# DESIGN ISSUES RELATED TO DYNAMIC EFFECTS FOR HIGH-SPEED RAILWAY BRIDGES IN SPAIN

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## ABSTRACT

High-speed railway lines are an important factor for modernisation of transport in Europe, specially in countries such as Spain with a historic deficit in infrastructure.

This work will also discuss some issues related to dynamic effects in railway bridges. Firstly we review and comment analysis methods from a practical point of view. Following, issues related to train types are discussed. Finally, some considerations related to the structural type of the bridge are studied.

## 1. INTRODUCTION

High speed railway lines are currently the main element in new transport infrastructure, contributing to improved opportunities for travel and commerce in Europe. They already provide an efficient transport link between some of the main Spanish cities, and will soon link Spain and Portugal together and with the rest of Europe.

One of the main design issues associated specifically to the design of bridges and structures in railway lines are the dynamic effects due to moving loads from train traffic. This has been studied since the early days of railways [1], as well as in more recent works (e.g. Fryba [2] and Alarcón [3]).

The design codes existing up to now [5,6,7] for design of railway bridges consider the dynamic response through an *impact factor*, which represents the increase in the dynamic response with respect to the static one for a *single moving load*.

However, high speed railway lines pose dynamic problems of higher order, due to the possibility of resonance from high speed traffic at speeds above 200 km/h, considering the typical distances between axles in railway coaches and the main eigenfrequencies of bridges. Resonance occurs when the excitation frequency coincides with that of the fundamental vibration mode of the bridge. This may be quantified through the so called *wavelength* of excitation, $\lambda = v/f_0$, where $f_0$ is the first natural frequency of deck vibration and $v$ the train speed. Resonance occurs when the characteristic length $D_k$ of separation between axles coincides with a multiple of the said wavelength:

$$\lambda = \frac{D_k}{i}, \quad i = 1,2,3,4. \quad \Rightarrow \quad \text{resonance} \tag{1}$$

Within Europe a joint effort for research and study of dynamic effects in high speed lines has been carried out within the European Railway Research institute (ERRI) by subcommittee D214 [13]. These and other findings have been included in the recently drafted engineering codes [8], [10] and [9].

In this paper we consider some issues related to our experience in Spain for the dynamic effects on railway bridges. Firstly (section 2) we discuss the various analysis methods for calculation and discuss the practical issues of their application. Following (section 3) we consider issues related to train types influencing the dynamic response. Finally (section 4), some considerations are made related to structural type of the bridge.

## 2. ISSUES RELATED TO ANALYSIS METHODS

### 2.1 Impact factor $\Phi$

The basic method followed up to now in the existing engineering codes for railway bridges [5,6,7] has been that of the impact factor, generally represented as $\Phi$. As has been discussed previously in section 1, such coefficient represents the dynamic effect of (single) moving loads, but does not include resonant dynamic effects.

The dynamic increment $\varphi'$ for a single moving load at speed $v$ on an ideal bridge (i.e. without track or wheel irregularities) is evaluated in [7] by the following expression:

$$\varphi' = \frac{K}{1 - K + K^4}; \quad K = \frac{\lambda}{2L_\Phi} \tag{2}$$

where $L_\Phi$ is the equivalent span of the element under study and $\lambda = v/f_0$ the wavelength of excitation. According to Equation (2) the value of the dynamic increment reaches a maximum value of $\varphi'_{\max} = 1.32$, for $K$=1.76. The final impact factor takes into account additionally the effect of irregularities through an additional term ( $\varphi''$ ):

$$\Phi \geq \max\left(1 + \varphi' + \varphi''\right) \tag{3}$$

The impact factor so defined is applied to the effects obtained for the static calculation with the nominal train type (LM71):

$$\Phi \cdot E_{\text{sta,LM 71}} \geq E_{\text{dyn,real}} \tag{4}$$

We remark that the impact factor Φ is applied not to the real trains, but to the effects of the LM71 load model, which is meant as an envelope of passenger, freight traffic and other special trains, generally much heavier than passenger trains. Finally, the applicability of impact factor $\Phi$ is subject to some restrictions, involving bounds for $f_0$ as well as a maximum train speed of 200 km/h [6].

