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Dynamic performance of existing double track railway bridges at resonance with the increase of the operational line speed

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ABSTRACT

This article addresses the dynamic behaviour of double track simply supported bridges of short to medium span lengths (10 m < L < 25 m) belonging to conventional railway lines. These structures are susceptible to experience inadmissible levels of vertical vibrations when traversed by trains at high speeds, and in certain cases their dynamic performance may require to be re-evaluated in case of an increase of the traffic velocity above 200 km/h. In engineering consultancies, these structures have been traditionally analysed under the passage of trains at different speeds using planar models, neglecting the contribution of transverse vibration modes and also the flexibility of the elastomeric bearings. The study presented herein endeavours to evaluate the influence of these two aspects in the verification of the Serviceability Limit State of vertical accelerations, which is of great interest in order to guarantee a conservative prediction of the dynamic behaviour. In the present study, the dynamic response of representative slab and girder bridges has been evaluated using an orthotropic plate finite element model, leading to practical conclusions regarding the circumstances under which the above mentioned factors should be considered in order to adequately evaluate the transverse vibration levels of the deck.

1. Introduction

During the last decades the extensive construction of new high-speed lines in developed and developing countries as well as the increase of the operating train velocity in the existing ones (above 200 km/h) has risen the concern of scientists and engineers for the dynamic behaviour of railway bridges. The regular and repetitive nature of groups of train axle loads can induce resonance situations in these structures, a phenomenon that takes place when the time interval between the passage of repeated groups of loads is a multiple of one of the natural periods of the bridge. $\widehat{\Psi}_n = 1$.

Regarding vertical accelerations, short-to-medium span simply supported (S-S) bridges (span lengths ranging between 10 and 25 m) are specially critical, and may experience considerably high amplifications of the acceleration levels due to resonance, entailing harmful consequences [1,2]. Hence the Serviceability Limit State of vertical acceleration prescribed by Eurocode (EC) [3] (3.5 m/s^2 for ballasted tracks, to avoid ballast instability) is one of the most demanding requirements and becomes crucial for the design of railway bridges.

A number of conventional railway lines has been partially adapted for high-speed traffic. Some representative examples are the Madrid-Sevilla and Valencia-Barcelona railway lines in Spain, as well as the first European high-speed line, Paris-Lyon. When a line is upgraded and existing bridges are unable to fulfil the Standards, the horizontal structures are sometimes replaced by new decks with higher transverse stiffness. Alternatively, the strengthening procedure consists in a partial embedment of the abutments, leading to a kind of portal frame or integral bridge [1]. On the other hand, a number of researchers have evaluated in the past years the possibility of applying passive control techniques [4–7] that could avoid the deck replacement by increasing structural damping. These facts point out the importance of using accurate enough numerical models, able to realistically predict the vibration levels in the deck with reasonable computational costs. Superfluous refinements are to be avoided, since engineers will employ them as a tool for deciding (according to the standards) what is the most adequate retrofit solution from the economical and technical point of view in each particular case.

Traditionally, planar numerical S-S beam models are very common in literature (see [8–10]). These models appear to be valid for single track, non-skewed bridges, since the response of this type of structures at resonance is mainly governed by the first flexural

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mode [10]. Nevertheless the contribution of three-dimensional modes, such as the first torsion mode of the bridge, could be significant in double track decks, due to the eccentricity of the loaded track and the proximity between the first bending and the first torsion natural frequencies in short span bridges. The quick development of computational technologies and the versatility of numerical methods have promoted the use of three-dimensional models for investigation purposes in recent years (see [11,12]), but their application for railway bridge dynamic analyses is still less frequent in engineering consultancies unless a singular structure is designed. An explanation to this tendency could be found in the regulations in force at each particular country. Eurocode 1 (EC1) [11], which will be adopted in most of the European countries in the near future, encourages the use of planar models when the frequency of the first torsion eigenform exceeds 1.2 times that of the first longitudinal bending mode (for non skewed beam or plate type decks on rigid supports).

Also EC establishes the minimum number of modes required for an accurate mode superposition analysis, which is a computationally efficient technique in structures with linear behaviour. In this regard EC recommends the consideration of the natural frequencies and the corresponding mode shapes up to the greater of (i) 30 Hz, (ii) 1.5 times the frequency of the fundamental mode of vibration or (iii) the frequency of the third one. However, this limitation highlights a potential inconsistency: when the natural frequency of the first torsion mode is higher than 1.2 times the fundamental frequency but falls below this previous criteria, should or should not be taken into account. This matter has not been reported yet in the scientific literature and is one of the main issues analysed in the present study.

On the other hand, for practical purposes the vertical stiffness of laminated elastomeric bearings found in short simply supported bridges tends to be neglected by engineers for several reasons: in first place, these elements introduce spurious high-frequency oscillations in the predicted dynamic response, which can only be attenuated by including the track in the model or simulating its distributive effect at the entrance and exit of the loads using specific functions [6], thus further complicating the modelling task. Secondly, the elastomeric bearings are rather stiff, and are traditionally assumed rigid in the vertical direction for practical purposes. However, several studies show that their vertical stiffness may affect the dynamic response of the structure when subjected to railway traffic [13].

