Vibration of the railway track-viaduct system under moving vehicles taking into account the interaction effect

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Abstract

The main purpose of this paper is to investigate the influence of the interaction on the dynamic behaviour of a simply supported single span railway concrete viaduct for which measurement results are available, taking into account the railway track. We compare the computed acceleration response of the track-bridge system with and without interaction considering the dynamic characteristics of the real moving vehicles and taking into account the available modal identification and the acceleration response measurements carried out during the field tests.

Furthermore, this paper analyse the dynamic behaviour of the railway track and viaduct under the action of high speed vehicles according to EN1990. The moving vehicles are modelled as a series of two degree of freedom mass-spring–damper systems at the axle location The track modelling consists of beam elements for the rails and a series of springs and dampers for the ballast and the sleepers.

1 Introduction

The dynamic response of railway bridges when subjected to moving vehicles has been a topic of research with increasing interest in the field of the bridge engineering. In the last years many scientist and engineers have studied the implementation of the high speed train in several countries. In some European countries there is an effort of re-evaluation of existing railway bridges for the increasing train speeds and interoperability conditions.

A large number of the existing bridges are classified as short span, up to 20 m, or medium span, up to 30 m. In these types of bridges the dynamic behavior is an important issue, since the displacements and the accelerations can be significantly amplified when the resonance speed is reached. As a consequence, the evaluation of the dynamic response is important to define maintenance procedures or, even, to decide about the strengthening or replacement of the bridges.

However, the evaluation of the dynamic response of those bridges can become complex task, since the influence of some parameters is not yet well defined, such as:

- the boundary conditions of the bridge deck, specially the continuity of the rail track over the supports and the soil-structure interaction for short bridges;

- the behavior of the ballast: its contribution to the structural dynamic behavior and the maximum accelerations allowed to ensure low maintenance and appropriate safety levels;

- the overall damping, including non-structural damping, such that a realistic resonant response of the bridge during the passage of the train can be predicted;

- the methodology for the load modeling: moving loads *versus* train-track-structure interaction scheme;

The real influence of the above aspects, or conditions, can be evaluated only when measurement data is available, to compare with the numerical results. The investigation that has been done recently by the authors focus on the dynamic behavior of simply supported single span railway concrete viaducts for which measurements were carried out and results were obtained [1,2,3], and on the numerical modeling of the track/bridge system when the moving loads model and the train-track-structure interaction model are considered .

The main purpose of this paper is to show how the bridge response predicted by the train/bridge interaction model compares with the response obtained by the moving loads model. This is also an issue that the specialist's committee D214 of the European Research Institute (ERRI) points out in [4].

In order to analyze the behavior of the bridge when the ballasted track dynamic model is also taken into account, we use three different track models already proposed by other authors [4,5,6]. These models include vertical spring, dampers and masses which are interposed between the vehicle and the structure. For the vehicle a simplified 2-DOF model is used.

2   Dynamic models of the railway ballasted track

2.1   General

The railway ballasted track model is made of several elements which represent the rails, the sleepers, the connections between rails and sleepers, and the ballast. The rails are an important component in the track structure, since they transfer the wheel loads and distribute them over the sleepers and supports, guide the wheels in the lateral direction, provide a smooth running surface and distribute acceleration and braking forces over the supports. In Europe the typical rail used in the high speeds lines is the flat-bottom rail, UIC60.

The connections rail-sleeper are materialized by fastenings and rail pads. This system provides the transfer of the rail forces to the sleepers, damps the vibrations and impacts caused by the moving traffic and retains the track gauge and rail inclination within certain tolerances.

The sleepers are elements positioned just below the rails usually made of timber or concrete. They provide support for the rail, sustain rail forces and transfer them as uniformly as possible to the ballast. They preserve track gauge and rail inclination and provide adequate electrical insulation between both rails. The sleepers must be resistant against mechanical and weathering influences over a long period.

Finally, the ballast bed consists of a layer of a coarse-sized, non cohesive, granular material. Traditionally angular, crushed, hard stones and rocks have been considered good ballast materials. The interlocking of ballast grains and their confined condition inside the ballast bed permit the load distributing function and is damping. They also provide the lateral and longitudinal support of the track, as well as the draining effect. The thickness of the ballast bed should allow the sub grade to be loaded as uniformly as possible. The usual depth for the ballast is about 0.3 meters measured from the underside of the sleeper.

In the early studies, the models of the ballasted track were developed in order to investigate the train/track interaction problem. A review of these studies is presented in [7]. In the 1900's Timoshenko published papers on the strength of rails; later on, Inglis, was active in this issue. In [8] and [9] are presented an overview of existing tracks models in the field of train/track interaction. The main purpose of these studies in the time domain was to evaluate the deflections of the track and the vertical displacements of the

vehicles, while the contact force wheel/rail is evaluated in the calculations. Complete models of the vehicles and the effects of the wheel and rail irregularities are also investigated.

A large variety of ballasted track models has been investigated, from simple 2D model, where a single rail is modeled as an infinite Bernoulli-Euler or Timoshenko beam resting on supports defined by springs, dampers and point masses, to more complex 3D models, where both rails are taken into account and bending and shear deformation of the sleepers are included. In these models, the ballast bed is included through vertical spring and damper elements. Some of these models consider the mass of the ballast as a point mass located below each sleeper and its value is taken relative to the amount of stiffness and damping. Furthermore, shear springs and dampers may interconnect these masses [10]. The values for the mechanical properties of the track components, such as mass, inertia and elasticity, are mentioned as an essential input for dynamic track behavior and, of course, for the study of the interaction between train and track.

Since the investigation focus on the comparison of the numerical results with the dynamic response obtained from field measurements regarding the influence of the ballast track on the vertical vibrations of the railway bridges, only the 2D tracks models were consider, neglecting unimportant torsion effects.

