DYNAMIC LOADS IN NEW ENGINEERING CODES FOR RAILWAY BRIDGES IN EUROPE AND SPAIN

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ABSTRACT

Bridges and viaducts for high speed trains are subject to demanding dynamic loads. In addition to the classical effect of the moving (single) load, much larger and potentially dangerous effects arise from dynamic resonance, for speeds above 220 km/h. The classical methods for evaluation of dynamic impact factors, reflected in the codes of practise existing until recently, do not cover this possibility of resonance. The design of such structures requires dynamic calculations which are the object of this paper. We discuss briefly available methods for dynamic analysis, as well as the new (draft) codes IAPF [9] and Eurocode 1 for actions on bridges [10].

One of the key aspects which is desirable for the new lines is their qualification for interoperability, so that all possible present and future European high speed trains may use them. The proposal for this is covered in [10, 9] through the new High Speed Load Model HSLM, whose background is discussed here.

Finally, some results obtained by our group are presented for high speed traffic loads on bridges. These studies focus on the evaluation of the bridge-vehicle interaction in bridges and a discussion and proposal for dynamic uplift dynamic effects. These topics originate from issues in the application of the new regulations for high speed lines, and are oriented toward being of practical use to designers of railway bridges.

1. INTRODUCTION

Currently one of the main efforts in civil engineering in Spain is dedicated to the new high speed railway lines. These will provide an efficient transport link between Spanish towns as well as with Portugal and the rest of Europe.

This important engineering activity highlights one of the main structural issues associated specifically to the design of bridges and structures in railway lines: the dynamic effects due to moving loads from train traffic. This has been considered since the early stages of railways as one of the main design requirements for the structures. The basic dynamic response for a moving load on a simply supported beam is known from classical solutions (among others) by Timoshenko [1]. More recently further studies have been developed among others by Fryba [2] and Alarcón [3, 4].

The design codes existing up to now [7, 6, 5] 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. This appears 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,

$$\lambda = \frac{v}{f_0}, \tag{1}$$

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:

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

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 subcomittee D214 [13]. These and other findings have been included in the recently drafted engineering codes [8], [10] and [9].

A further aspect which must also be considered is the convenience that the railway lines not be restricted to their use by a limited family of trains. On the contrary, they should allow the transit of all possible high speed trains, enabling *interoperability* of the infrastructure by all possible trains. This issue is not only essential from a social and economical point of view, but has also other implications, following the new European directive to separate the business of the infrastructure with that of the transport operators. Only with adequate interoperability conditions can this be realised.

Hence in principle all European high speed train types should be considered for the design and the corresponding dynamic analyses. The current high speed train types described in [9] and [10] vary widely as to distance between axles, coach lengths, etc. They may be classed into three categories: articulated trains (one bogie between coaches), conventional trains (two bogies per coach), and regular trains (one axle between coaches). However, this strategy of performing dynamic analyses for all train types is not only cumbersome and time consuming, but it also does not guarantee the validity of possible new trains which may appear in the future during the life of the structure. One of the most valuable results of the work of ERRI D214 comittee has been the establishment of a *High Speed Load Model (HSLM)* [14]. This model comprises a family of (fictitious) articulated trains whose dynamic effect has been proved to be an envelope of all current trains as well as those foreseen within an agreed set of *interoperability criteria*. These aspects are discussed in section 3.

The models available in practice for consideration of dynamic effects are, in terms of increasing complexity: 1) impact factor (section 2.1); 2) Dynamic train signature (section 2.2); 3) Moving load dynamic analysis (section 2.3); and 4) Vehicle–structure interaction dynamic analysis (section 4).

The consideration of the vehicle–structure interaction models discussed in section 2.4 produces a reduction of the effects due to the existence of mechanisms which permit energy dissipation (dampers) or systems which interchange energy between structure and vehicle (suspension springs). For non resonant situations or statically redundant bridges, the interaction effects are not usually relevant in the calculation, being sufficient to consider constant load models. However, for isostatic decks with short spans (10 m - 30 m), significant resonant effects appear with high accelerations, and often these constant load models yield results above the design limits. With vehicle–structure interaction models a significant reduction of these results may be obtained (section 4).

