node located at the support when the train runs at 130 km/h. As previously mentioned, the maximum stresses occur when the contributions of modes up to 60 Hz are considered in the calculations.

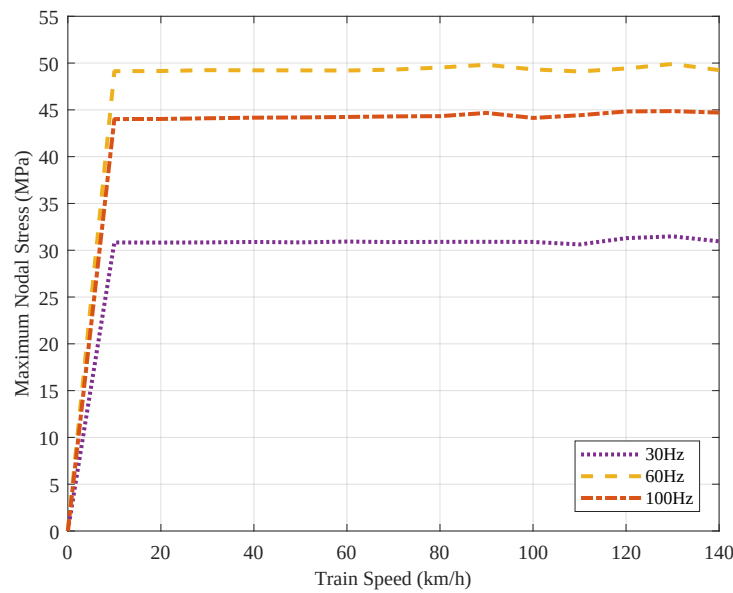


Figure 5.6: Maximum stresses of the node at the support ($\Delta t = 2ms$).

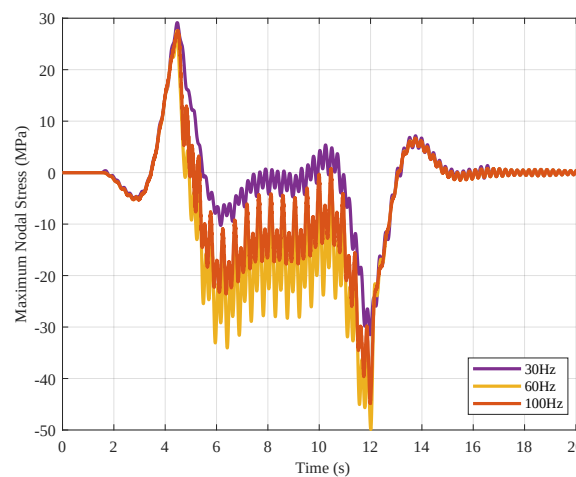


Figure 5.7: Stresses time-history of the node at the support for a train speed of 130 km/h ($\Delta t = 2ms$).

5.4 Fatigue Damage

In this section, it is presented some fatigue damage analyses considering the two nodes previously selected. The calculations were performed using a program developed by Souto et al. (2019) called Fatigue Damage Tool (FDT). The present section starts with a brief presentation of the aforementioned software, followed by the presentation of the results obtained in the fatigue analysis performed with that software.

5.4.1 Fatigue Damage Tool Software

5.4.1.1 Input Parameters

The first step consists in defining the parameters that are to the program initiation. As shown in Figure 5.8, it is mandatory to select the stress type, fatigue strength curve (slope and construction detail definition), partial factor for fatigue strength, partial safety factor for fatigue loading and number of repetitions. The following variables were set for all fatigue damage calculations performed in this work:

- Stress type: since only normal stresses are evaluated, select direct stresses;
- Partial factor for fatigue strength: it was considered a damage tolerant assessment method and a low consequence of failure;
- Partial safety factor for fatigue loading: it was used the default value of 1;
- Number of repetitions: it was assumed only one train passage per analysis performed in this program.

Figure 5.8: Fatigue Damage Tool - input parameters interface.

Different S-N fatigue curves have been used to assess the fatigue damage in the aforementioned critical details. According to Kühn et al. (2008), for riveted structures, detail category 71 should be used where the nominal stress $\Delta\sigma$ has to be calculated on the net cross-section. This represents a code gap in this assessment because its criteria do not include information for riveted structures, such as the Vázear bridge, it only includes regulations directed to welded and bolted structures. Thus, the S-N curve 71 defined in EN1993-1-9 (2004) was used and its detail category corresponds to $\Delta\sigma_C = 71$ MPa.