2.2   Models and parameters

Three different models of ballasted tracks are presented in Figures 1, 2 and 3 from [5], [6], and [4]. In Model I the rails are considered as infinite long beams with in-plane and out-of-plane flexural stiffness as well as axial stiffness. The linear springs and dampers on the vertical and longitudinal directions represent the ballast. These three models are included in the finite element model of the bridge, which is acted by moving loads or vehicle models representing real trains. The model parameters remain constant along the track, despite some deviations due to construction and maintenance works. In the other two models, Model II and Model III, the connections between rail and sleeper are included as linear springs and viscous dampers acting in parallel. Their elastic and damping properties are mainly determined by the properties of the material and the manufacturing processes. The sleepers are included as rigid bodies with point mass. The ballast bed is included as discrete linear springs and viscous dampers. In Model III the mass of the ballast is included as point mass instead of distributed mass, and springs and dampers are used to simulate the connection between bridge and ballast [4]. The values of the mechanical properties for each model are included in Table 1 to Table 3.
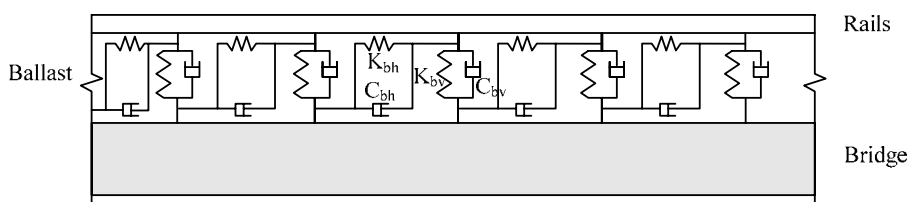

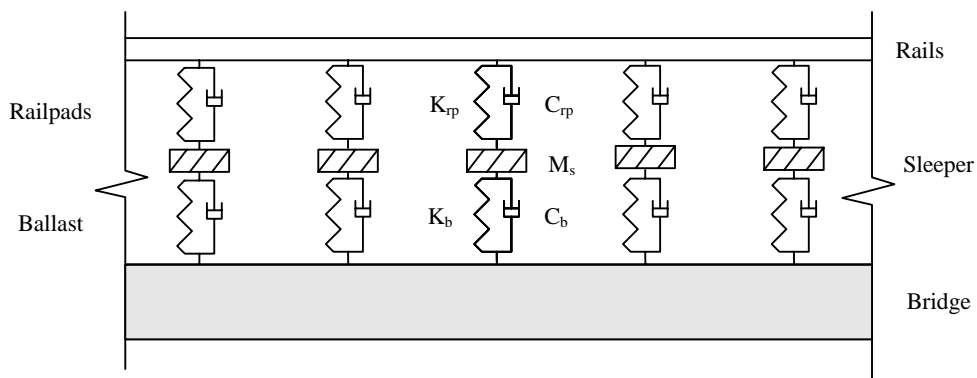**Figure 1: Ballasted track Model I [5].**
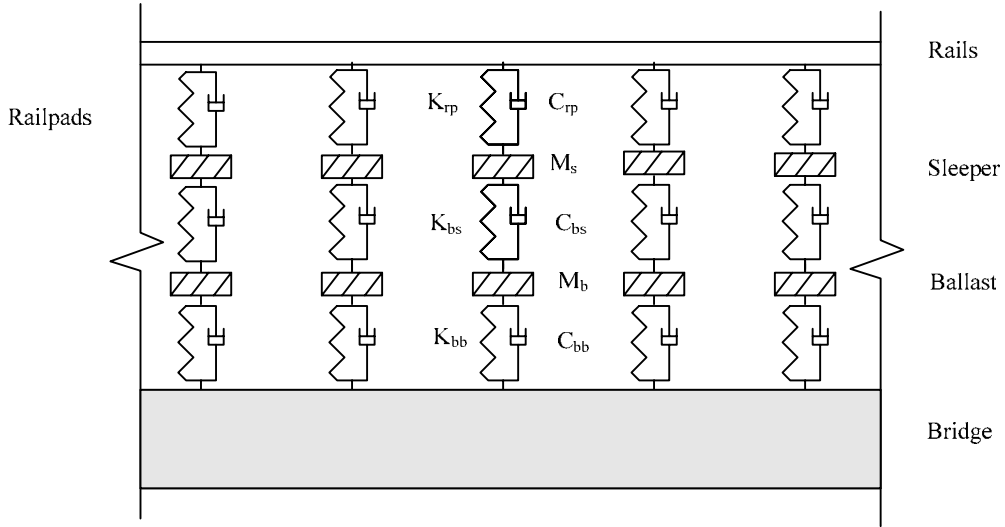

Figure 2: Ballasted track Model II [6].

Figure 3: Ballasted track Model III [4].

Parameters of the track model I	Notation	Value	Units
Rail UIC60			
Young Modulus	E_r	210E+09	N/m^2
Density	ρ_r	7850	kg/m^3
Flexural moment of inertia	I_r	3055E-08	m^4
Sectional area	A_r	76.9E-04	m^2
Ballast			
Per unit of length Vertical stiffness	K_{bv}	104E+03	N/m
Per unit of length Vertical damping	C_{bv}	50E+03	N.s/m
Per unit of length Horizontal stiffness	K_{bh}	104E+03	N/m
Per unit of length Horizontal damping	C_{bh}	50E+03	N.s/m

Table 1: Properties of track Model I [5].