### 2.2 Simplified models based on dynamic train signature

The so-called *dynamic train signature* models develop the response as a combination of harmonic series, and establish an upper bound of this sum, avoiding a direct dynamic analysis by time integration. In counterpart their application is limited to *simply supported bridges*, which can be represented dynamically by means of a single harmonic vibration mode. Their basic description may be found in [13].

All these methods furnish an analytical evaluation of an upper bound for the dynamic response of a given bridge, as a product of three terms: a constant term, a *dynamic influence line* of the bridge, and a *dynamic signature* of the train. Let us take as an example the LIR method [13] for evaluating the maximum acceleration. This procedure is based on the analysis of the residual free vibrations after each individual single load crosses a simply supported bridge. The acceleration $\Gamma$ at the centre of the span is given by:

$$\Gamma = C_{\text{acel}} \cdot A(K) \cdot G(\lambda), \tag{5}$$

$$A(K) = \frac{K}{1-K^2}\sqrt{e^{-2\zeta\frac{\pi}{K}} + 1 + 2\cos\left(\frac{\pi}{K}\right)e^{-\zeta\frac{\pi}{K}}} \tag{6}$$

$$G(\lambda) = \max_{i=1}^{N}\sqrt{\left[\sum_{x_1}^{x_i} F_i \cos(2\pi\delta_i)e^{-2\pi\zeta\delta_i}\right]^2 + \left[\sum_{x_1}^{x_i} F_i \sin(2\pi\delta_i)e^{-2\pi\zeta\delta_i}\right]^2} \tag{7}$$

In these expressions $C_{\text{acel}} = 1/M$ is a constant (the inverse of the total mass of the bridge), $\zeta$ is the damping rate, $x_i$ are the distances of each one of the $N$ load axes $F_i$ to the first axis of the train, and $\delta_i = (x_i - x_1)/\lambda$.

The term $G(\lambda)$ (Equation (7)) is the *dynamic signature* referred to above. It depends only on the distribution of the train axles and the damping rates. Each train has its own dynamic signature, which is independent of the mechanical characteristics of the bridges. As an example, Figure 1 represents the dynamic signature of train ICE2, for different values of damping.

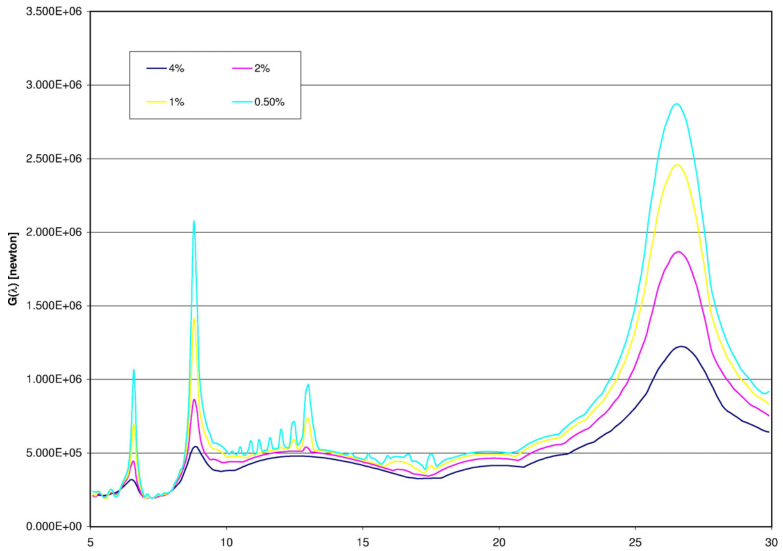



Figure 1: Dynamic signature (LIR) of ICE2 train for different damping rates. The peak response is attained for excitation wavelengths near the coach length

The term $A(K)$ (Equation (6)) defines a function of $K$ (itself dependent on speed $v$), called the bridge *dynamic influence line*. It depends solely on the span $L$, the first natural frequency ($f_0$) and damping ($\zeta$).