The other S-N curve used was the Taras and Grainer (TG) S-N curve 90 (Taras and Greiner, 2018), whose reference fatigue resistance $\Delta\sigma_C$ is equal to 90 MPa. This curve was adopted due to the similarity between the constructional detail presented by Taras and Greiner (2018) and the connection detail evaluated in the present study. The characteristics of both curves are presented in Table 5.2.

Table 5.2: 71 and TG 90 S-N curves characteristics.

Parameters	S-N curve 71	S-N curve TG 90
Detail Category, $\Delta\sigma_C$ (MPa)	71	90
Fatigue Limit, $\Delta\sigma_D$ (MPa)	52.31	74.93
Cut-off Limit, $\Delta\sigma_L$ (MPa)	28.73	-
Slope, m	$m_1 = 3, m_2 = 5$	m=5

5.4.1.2 Stress-Time History

The program interface relative to this step is presented in Figure 5.9. After loading a text file with the stress time-history for a certain scenario (see Section 5.3), the software automatically plots that time-history in the interface. This step corresponds to the step represented in 3.6b, defined in EN1993-1-9 (2004).

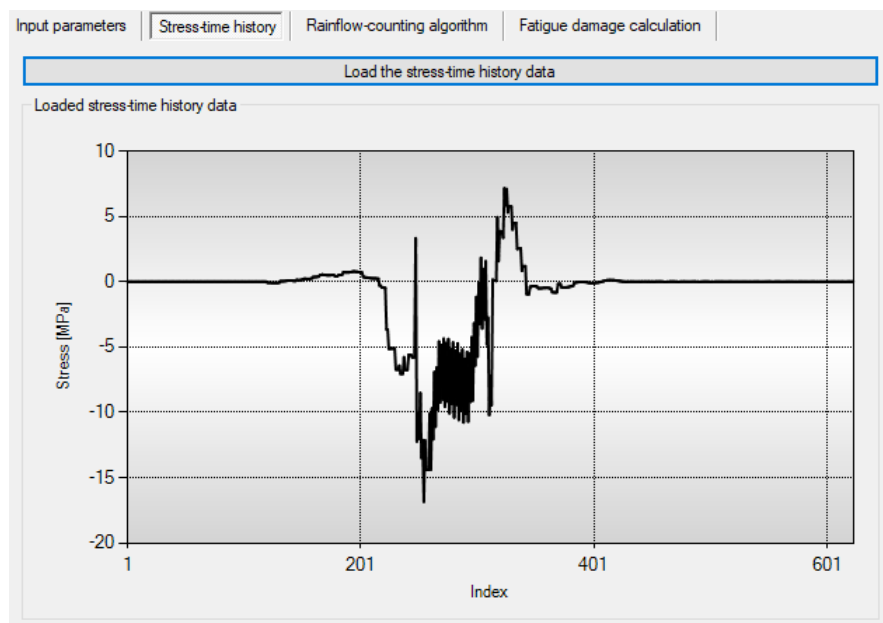
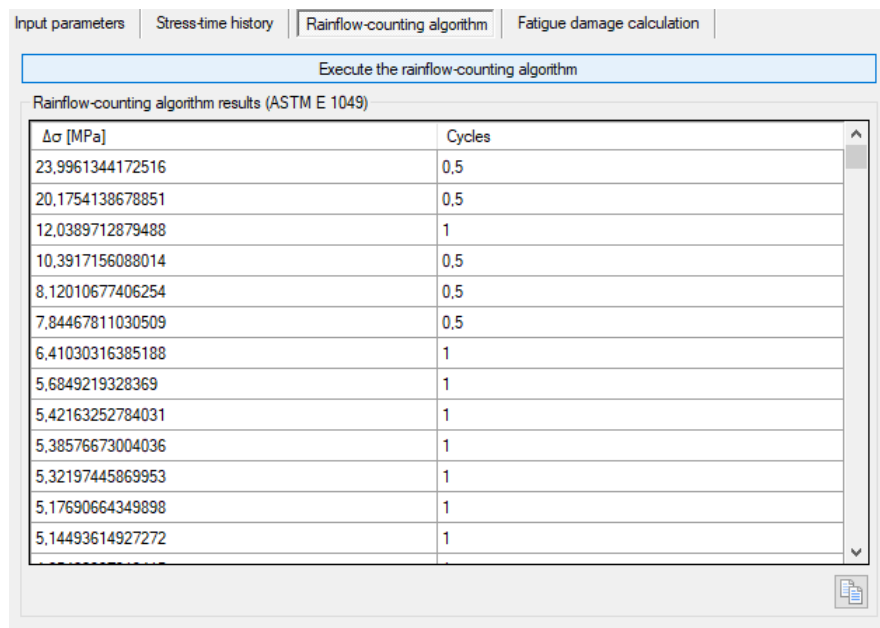


Figure 5.9: Fatigue Damage Tool - stress-time history interface.