Parameters of the track model II	Notation	Value	Units
Rail UIC60			
Young Modulus	E_r	210E+09	N/m^2
Density	ρ_r	7850	kg/m^3
Flexural moment of inertia	I_r	3055E-08	m^4
Sectional area	A_r	76.9E-04	m^2
Connection rail/sleeper			
Vertical stiffness	K_{rp}	300E+06	N/m
Vertical damping	C_{rp}	80E+03	N.s/m
Sleeper			
Mass	M_s	300	kg
Length between sleepers	d_s	0.60	m
Ballast			
Vertical stiffness	K_b	120E+06	N/m
Vertical damping	C_b	114E+03	N.s/m

Table 2: Properties of track Model II [6].

Parameters of the track model III	Notation	Value	Units
Rail UIC60			
Young Modulus	E_r	210E+09	N/m^2
Density	ρ_r	7850	kg/m^3
Flexural moment of inertia	I_r	3055E-08	m^4
Sectional area	A_r	76.9E-04	m^2
Connection rail/sleeper			
Vertical stiffness	K_{rp}	500E+06	N/m
Vertical damping	C_{rp}	200E+03	N.s/m
Sleeper			
Mass	M_s	290	kg
Length between sleepers	d_s	0.60	m
Ballast			
Vertical stiffness ballast/sleeper	K_{bs}	538E+06	N/m
Vertical damping ballast/sleeper	C_{bs}	120E+03	N.s/m
Mass	M_b	412	kg
Vertical stiffness bridge/ballast	K_{bb}	1000E+06	N/m
Vertical damping bridge/ballast	C_{bb}	50E+03	N.s/m

Table 3: Properties of track Model III [4].

3 Considerations about the dynamic analysis

3.1.1 Modeling the train action on the structure

The present study considers two methodologies for the dynamic load evaluation: *i)*the moving loads model (Figure 4b) and *ii)*the interaction model (Figure 4c). In this figure, the upper and lower beam elements model the rail and the bridge deck, respectively, which are interconnected by the ballasted track model. Since the train considered corresponds to a conventional train, each bogie has two axles represented by two forces in the moving loads model. For the interaction model, a 2-DOF vehicle is considered. The vehicle model is defined by a sprung mass model with one node associated at each of the two concentrated masses (Figure 4c). The stiffness and damping of the suspension, denoted by k_v and C_v, respectively, correspond to the primary suspension of the train vehicle. The mass of the wheel is denoted by M_w and the mass lumped from the car body by M_v, which is assumed to be equal to a quarter of the mass of the car body and bogie mass. D_k represent the characteristic length of the carriage. The train crosses the bridge at a constant speed v_i.

3.1.2 Solution of the equilibrium equations

The software ADINA is used to compute the dynamic response, which can be obtained by direct time integration of the system of the dynamic equilibrium differential equations using the Newmark method or the Wilson-θ method. For these algorithms the time step may be selected independently of numerical stability considerations.

According to [12], for the determination of the maximum deck acceleration, the frequencies in the dynamic analysis should be considered up to a maximum of: i) 30 Hz ; ii)1.5 times the frequency of the first mode shape of the structural element being considered, including at least the first three mode shapes. For that reason, in railway dynamic problems, in addition to being unconditionally stable, only low mode

response is of interest. Therefore, it is desirable that the integration scheme possesses the capability of numerical dissipation to damp out the spurious participation of the higher modes. Both the Wilson θ method and the Newmark method, restricted to parameter values of $\gamma > 1/2$ and $\beta \geq 0.25 \times (\gamma + 1/2)^2$, where the amount of dissipation, for a fixed time step Δt, is increased by increasing γ, possess this advantage.

The dissipative properties of the Newmark algorithm are considered to be less efficient then those of the Wilson-θ method, since the lower modes are strongly affected. The Wilson-θ method, with $\theta = 1.4$, is highly dissipative at the highest modes, unconditionally stable and accurate when $\Delta t/T_n \leq 0.01$, where T_n is the lowest vibration period to take into account in the structural response analysis, [13]. Considering this conclusions, the dynamic response of the bridge was computed with the Wilson-θ method.

Concerning the damping, the Rayleigh matrix was used, that is, $C = \alpha \cdot M + \beta \cdot K$, with constants α and β compatible with the measurements at the first and third eigenfrequencies of the bridge.
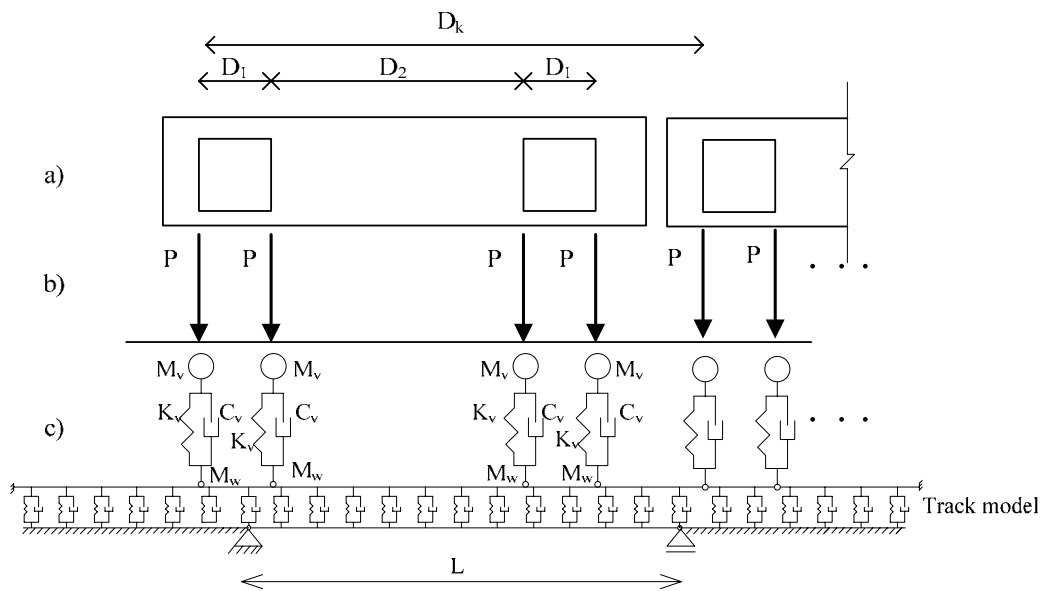

Figure 4: The train: a) Plan view; b) Moving load model; c) Vehicle interaction model.