In the present contribution the authors have simulated the dynamic behaviour of S-S reinforced concrete slabs and prestressed concrete girder bridges belonging to conventional lines in which an increase of the maximum design velocity is envisaged. The numerical models adopted for this investigation follow purposely the main simplifications and tendencies adopted by engineers for practical applications (which are also in accordance with the European Standards) in a view to analyse their suitability. The selected case studies are intentionally restricted to double track bridges, since these are common structures in existing railway lines and may experience a significant contribution of the first torsion mode, with a natural frequency in the vicinity of the fundamental one. The results and conclusions presented herein provide an enhanced understanding of the importance of using three dimensional numerical models and of the effect of the elastic bearings for the assessment of the maximum acceleration levels in double track decks.

2. Theoretical background

The maximum dynamic response of a bridge under the circulation of trains is mainly conditioned by two of the classical phenomena related with the moving load problem: resonance and cancellation. For that reason, in a view to obtain a theoretical basis to enhance the understanding of the elastic supports effect on the dynamic response, these phenomena have been studied in a first approach by using the simplest beam model: a Bernoulli-Euler beam supported on vertical elastic supports (Fig. 1). The starting point of this investigation are the results published by the authors in [14], where the effect of the supports vertical stiffness on the frequency and amplitude of the resonant response of elastically supported (E-S) beams is analysed in detail.

The exact frequency equation and mode shapes of the E-S beam can be found in [15,16], neglecting structural damping, shear deformation and rotary inertia effects. In this case the frequency equation is given by

$$\left(\frac{\pi^3}{\kappa}\right)^2+\frac{\pi^3}{\kappa}\lambda^3\frac{\sinh(\lambda)\cos(\lambda)-\cosh(\lambda)\sin(\lambda)}{\sin(\lambda)\sinh(\lambda)} + \lambda^6\frac{1-\cos(\lambda)\cosh(\lambda)}{2\sin(\lambda)\sinh(\lambda)}=0, \tag{1}$$

with $\lambda_n=\lambda_n(\kappa)$ being the roots of Eq. (1), $\kappa=EI_z\pi^3/(K_vL^3)$ the ratio of the flexural stiffness of the beam to the vertical stiffness of the elastic bearings, where L, K_v and EI_z are the beam length, vertical stiffness of the support and bending stiffness of the cross section, respectively. The circular frequencies are then defined as follows,

$$\omega_n=\left(\frac{\lambda_n}{L}\right)^2\sqrt{\frac{EI_z}{m}} \tag{2}$$

with m being the linear mass of the beam. In the S-S case, $\kappa=0$ and $\lambda_n=n\pi$, leading to the well-known natural frequencies and mode shapes of this particular case. For the E-S beam, the analytical expressions of the eigenforms are

$$\phi_n(\kappa,l)=\frac{\psi_n(\kappa,l)}{\max|\psi_n(\kappa,l)|} \tag{3}$$

where $l=x/L$ and $\psi_n(\kappa,l)$ is defined as,

$$\psi_n(\kappa,l)=\sin(\lambda_nl)+\sinh(\lambda_nl)\frac{\sin(\lambda_n)}{\sinh(\lambda_n)}+\gamma_{1n}(\cos(\lambda_nl)+\cosh(\lambda_nl) + \gamma_{2n}\sinh(\lambda_nl)),\quad 0\leqslant l\leq 1; \tag{4a}$$

$$\gamma_{1n}=\frac{\sinh(\lambda_n)-\sin(\lambda_n)}{\frac{2}{\kappa}\left(\frac{\pi}{\lambda_n}\right)^3\sinh(\lambda_n)+\cos(\lambda_n)-\cosh(\lambda_n)} \tag{4b}$$

$$\gamma_{2n}=\frac{\cos(\lambda_n)-\cosh(\lambda_n)}{\sinh(\lambda_n)} \tag{4c}$$

The phenomena of resonance and cancellation are both associated to the free vibrations created by each of the axle loads that have crossed the structure. Such free vibrations can possibly

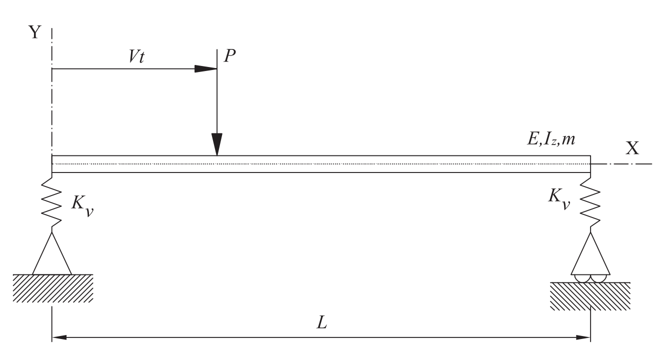

Fig. 1. Elastically supported beam traversed by a moving load at constant speed.