5.4.1.3 Rainflow-Counting Algorithm

The stress cycles counting is executed using an appropriate methodology, in this case, the rainflow method which is described in E1049-85 (2017) and represented in Figure 3.6c. After the counting, the program interface relative to this process, which is represented in Figure 5.10, shows the outputs obtained: the cycles amplitude, $\Delta\sigma$, and the corresponding number of cycles. This step was already approached and it is represented in Figure 3.6d.



$\Delta\sigma$ [MPa]	Cycles
23,9961344172516	0,5
20,1754138678851	0,5
12,0389712879488	1
10,3917156088014	0,5
8,12010677406254	0,5
7,84467811030509	0,5
6,41030316385188	1
5,6849219328369	1
5,42163252784031	1
5,38576673004036	1
5,32197445869953	1
5,17690664349898	1
5,14493614927272	1

Figure 5.10: Fatigue Damage Tool - stress cycles counting interface.

5.4.1.4 Fatigue Damage Calculation

The accumulation of the fatigue damage is evaluated using the Palmgren-Miner rule applied to the several stress amplitude ranges (see Equation 5.4), as previously described in Section 3.3.3 and represented in 3.6f. Figure 5.11 shows the interface of FDT relative to this last step, in which the fatigue damage is equal to zero, meaning that all stress amplitude intervals are below the fatigue cut-off limit.

$$D = \sum \frac{n_i}{N_i} \leq 1 \quad (5.4)$$

The results relative to the fatigue damage calculation caused by the passage of a single train and by a traffic mix for a year are presented in the following section.

Input parameters Stress-time history Rainflow-counting algorithm **Fatigue damage calculation**

Calculate the fatigue damage using the Palmgren-Miner rule

Fatigue damage results

$\Delta\sigma$ [MPa]	$\Delta\sigma_{\gamma FF}$ [MPa]	Cycles	Slope, m	n_i	N_i	n_i/N_i
23,99613441...	23,99613441...	0,5	-	-	-	-
20,17541386...	20,17541386...	0,5	-	-	-	-
12,03897128...	12,03897128...	1	-	-	-	-
10,39171560...	10,39171560...	0,5	-	-	-	-
8,120106774...	8,120106774...	0,5	-	-	-	-
7,844678110...	7,844678110...	0,5	-	-	-	-
6,410303163...	6,410303163...	1	-	-	-	-

Fatigue damage check

Fatigue damage accumulation, $D = \sum (n_i/N_i)$:

Design fatigue factor, DFF:

 $\mu = 1/DFF$:

Figure 5.11: Fatigue Damage Tool - fatigue damage calculation interface.

5.4.2 Fatigue Analysis Results

5.4.2.1 Fatigue Damage Caused by a Single Train Passage

For this analysis, it was assumed only a single passage of the Locomotive-hauled freight train (type 5) presented in EN1991-2 (2003). In Figure 5.12, it is presented the results obtained by the fatigue analysis in the node located at the mid-span for single train passage and considering the S-N curve 71.

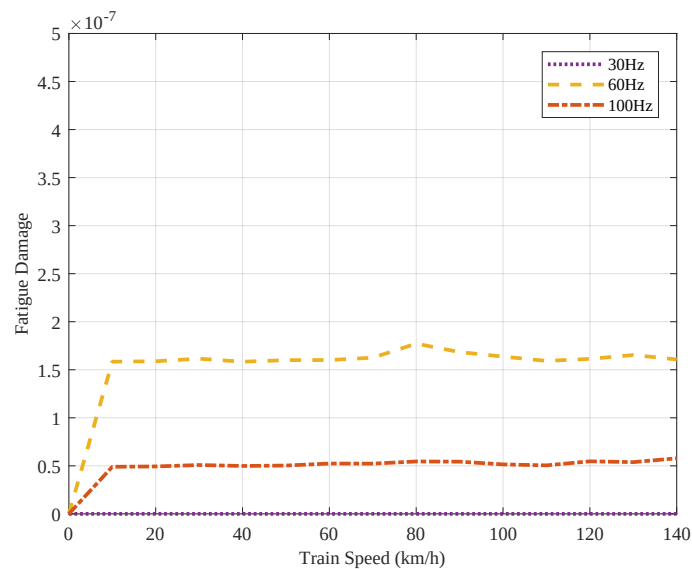


Figure 5.12: Fatigue damage in the node located at the mid-span caused by a single train passage - S-N curve 71.