4   Analysis of a railway bridge

4.1   Description of the bridge

The bridge deck is a prestressed concrete 23.5 meters length simply supported slab with a slightly variable depth and a mass per unit length of about 21 Ton/m. The geometrical characteristics are shown in Figure 5.

The bearing supports, two at each end, as shown in Figure 6, are made of steel pots filled with elastomeer. There is no continuity of the slab over the supports to the abutments, except the one materialized by the ballast track, see Figure 7.
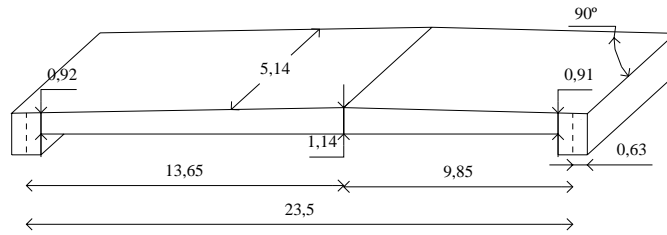
Figure 5: Structural layout of the bridge.

Figure 6: Bearings supports.

Figure 7: View of the railway track over the bridge.

4.2 Numerical model

A numerical model was developed with the purpose of comparing the computed dynamic characteristics, modes, frequencies and damping, with those obtained from measurements. Although the structural layout used for the bridge design corresponds to a simply supported bridge, the measurements showed that the bridge behaves as a simply supported slab with some flexural stiffness at the supports. Therefore, the numerical model includes a spring of stiffness K_{rail} over the supports at the level of the rails (see Fig. 8) simulating the continuity of the rail track, and a spring of stiffness K_s at the level of the bearing supports, simulating the slip resistance.

Since, in this model, only the rail axial stiffness can be estimated from the rail characteristics, and no information can be obtained for the slip resistance of the supports, this stiffness K_s was considered equal to K_{rail}, which can be estimated by

$$K_{rail} = \frac{E_r A_r}{4d_s} \tag{1}$$

where E_r, A_r and d_s correspond, respectively, to the Youngs modulus, section area and length between sleepers, with values given in Table 1. Using these values, $K_s = K_{rail} = 670 \text{ MN}/\text{m}$.
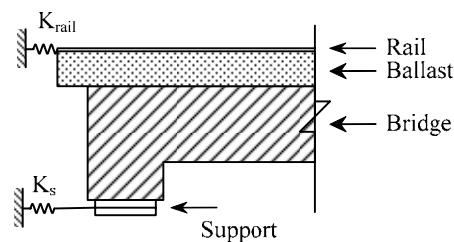

Figure 8: Model of the support.

The concrete properties used in the computations are the specific weight, $\gamma_c = 25 \text{ kN/m}^2$ and the Youngs modulus $E_c = 54.6 \text{ GPa}$. For the ballast, the specific weight $\gamma_b = 20 \text{ kN/m}^2$ was considered. The values of the first four natural frequencies of the model, $F_1 = 4.40 \text{ Hz}$, $F_2 = 13.70 \text{ Hz}$, $F_3 = 16.00 \text{ Hz}$ and $F_4 = 28.75 \text{ Hz}$, are in good agreement with those from free vibration measurements, immediately after the train leaves the bridge.

However, if larger amplitudes are considered, as it happens when the bridge is loaded during train passages, the agreement is poorer due to the non-linear behavior of the support flexural stiffness included in the model [1]. Moreover, if the mass of the vehicles on the bridge is also taken into account, the first frequency will further decrease[1].

In order to adjust the model parameters to the measured response during forced vibrations, and taking into account the conclusions above, a lower modulus of elasticity was considered, $E_c = 40 \text{ GPa}$, corresponding to the first natural frequency of the bridge of approximately 4.0 Hz.

4.3   Dynamic response due to train passages using the moving loads model

The main purpose is to analyze the vibration of the bridge with and without consideration of the track models and compare it with the response obtained from the measurements. The results are obtained separately for the moving loads model and for the model including interaction between train and bridge. Two different trains are considered for the comparison: *(i)* a single locomotive type 1116 and *(ii)* an ICE train with seven vehicles – power car type 1044 and six carriages type 2094.

4.3.1   Analysis of the locomotive type 1116

The locomotive data for the moving forces model, according the Figure 4, is $D_1 = 3.0 \text{ m}$, $D_2 = 6.90 \text{ m}$ and the force corresponding to each axle is $F = 210.925 \text{ kN}$. The speed of the locomotive, 130 km/h, was measured independently with a speedometer when it passed over the bridge.
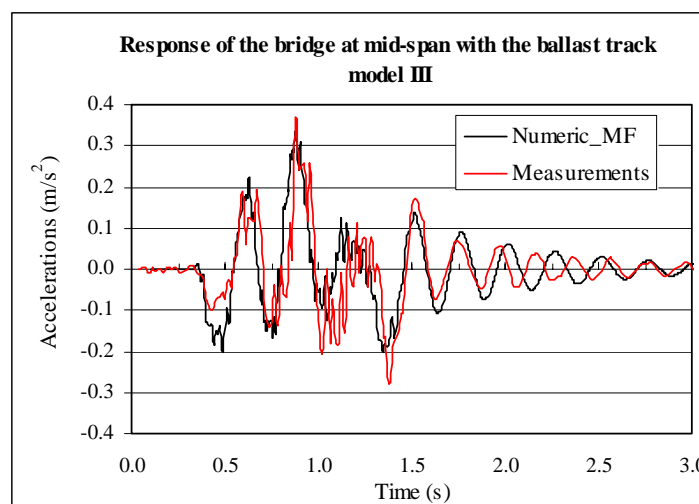

Figure 9: Comparison between numeric and measured response of the bridge,

with the track Model III, during the passage of the locomotive.