Neither $C_{acel}$ nor $A(K)$ depend on the characteristics of the train. Separating the contributions from the bridge and those from the train ($G(\lambda)$, dynamic signature), it is possible to determine easily the critical parameters of span and wavelength for which the dynamic response of the bridge is maximum.

As has been said before, these dynamic signature methods have been developed in principle for simply supported, isostatic bridges. However, some studies have been carried out which in some cases extend their applicability to certain classes of redundant structures. For instance, Liberatore [15] has developed dynamic signature methods to establish the *modal agressivity* of continuous decks with 2 spans.

### 2.3 Dynamic analysis with moving loads

This general class of models are based on time integration of the dynamic equations for the structure, subject to a series of moving loads of constant values, representative of each axle of a given train. The structure is first discretised using a finite element method or a more particular bar technique, arriving to a vector $\boldsymbol{d}$ of $N$ (unknown) nodal displacements. The model for the structure may be analysed either through an direct integration of the complete system,

$$\boldsymbol{M} \cdot \ddot{\boldsymbol{d}} + \boldsymbol{C} \cdot \dot{\boldsymbol{d}} + \boldsymbol{K} \cdot \boldsymbol{d} = \boldsymbol{f}(t), \quad (8)$$

where $\boldsymbol{M}$ is the mass matrix, $\boldsymbol{C}$ the damping matrix, $\boldsymbol{K}$ the stiffness matrix, and $\boldsymbol{f}(t)$ the external load vector. In the case of moving loads, this load vector must represent the load histories of each of the train axles ($F_k$, $k = 1,\ldots,n_{ax}$) at the particular speed considered (Figure 2).

A reduction of degrees of freedom may be performed through a modal analysis. Modal reduction reduces substantially the number of equations to integrate, and may be performed through an approximate numerical procedure to obtain the eigenmodes of vibration. This capability is provided by most commercial finite element programs. Alternatively the modal reduction may be achieved through an analytical (closed form) calculation, for certain cases of simple structures.

In references [4,11,16,18] more details are given regarding the numerical algorithms.

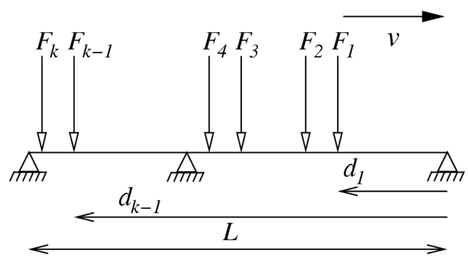



Figure 2: Load model for a train of moving loads

Except for particular cases of simple structures the above Equation (8) must be evaluated numerically by finite element methods. These provide an efficient method for calculation in arbitrary structures. Adequate procedures for pre-processing (definition of load histories for all individual axles) and post-processing are necessary for their practical use in engineering design work [18].

### 2.4 Dynamic analysis with vehicle-structure interaction

The consideration of the vertical motion of the vehicles with respect to the bridge deck allows for a more realistic representation of the dynamic overall behaviour. The train is no longer represented by moving loads of fixed value, but rather by point masses, bodies and springs which represent wheels, bogies and coaches. In some cases these models may have a non negligible influence on the dynamic response of the bridge.

A general model for a conventional coach on two bogies is shown in Figure 3a, including the stiffness and damping ($K_p$, $c_p$) of the primary suspension of each axle, the secondary suspension of bogies ($K_s$, $c_s$), the unsprung mass of wheels ($M_w$), the bogies ($M_b$, $J_b$), and the vehicle body ($M$, $J$). Similar models may be constructed for articulated or regular trains.