For an analysis considering modes with frequencies up to 30 Hz, it was not verified fatigue damage in the critical detail. However, when considering a higher number of modes, the damage is no longer null, proving the importance of evaluating the number of modes that should be considered in the dynamic analysis. The maximum value of fatigue damage occurs for a train speed of 80 km/h (60 Hz). The fatigue damage analysis considering the S-N curve TG 90 is not presented due to the fact that the fatigue damage obtained in that analysis was null.

The graph presented in Figure 5.13a shows the results calculated by the fatigue analysis for the node located at the support for a single train passage considering the S-N curve 71. The most evident difference between this graph and the graph presented in Figure 5.12 is the fact that the values obtained in this analysis are substantially higher.

On the other hand, the graph presented in Figure 5.13b shows that the results obtained by the analyses considering modes with frequencies up to 30 and 100 Hz are null. The maximum value of the fatigue damage verified for this critical detail, considering the different curves, occurs for the train speed of 130 km/h (60 Hz).

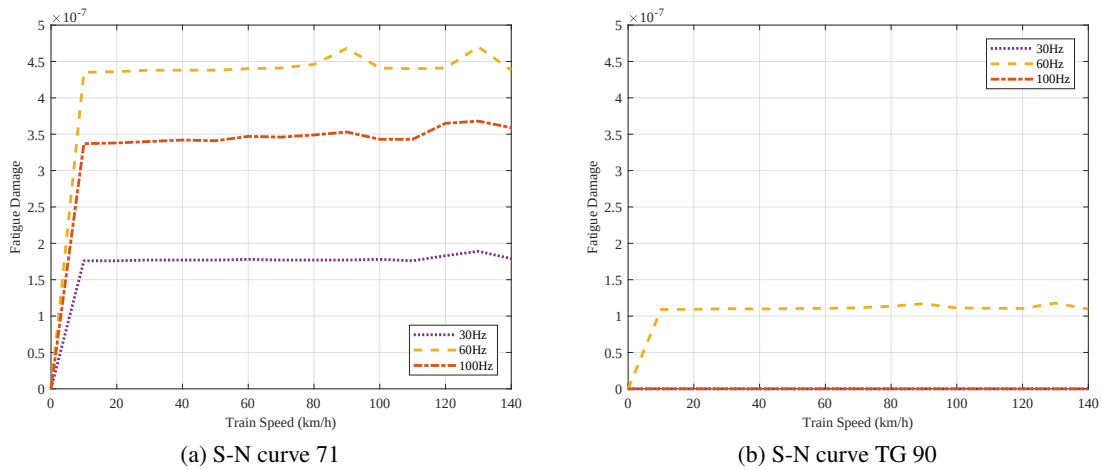


Figure 5.13: Fatigue damage in the node located at the mid-span caused by a single train passage.

5.4.2.2 Fatigue Damage Caused by a Traffic Mix for a Year

The aim of this section consists in calculating the fatigue damage caused by a heavy traffic mix for a year. Therefore, it was considered the daily passage of 32 Locomotive-hauled freight trains to match the traffic volume relative to the heavy traffic mix settled by for a year EN1991-2 (2003) (see Table 2.6) - $24.78 \times 10^6 t$. The calculation that led to this assumption is presented in Equation 5.5. The value of 2160 represents the mass of the train considered (see Table 3.3).

$$\frac{24.78 \times 10^6}{2160 \times 365} = 31.4 \text{ trains/day} \quad (5.5)$$

For the node located at the mid-span, it was verified that the maximum fatigue damage value occurred for a train speed of 80 km/h, considering modes with frequencies up to 60 and 100 Hz

(see Figure 5.12) and therefore, those were the damage values used in the calculations. Paying attention to the bar graph presented in Figure 5.14, it is verified that the fatigue damage values provided by the analysis using the S-N curve TG 90 were null. Considering now the analysis using the S-N curve 71, the highest fatigue damage value obtained was 2.07×10^{-3} and it occurs when it was performed an analysis considering modes with frequencies up to 60 Hz, as expected. The values calculated when it is considered frequencies up to 100 Hz are lower and null for frequencies up to 30 Hz.