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[1] When the mass of the locomotive type 1116 is included as a uniformly distributed structural mass, the decrease of the eigenfrequency is about 7%.

Since the results calculated for the bridge considering the moving load model and each of the track models are similar to those computed without the track model, only the comparison between the measured and the computed acceleration at the mid-span of the bridge with the ballast track model III are represented in Figure 9. Analyzing the evolution of the time histories, it is obvious that, during the passage of the locomotive, the numeric response fit the measurements quite well, but, after that, for free vibrations only the amplitudes are similar.

Indeed, in the calculations the first eigenfrequency of the structure, 4.0 Hz, remains constant for the entire duration of the response. The measurements, however, reveal that for the initial time, up to about 1.4 s, when the locomotive is on the bridge, the first frequency match quite well the 4.0 Hz but, after the passage of the vehicle, an increase on the value of the frequency is observed, confirming that there is some influence of the vehicle mass and of the non-linear increase of the stiffness for lower amplitudes of vibration. During the free vibrations the measured first frequency of the bridge increases to the above referred value of 4.40 Hz.

Concerning the damping, the value obtained from the measurements, $\xi = 0.05$, is used and corresponds to the amplitude variation that can be observed in the free vibration part of the time history.

In the time domain, the values calculated for the bridge considering the track models are similar to those obtained without the track model. Using a frequency domain representation, however, it can be seen that the use of the ballast track model suppresses the contribution of the response frequency components in the range 20-30 Hz, acting as a low pass filter (Figure 10). From this point of view the track model III is the most efficient in filtering the higher frequencies.
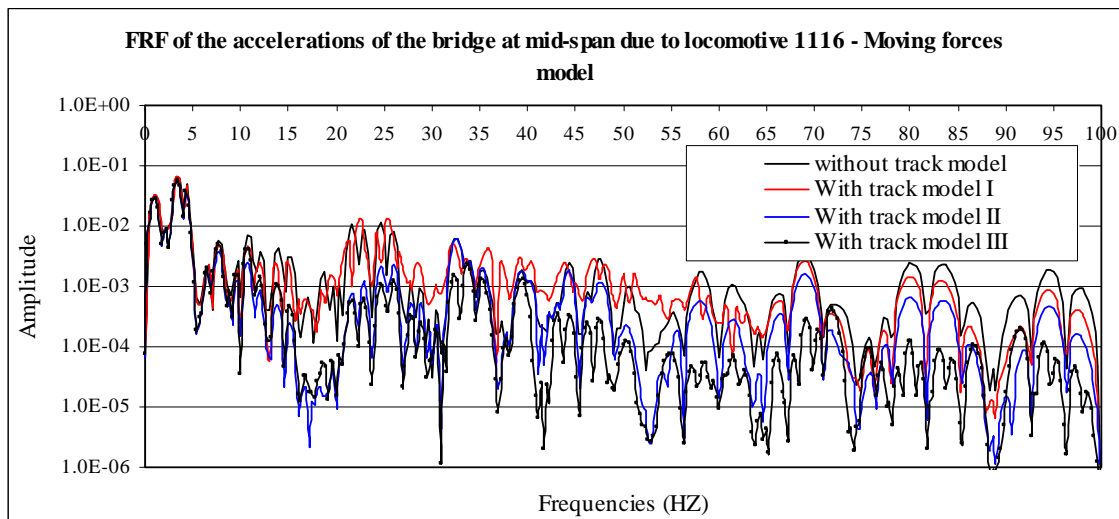

Figure 10: Frequency domain representation of the response acceleration of the bridge mid-span section taking for all the three ballast track models.

Unlike the case of the ICE analyzed hereafter, the values of the stiffness and damping of the primary suspension of the locomotive are not known. Therefore no comparison between the numerical results, using interaction, and the measurements is presented. Indeed, a preliminary study using assumed values for those parameters was done, which showed that the values for the primary suspension decisively influence the results.

4.3.2   Analysis of the ICE train

The available measurements include the acceleration response to the passage of the ICE train with seven vehicles, at a speed of 140 km/h. According to Figure 4 the data of intermediate carriage of this train can be briefly described with the following parameters: $D_1 = 2.50$ m, $D_2 = 15.18$ m and $D_k = 26.40$ m.

Concerning the forces applied by axle, the power car is characterized by $F = 204.05\ \text{kN}$ and the intermediates carriages by $F = 127.53\ \text{kN}$.
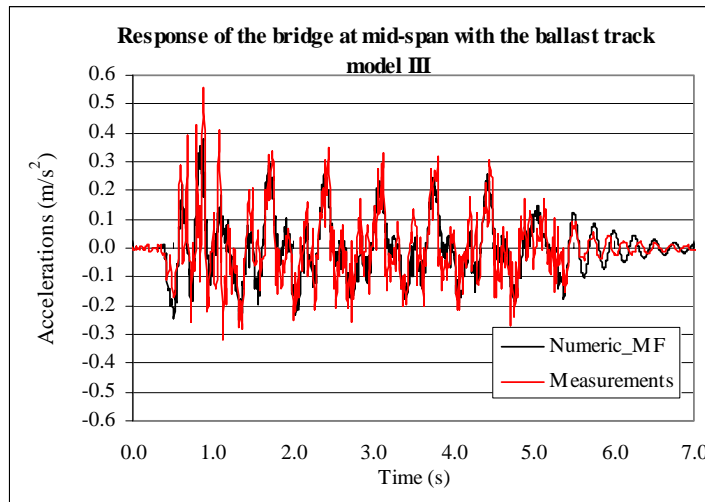

Figure 11: Comparison between numeric and measured response of the bridge, with a track model III, subject to the passage of the ICE549 train at a speed of 140km/h.