In the first part of this paper we present a description of the basic features of the calculation methods available for dynamic analysis of railway bridges subject to traffic loads. Following a summary of the methods prescribed in the new drafts of codes IAPF and Eurocode 1 is done. Finally some research results for specific problems obtained by our group are presented.

2. DYNAMIC ANALYSIS METHODS

2.1 Impact factor Φ

The basic method followed up to now in the existing engineering codes for railway bridges [7, 6, 5] has been that of the impact factor, generally represented as Φ. 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 for a single moving load at speed v on an ideal bridge (i.e. without track or wheel irregularities) is evaluated in [7] to be covered by the following expression:

$$\varphi' = \frac{K}{1 - K + K^4}; \qquad K = \frac{\lambda}{2L_\Phi}, \tag{3}$$

where L_Φ is the equivalent span of the element under study and $\lambda=v/f_0$ the wavelength of excitation. 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 (φ''):

$$\Phi \geq \max(1+\varphi'+\varphi''). \tag{4}$$

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

$$\Phi \cdot E_{\mathrm{sta,LM71}} \geq E_{\mathrm{dyn,real}}. \tag{5}$$

The applicability of impact factor Φ 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. They have been developed and used for a number of years within SNCF, and their basic description may be found in [13].

The *dynamic signature* of a train may be interpreted as a function which characterises its dynamic effect on a given railway bridge. The models of this type proposed are:

- DER: Based en the Decomposition of the Resonance Excitation.
- LIR: Simplified method based on the Residual Influence Line.
- IDP: A slightly modified version of LIR method with improved accuracy, proposed by [11].

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 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 Γ at the centre of the span is given by:

$$\Gamma = C_{\mathrm{acel}} \cdot A(K) \cdot G(\lambda), \tag{6}$$

where $C_{acel}=1/M$ is a constant (the inverse of the total mass of the bridge), and the remaining terms are:

$$A(K) = \frac{K}{1-K^2}\sqrt{e^{-2\zeta\frac{\pi}{K}} + 1 + 2\cos\left(\frac{\pi}{K}\right)e^{-\zeta\frac{\pi}{K}}} \tag{7}$$

$$G(\lambda) = \max_{i=1}^{N}\sqrt{\left[\sum_{x_1}^{x_i} F_i\cos(2\pi\delta_i)\,e^{-2\pi\zeta\delta_i}\right]^2 + \left[\sum_{x_1}^{x_i} F_i\sin(2\pi\delta_i)\,e^{-2\pi\zeta\delta_i}\right]^2} \tag{8}$$

In these expressions ζ 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 (8)) 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.

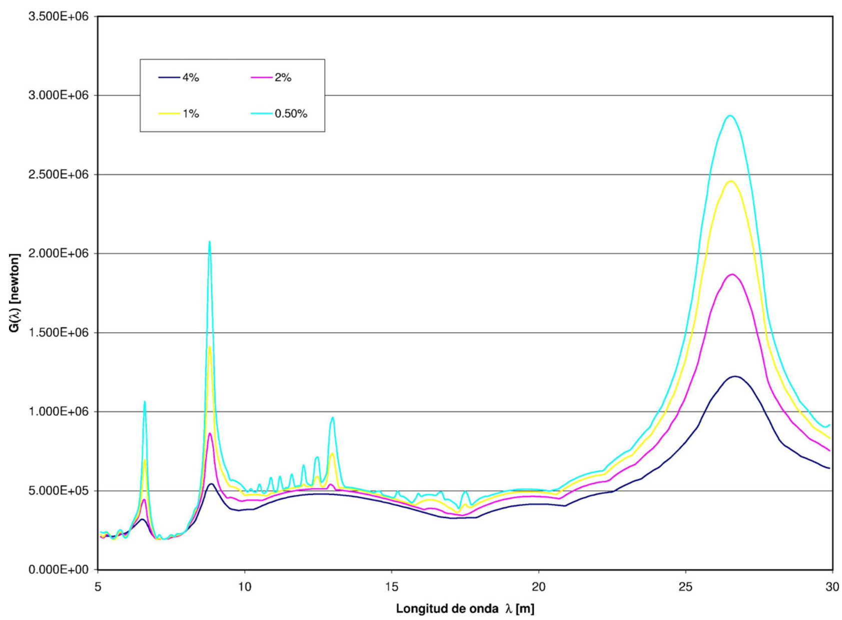

Figure 1: Dynamic signature of ICE2 train for different damping rates.