Calculating the bridge lifetime according to the approaches to this critical detail:

$$\frac{1}{2.07 \times 10^{-3}} \cong 483 \text{ years} , \quad (5.6)$$

it is expected that the bridge has a lifetime of 483 years since its construction.

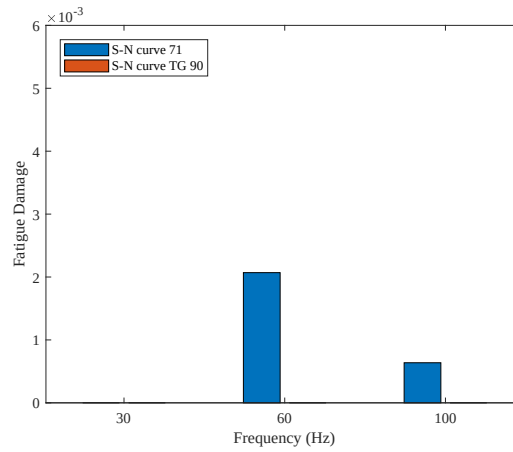


Figure 5.14: Fatigue damage in the node at the mid-span caused by the heavy traffic mix assumed for a year (train speed = 80 km/h).

The graph presented in Figure 5.13 shows that the worst scenario for all analyses occurs for a train speed of 130 km/h. Then, the fatigue damage values used for the generation of the graph presented in Figure 5.15 were the values obtained for that train speed. In contrast with the analysis of fatigue damage in node located at the mid-span, both analyses considering the different S-N curves provided non-null results, at least for analysis assuming nodes frequencies up to 60 Hz. The highest fatigue damage value obtained for a year was 5.49×10^{-3} considering the S-N curve 71 and 1.33×10^{-3} considering the S-N curve TG 90.

The bridge lifetime considering the analysis of the node located at the support, when adopted the S-N curve 71, is calculated by the following expression:

$$\frac{1}{5.49 \times 10^{-3}} \cong 182 \text{ years} , \quad (5.7)$$

and, on the other hand, using the S-N curve TG 90:

$$\frac{1}{1.33 \times 10^{-3}} \cong 752 \text{ years} . \quad (5.8)$$

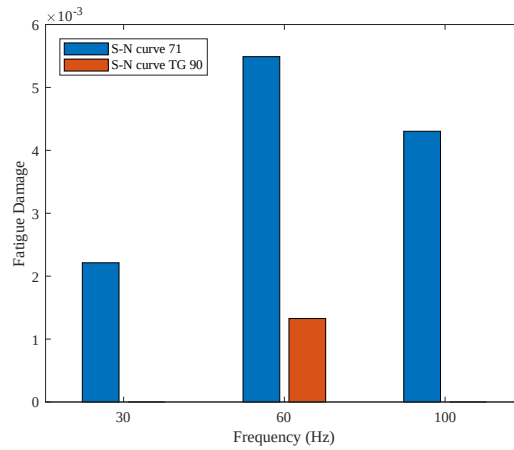


Figure 5.15: Fatigue damage in the node located at the support caused by the heavy traffic mix assumed for a year (train speed = 130 km/h).

It is expected that since its construction the bridge has a lifetime of 182 years for the analysis considering the S-N curve 71 and 752 years for the analysis considering the S-N curve TG 90.

5.5 Results Discussion

Analysing the stress and the fatigue damage analyses performed in this chapter, it was proved that the most critical detail is located at the support. Consequently, and as demonstrated in Section 5.4.2.2, when it is considered the S-N curve 71, the critical detail represented by the node located at the support presents a lower bridge lifetime when compared to the critical detail represented by the node located at the mid-span. Normally, a bridge is projected in order to provide a safe operation during at least 100 years. The calculations performed showed that, for both nodes, that minimum lifetime was surpassed.

The results obtained by the fatigue damage analyses showed lower values for the ones which the S-N curve TG 90 was used. This can be explained by the fact that the S-N curve 71 has lower values of detail category, $\Delta\sigma_C$, and cut-off limit, $\Delta\sigma_L$ (see Table 5.2), which means that there are more stress cycles that will contribute to the damage accumulation of fatigue damage. Consequently, this curve is more conservative than the S-N curve TG 90 and that is why it is widely used in the analysis of riveted structures.