The Figure 11 compares the measured and the computed accelerations at the mid-span of the bridge for the cases where the ballast track model III was used. The represented times histories show the goodness of fit between the numeric response and the measurements. The damping $\xi = 0.05$, for the first eigenmode, seems to fit quite well.

Since the results in time domain obtained from the model of the bridge with and without any track model are similar only the result with the track model III is presented. However, if the comparison is made in the frequency domain, it is concluded once again that the ballast track Models I, II and III suppress the contribution of the higher frequencies, this fact is emphasized when analyzing the frequency response accelerations at the mid-span of the bridge taking into account the train-track-bridge interaction model with different systems (see Figure 12).
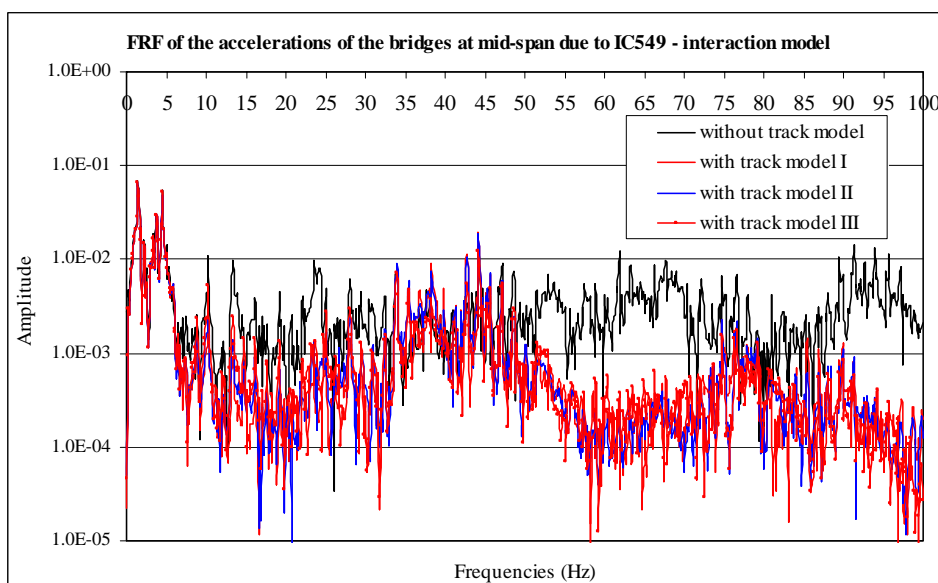

Figure 12: Comparison between the FRF of the accelerations of the bridge, with all the ballast track models for the ICE549 train under a speed of 140km/h.

4.4 Interaction model *versus* moving loads model

4.4.1 Description of the dynamic analysis

In order to investigate the influence of the interaction on the dynamic behavior, the same bridge described in Figure 5 was considered. The static deflection at the mid span of this bridge due to the Load Model 71[2] [11] equals $\delta = 23.04\text{mm}$ and the length/deflection ratio is $L/\delta = 1020$. Concerning the damping, the Rayleigh matrix was used, $C = \alpha \cdot M + \beta \cdot K$, with constants $\alpha = 0.235$ and $\beta = 3.640\text{E} - 04$, which correspond to a damping ratio, for the first frequency of the bridge of about 1%. It was decided to use this value instead of the measured one since it is recommended in eurocode 1 [11] for prestressed concrete railway bridges with span higher than 20 m.

When a dynamic analysis is made, a series of speeds up to the maximum design speed must be considered. The maximum design speed is 1,2 times the maximum live speed at the site. Calculations were made for speeds in the interval [140, 300]km/h with a step of about 5km/h. A smaller step was adopted around the resonant speeds.

4.4.2 The High Speed train ICE2

The high speed train ICE2 consists of a total of 14 carriages including two power cars located in the front and in the rear end of the train. According to Figure 4, the data for the carriages are $D_1 = 3.00$ m, $D_2 = 8.46$ m, $D_k = 26.40$ m and $F = 112.0$ kN. For the power cars $F = 195.0$ kN.

The passage of successive loads with uniform spacing, which in this case is 26.40 m, can excite the structure in resonance. This occurs when critical speed is reached, which can be calculated as follows:

$$v_{cri.} = \frac{D_k}{i} \times n_0, \ i = 1, 2, 3, \dots, n \tag{2}$$

For $i = 1$, $D_k = 26.40$ m and $n_0 = 2.70$ Hz the critical speed is ≈ 257 km/h.

4.4.3 Results for the moving loads model

Considering the moving loads model and the bridge model without and with each of the three track ballast models, the displacements and accelerations were computed and the maximum values plotted against the speed. As we can see in Figure 13 and Figure 14, the responses of the bridge are very similar for all the modeling situations.

The effect of the ballast track models can be shown in Figure 15, for the speed of 140 km/h, where the representation of the accelerations at the mid span of the bridge is made in the frequency domain. The contribution of the frequencies in the range 10-30 Hz is suppressed when a track model is used. From this point of view and for this case study, all the track models have the same behavior in filtering the higher frequencies.

The Figure 15, also allows the conclusion that the use of the Wilson-θ method for the time integration leads to a good result in damping out the spurious participation of the higher modes, that is, the contribution of the frequency components above 30 Hz is very low [12].