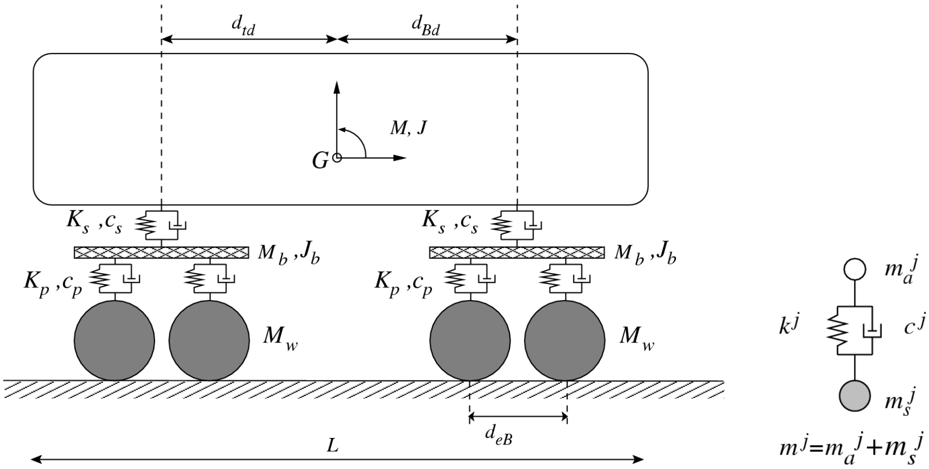



Figure 3: Vehicle–structure interaction models: (a) full interaction model; (b) simplified interaction model

The level of detail in the above model is not always necessary. Often simplified models may be employed in which for each axle only the primary suspension, equivalent unsprung and sprung masses are considered (Figure 3b). In this model each axis is independent from the rest, thus neglecting the coupling provided by the bogies and vehicle box, as well as the rocking motion of the vehicle box. Further details of these models are described in [11].

The consideration of vehicle structure interaction often reduces the dynamic response of the bridge, as is shown in [4,16] (see also Figure 9). This may be explained considering that part of the energy from the vibration will be transmitted from the bridge to the vehicles. In short span bridges (i.e. $L \leq 30$m) this may produce a reduction of up to 30% [4]. However, in longer spans or in continuous deck bridges the reduction will generally be small. As a consequence it is not generally necessary to perform dynamic analysis with interaction, except for particular cases.

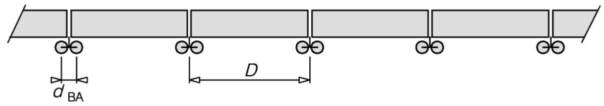
## 3. ISSUES RELATED TO TRAINS

As has been mentioned, the configuration of trains and axle spacing is one of the key factors defining the dynamic effects and the possibility of resonant response in bridges. With regard to distribution of axles and bogies three types may be identified in European high speed trains (Figure 4):

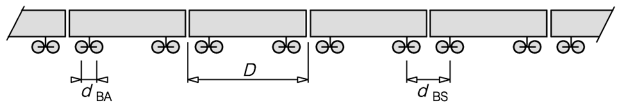
1. *Articulated trains:* each two coaches share one bogie between them. This type includes current European models Thalys, AVE and Eurostar.

2. *Conventional trains:* each coach has two bogies. This includes Ice2, Ice3, Etr-y, Virgin.

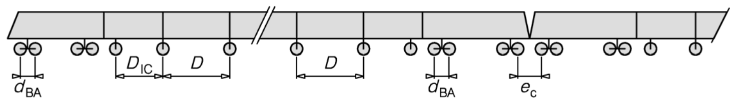
3. *Regular trains:* coaches are supported not on bogies but on single axles in the junction between each two of them. This is the case of TALGO.




1 – Articulated train




2 – Conventional train




3 – Regular train

Figure 4: Different types of high-speed trains, according to Eurocode 1 [10]

To ensure dynamic performance not only for the above trains but also for their possible variations and future developments through dynamic analysis ("brute force method") would be extremely costly as well as of doubtful efficiency. Small variations in the configuration of a given train may influence significantly the resonant peaks, making it extremely difficult to assure the fulfilment of the dynamic performance interoperability conditions.