The traffic mix select was a conservative choice too. In this bridge, the traffic is composed by trains which are lighter and slower than the Locomotive-hauled freight train. Further, the traffic mix adopted in this analysis is used for dimensioning and therefore, this scenario is very conservative when compared to a scenario where the real traffic is considered.

It was verified that the results obtained by all analyses performed in this chapter showed higher values for the analysis considering modes with frequencies up to 60 Hz. This fact proves the importance of evaluating the number of modes that should be considered in the dynamic analysis.

Chapter 6

Conclusions and Future Developments

6.1 Conclusions

This work consisted in studying the dynamic behavior of the Várzeas bridge and the fatigue performance of one of the critical details belonging to this bridge. Currently, the bridge model is being the target of several studies and therefore, the present work aimed to contribute in different ways to those researches.

Initially, the dynamic analysis was performed for two different nodes at mid-span of span 3: (i) a node located at the cross-beam and (ii) a node located at the vertical main truss. In the present work, a numerical methodology of moving load models to simulate the train passage based on the modal superposition method was adopted to evaluate the dynamic response of the bridge. This method was chosen due to its efficiency in computing any type of quantity, such as displacements, velocities, accelerations and forces. Several sensitivity analyses were performed in order to evaluate the influence of the number of vibration modes and the time increment adopted on the dynamic response of the bridge.

The results provided by the analysis of the displacements were reasonable and accurate, however for the accelerations analyses, it was not possible to reach a commitment between precision and computational effort. In other words, using the different number of vibration modes and time increments it was not obtained a convergence of results. This occurs due to the fact that this analysis highly depends on the number of vibration modes adopted. There is also a significant contribution of the local modes to the dynamic response obtained, especially in the control point situated at the cross-beam. Nevertheless, for the range of train speeds analysed in this work (0 - 140 km/h), the accelerations are always below the acceleration limit imposed by the EN1991-2 for ballastless track bridges of 5 m/s².

The fatigue damage analysis was performed for a critical detail - the connection between the upper cross-beam and the upper chord of the vertical trusses - located at two different parts of the bridge: the first located at the mid-span of span 3, and the other located at the support between span 2 and 3. The calculations proved that the most critical detail was the one located at the bridge support due to the higher values of stresses and fatigue damage obtained. Consequently, and using

the Linear Damage Accumulation Method, this critical detail presents a lower bridge lifetime when compared to the critical detail represented by the node located at the mid-span. For the most critical detail, the bridge residual life calculated, considering the S-N curve 71 provided by the EN1993-1-9 and the S-N curve TG 90 provided by the research performed by Taras and Greiner, was 182 and 752 years, respectively. The same calculation procedure for the mid-span node showed that the bridge lifetime since its construction is 483 years considering the S-N curve 71 and infinite according to S-N curve TG 90. The higher values of fatigue damage and consequently, the lower residual life obtained when considering the S-N curve 71, is justified by the fact that this curve has a lower reference value for fatigue strength. This result shows that using the S-N curve 71 for any type of riveted detail, as proposed by EUR2325EN, is conservative, since studies presented by other authors, in this particular case by Taras and Greiner, show that for these type of structural details less conservative curves may also be used. Nevertheless, since EN1993-1-9 does not present any type of classification for riveted connections, the adoption of a conservative S-N curve is advisable and recommend. Furthermore, the traffic mix adopted in the present study is also a conservative assumption, since this traffic scenario is used to design new bridges, thus representing a much higher value of tonnage when compared to the real traffic that actually passes (and passed in the past) in the bridge.

Throughout all fatigue analyses, it was verified that the results obtained by all analyses performed, considering modes with frequencies up to 60 Hz, showed higher values of stresses. This evidence enhances the importance of performing sensitivity analyses to evaluate the number of modes that should be adopted in order to provide accurate results.

6.2 Future Developments

Further improvements could be done on the dynamic analysis, such as evaluating the structure response considering modes with several frequencies between 60 and 100 Hz. In this way, the divergence of results obtained in the stress analysis could be study and discussed more efficiently.

It would be interesting to analyse the fatigue of the bridge using advanced methods and perform the passage of trains with higher speed to discuss the differences obtained for the nodal stresses and, consequently, for the fatigue damage induced.

Finally, a safety analysis would be an improvement to this work, according to EN1990-A2 and EN1991-2, aiming the evaluation of the structure and railway safety, and also the passengers comfort.

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