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[2] The Load Model 71 is a static load pattern proposed by the Eurocode 1 [11] for the design of railway bridges.
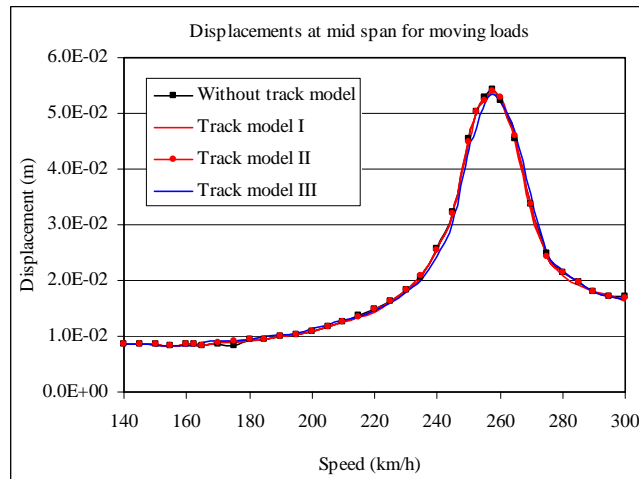
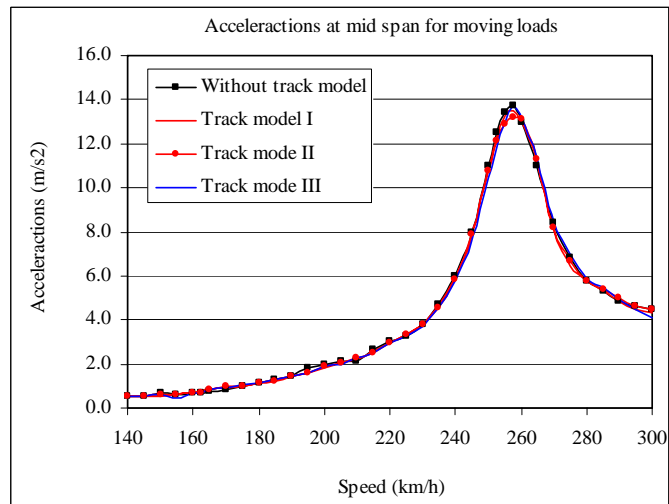**Figure 13: Maximum displacements at mid span considering the moving force model.**

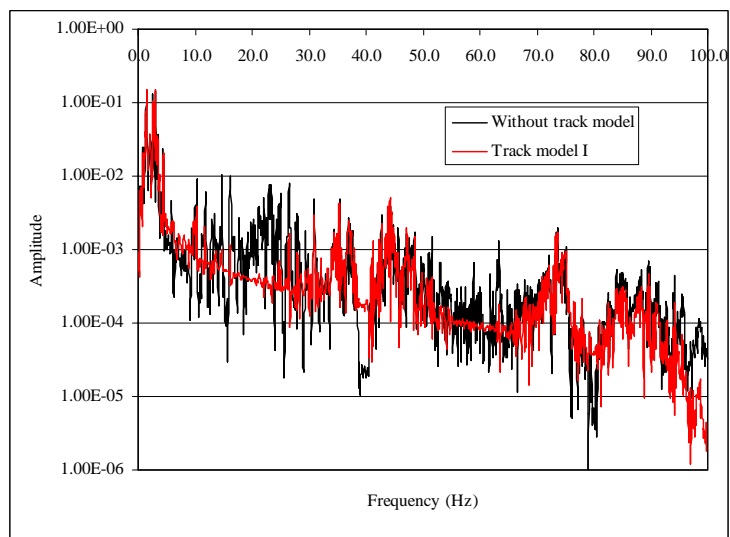**Figure 14: Maximum accelerations at mid span considering the moving force model.**

Figure 15: Comparison of the accelerations of the bridge with a track model I and without track model in frequency domain during the passage of the ICE2 train at a speed of 140km/h, for the moving load model.

4.4.4   Results for the interaction model

Considering now the results obtained when the vehicle/bridge or vehicle/track/bridge interaction is present, the maximum values of the displacement and acceleration response are represented in Figure 16 and Figure 17, respectively. The resonance speed is reached at about the same value as before, but the maximum values of displacement and acceleration obtained with these models are much lower than those obtained with the moving loads model. The maximum displacement is $3.60 \, cm$, obtained for the vehicle/track/bridge interaction models II and III. The maximum displacement obtained with the model without ballast track is 3,4 cm.

Considering the response accelerations, all the models furnish identical results. Out of the resonance situation the vehicle/bridge model, without track, shows modest higher values than the other models. One possible explanation for this is the possible enhancement of energy dissipation mechanisms at low amplitudes of vibration due to the longitudinal distribution effects of the vehicle loads through the ballast layer.

Since the results for the three track models are quite similar, only the response in frequency domain for the model III is represented in Figure 18 and compared with the modeling situation without ballast. The results for this situation show a greater contribution of high frequency components, in accordance with the conclusions obtained when the moving force model is used (compare Figures 15 and 18).
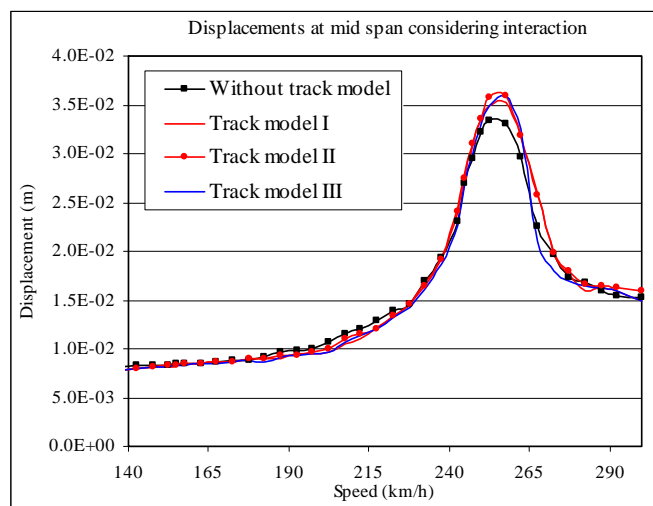