The concept of *train signature* (Section 2.2) is very useful for the purpose of understanding the dynamic effects of each particular train, and for comparing the responses. This train signature is a characteristic of each particular train, independent of the bridge. Figure 6 shows the dynamic signature (DER method) obtained for the most common current European high-speed trains. The dynamic response of the bridge may then be obtained by multiplying the train signature by the so-called *dynamic influence line* of the bridge. In Figure 5 one can clearly see the variation in the response for the different classes of trains. The articulated trains show response peaks for wavelengths between 17 and 21 m. The conventional trains exhibit the highest response for wavelengths between 24 and 28 m. Finally, the regular train TALGO shows the peak for 13 m. These critical wavelengths are related in each case to the length of the wagons defining characteristic axle or bogie spacings. In effect, the shortest coach length is that of the TALGO, with axle spacings of 13.14 m.

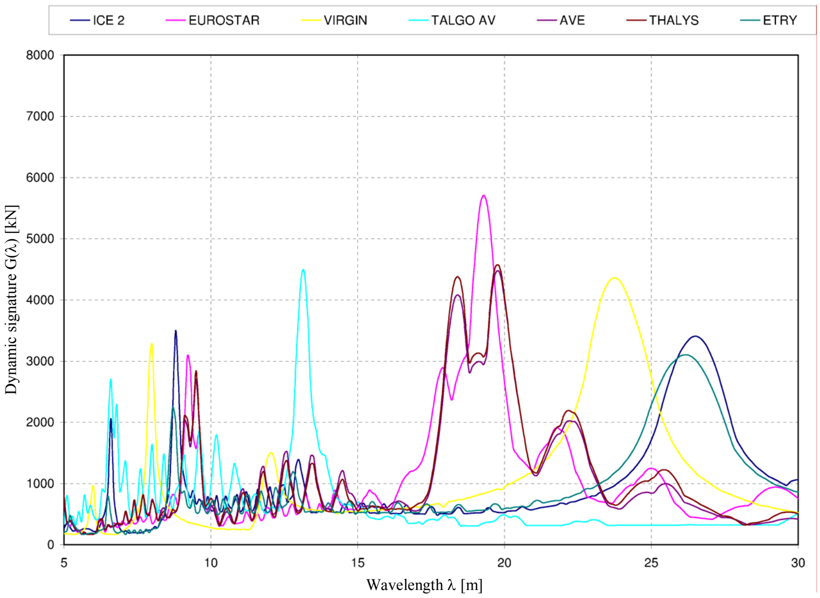



Figure 5: Dynamic signatures (zero damping) for European high-speed trains, including articulated trains (EUROSTAR, AVE, THALYS), conventional trains (ICE2, VIRGIN, ETRY) and regular trains (TALGO)

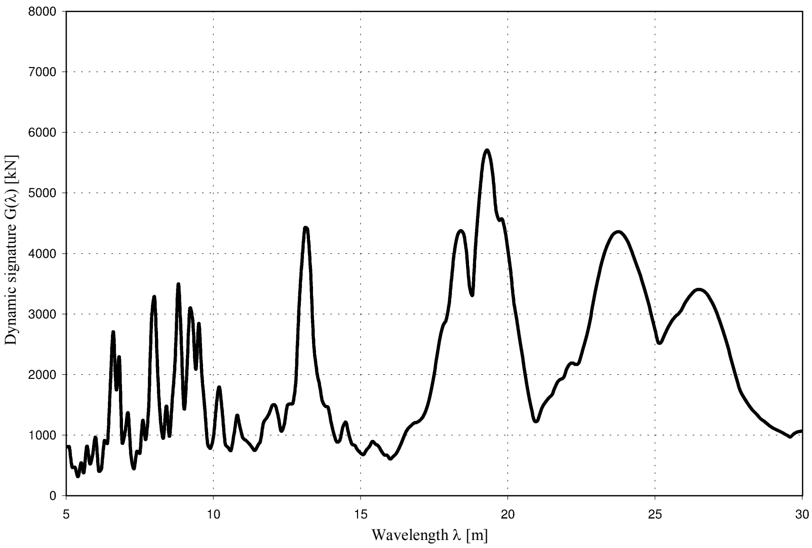



Figure 6: Envelope of dynamic signatures (zero damping) for European high-speed trains

An envelope of these signatures may be easily obtained, as shown in Figure 6. This envelope may be used for design purposes, in order to determine the maximum response of a given bridge in which any of the above high speed trains may circulate. However, this kind of design procedure has two main drawbacks:

1. it is only valid for the cases in which the train signature is applicable, that is simply supported isostatic bridges.
2. it covers the current catalogue of high-speed trains, but does not include possible variations which may be incorporated in the future by the constructors.