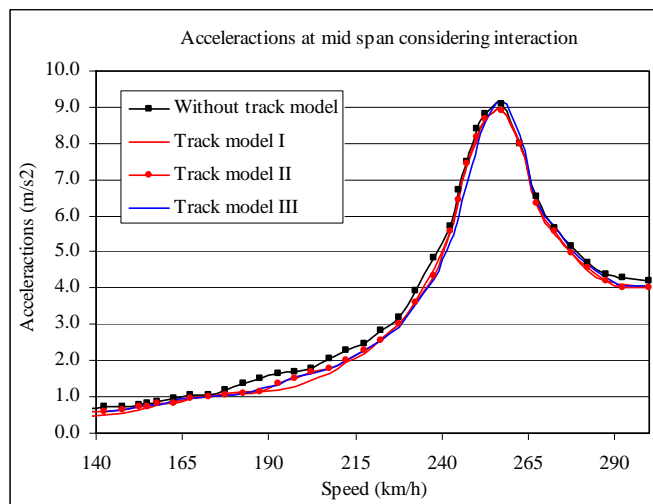**Figure 16: Maximum displacements at mid span considering the interaction model.**

Figure 17: Maximum accelerations at mid span considering the interaction model.
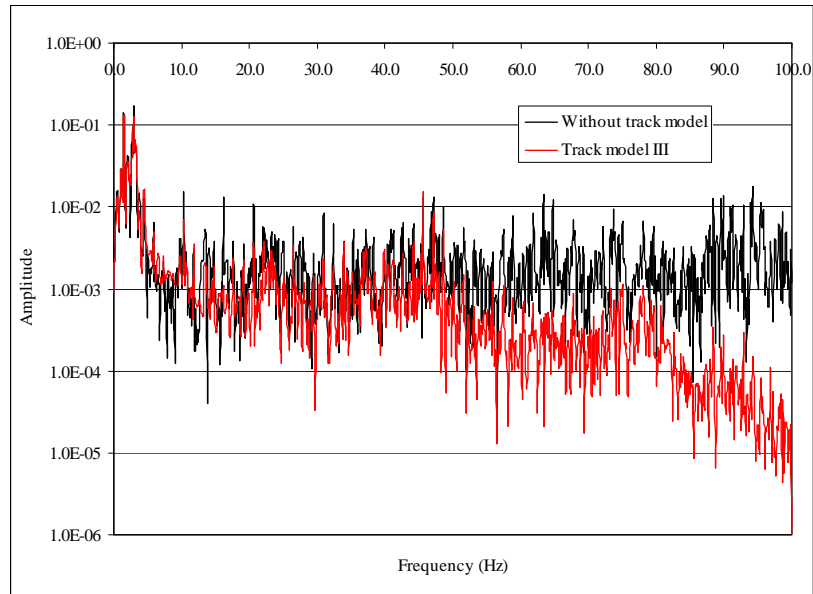
Figure 18: Comparison of the accelerations of the bridge with a track model III and without track model in frequency domain during the passage of the ICE2 train at a speed of 140km/h, for the interaction model.

When comparing the results obtained from the two different loading models, the maximum response accelerations and displacements for the interaction model are about 33% lower than the equivalent results for the moving loads model.

5   CONCLUSIONS

The main purposes of this investigation were: at first, the calibration of the structural model of one simply supported medium span concrete bridge, for which former work was done [1,2,3] and measurements of the acceleration response under real traffic were available; second, the study of the same bridge in a design situation, using eurocode 1[11] recommendations and the loads corresponding to the high speed train ICE2. For all studied situations three types of ballasted track models were considered and two different methodologies for the load modeling were used: the vehicle/track/bridge interaction methodology and the moving loads methodology.

The performed calibration confirmed that the non-linear effects at the supports together with the variation of the mass during the passage of the trains were responsible for the variation of, at least, the first eigenfrequency. When the bridge is being loaded during the train passage, the first frequency can be considered to be about 4,0 Hz and when the very low amplitudes of vibration of the unloaded structure are used for the modal identification the first eingenfrequency increases about 30% [2]. This conclusion was of major importance for the comparison of the measured and the computed response during the train passage.

According to the results obtained for the acceleration in the frequency domain, it can be concluded that the use of the Wilson-θ method in railway problems shows to be suitable in filtering the high frequency components. The results reveal a good numerical dissipation of the spurious participation of the higher modes.

The response of the system track/bridge when subject to the moving loads model shows that the different track models do not influence the maximum displacements and accelerations. The results obtained for the response accelerations in the frequency domain show that those models act as a filter in the high frequency components.

The response of the system vehicle/track/bridge interaction model shows that the track models do not influence the maximum accelerations at the resonance speed. However, for out of resonance situations, the system vehicle/track/bridge interaction shows lower values, than the system vehicle/bridge interaction, that it could be due it to the longitudinal distribution of the effects of the vehicle through the ballast track as an energy dissipation of the bridge under the action of the moving train at lower speeds or lower amplitudes of vibrations. Analyzing the response of the system vehicle/track/bridge in frequency domain in terms of acceleration shows that system as more contribution of higher frequencies than the system track/ballast. But once again the ballast track model as a good behavior in filtering this higher frequencies.

Comparing the results obtained for the maximum displacements and accelerations at the mid span for the two different methodologies, interaction model and moving load model, it can be concluded that the use of the interaction model results in 33% lower displacements and accelerations. Therefore, the inclusion of the inertia effects of the moving vehicles contributes decisively to the reduction of the peak response.

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