It is highly desirable from a social and economical point of view that the high-speed line infrastructure is interoperable, that is all high speed trains from other European lines may also use them even though the line was not foreseen initially specifically for them. From the point of view of structural requirements on bridges the static strength is assured by the static load model LM71. The dynamic performance must be assured by a set of dynamic analyses that covers all possible present (and future) trains.

The task of developing a High Speed Load Model (HSLM) which would ensure interoperability conditions was performed by ERRI D214.2 [14], which first drafted envelopes of DER signatures for all current high-speed trains and their possible variations.

As an example, we document the fit of TALGO HS train within the initially considered UNIV-A envelope (Figure 7), which included only articulated and conventional trains. It may be seen that for short wavelengths ($\lambda < 15$) it does not cover the TALGO signature. In order to cover also regular trains, the final version of this envelope and the resulting HSLM-A family of trains was modified accordingly.

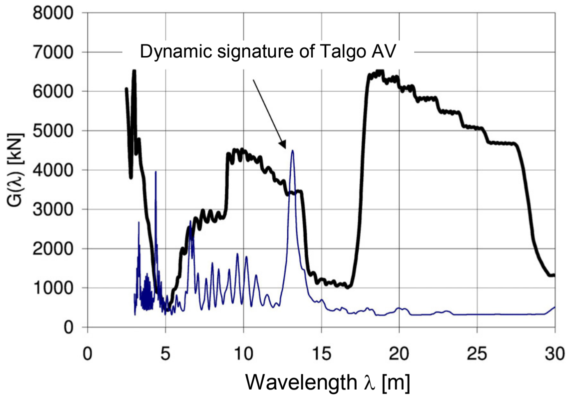



Figure 7: Dynamic signatures of the initial UNIV-A envelope and of Talgo AV

Following, a family of fictitious articulated trains *(Universal trains)* was devised ensuring that their signature envelope effectively covered the signatures of all real trains. This has lead to the HSLM-A and HSLM-B families of universal trains [4,10].

## 4. ISSUES RELATED TO BRIDGES

Some brief aspects regarding the main differences in behaviour to be expected from different structural types of bridges are discussed here.

We first consider the dynamic response of a simply-supported isostatic bridge (Figure 8). A very sharp and intense resonant peak in the response is obtained for a speed around 200 km/h, for which the main bridge frequency equals that of the travelling loads. The dynamic response of isostatic simply-supported bridges is determined mainly by the first mode of vibration, higher modes have a very small participation in the response. However, we remark that for the purpose of calculating reaction forces and section moments higher modes have a larger influence [16]. Further, a remarkable feature of the response is that the effects are maximum at a critical train speed, and do not increase for larger speeds. In fact, it is clearly seen that the response at even 400 km/h is considerably lower to that at the critical speed of 200 km/h.

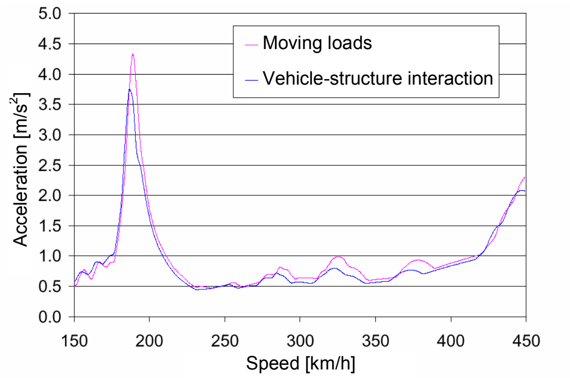



Figure 8: Maximum acceleration at centre of span as a function of train speed for TALGO AV. $L$ =20 m, $m$=7 ton/m, EI=453919 kN/m$^2$, $f_0$=16 Hz, $\zeta$= 1%. Results with and without vehicle-bridge interaction

In current high-speed lines it is very common to have fairly long viaducts of between 100 to 2000 m. Continuous deck bridges are built very often in these cases, as they have a number of advantages. One of the main benefits is from the construction point of view, which very often is by pushing the deck being fabricated at one of the extremes. An important consideration to be made is that of the deck-track interaction, in order to decide the placement of fixed neutral points and track dilatation devices. Finally, from a dynamic point of view, continuous deck bridges exhibit a much different (and generally lower) response to traffic loads than isostatic bridges. This is due to the fact that at any time the response of the bridge is governed not by one vibration mode but by a large number of modes. These modes are excited positively or negatively by traffic loads at the various spans, and the sum of the responses cancels much of their effects.

As an example we present some results for a viaduct in the Madrid-Barcelona line. It is a continuous beam with 9 spans, the outer ones of 37.5 m span and the central ones of 45 m, a total length of 390 m, with a slight curve in plan view (Figure 9).

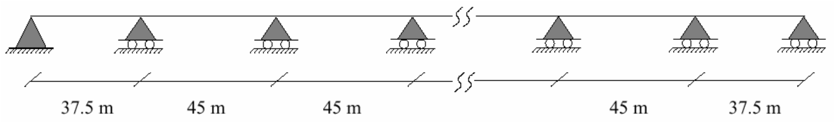



Figure 9: Geometric characteristics of continuous deck viaduct

For this viaduct a complete dynamic modal analysis was carried out under the actions of high speed trains in Europe. In Figure 10 we show the results for the AVE Spanish HS train. In this figure we also show the individual contributions from the individual modes of vibration.

Two main consequences may be drawn from these results. Firstly, in broad terms the maximum structural response does increase monotonically with the higher speeds, being larger for the highest speeds. However, in this case significant modal resonance is obtained for modes 3 and 5 which distort this character slightly.

Secondly, the peak structural response is much lower to that which would be obtained by isostatic simply supported independent spans.

In this viaduct it was performed a complete modal analysis of the structure under the action of six of the most important European high speed trains (Figure 10).

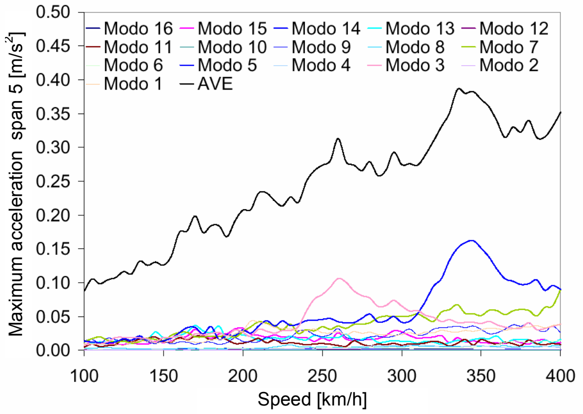



Figure 10: Maximum displacement at the center of span 5 versus speed for a double composition AVE HS train (black line). Also shown in colours are the contributions of the various modes with $f \leq 20$ Hz.

## 5. CONCLUDING REMARKS

As a consequence of the work described above we point out the following remarks:

- Dynamic effects in general and the possibility of resonance in particular require in general a dynamic analysis for the design of high speed railway bridges.
- Simplified models which provide upper bounds for dynamic effects are of limited applicability. Moving load finite element models or even vehicle-structure interaction models for more special cases provide a general methodology.
- The consideration of dynamic vehicle-structure interaction leads to more realistic predictions, in the case where adequate data from the trains are available to build such models. The structural response predicted is somewhat lower to that of moving load models for resonant scenarios. It is these resonant situations that generally limit the design.
- Hyperestatic continuous deck bridges lead generally to a less marked resonance, although a dynamic analysis is still necessary for them. In practice, HSLM models for interoperability of railway lines are adequate bounds of the dynamic effects in the cases studied.

- It is necessary co consider both signs of dynamic effects, including also the dynamic uplift which may be significant in some design scenarios. This may be done through specific design provisions or through a special unloaded train.

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