The term $A(K)$ 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 (ζ).

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 model for the structure may be analysed either through an direct integration of the complete system,

$$\mathbf{M}\ddot{\mathbf{d}} + \mathbf{C}\dot{\mathbf{d}} + \mathbf{K}\mathbf{d} = \mathbf{f}, \tag{9}$$

where $\boldsymbol{M}$ is the mass matrix, $\boldsymbol{C}$ the damping matrix, $\boldsymbol{K}$ the stiffness matrix, $\boldsymbol{f}$ the external load vector, and $\boldsymbol{d}$ the vector of (unknown) nodal displacements.

By means of the direct integration of the model, the complete system (9) of N degrees of freedom would be solved for each time step; the equations are generally coupled, and therefore must be solved simultaneously. This procedure is also valid when nonlinear effects are to be included in the response; in this case the elastic internal forces and viscous damping forces from the previous expression should be replaced by a general term (nonlinear) of the type $\boldsymbol{F}^{int}(\boldsymbol{d},\dot{\boldsymbol{d}},...)$.

Alternatively, 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 general it is far more efficient to integrate the reduced modal equations. The first step is to obtain the eigenmode shapes and associated eigenfrequencies. For the trivial example of a simply supported bridge, the eigenmodes may be expressed analytically [12] as $\phi_i(x)=\sin(i\pi x/l)$, with associated eigenfrequencies $\omega_i=(i\pi)^2\sqrt{EI/(\overline{m}l^4)}$. For this simple case, it is generally sufficient to consider a single vibration mode; this way the problem is reduced to a dynamic equation with one degree of freedom. However, for an accurate evaluation of section resultants or reactions a larger number of modes may be necessary, as discussed in [17]. For more complex structures it is also necessary to consider more vibration modes.

Once the vibration modes are known, the basic response of each mode to a single moving load F or to a complete load train $F_k, k=1...n_{ax}$ (Figure 2) may be evaluated. This may be assembled as the superposition of the responses for each individual load F_k in the following manner:

$$M_i \ddot{y}_i + 2\zeta_i \omega_i M_i \dot{y}_i + \omega_i^2 M_i y_i = \sum_{k=1}^{n_{ax}} F_k \langle \phi_i (vt - d_k) \rangle . \tag{10}$$

In the above equation , $\phi_i(x)$, M_i and ω_i are respectively the modal shape, modal mass and eigenfrequency for eigenmode i; y_i is the modal amplitude, ζ_i the damping fraction, and $\langle \phi(\bullet) \rangle$ represents a bracket notation with the following meaning:

$$\langle \phi(x) \rangle = \begin{cases} \phi(x) & \text{if } 0 < x < l \\ 0 & \text{otherwise.} \end{cases} \tag{11}$$

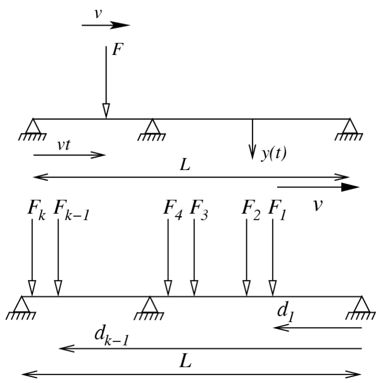

Figure 2: Model for a) single moving load and b) train of loads.

Except for particular cases of simple structures the above equations (10) must be evaluated numerically by finite element methods. These provide an efficient method for calculation in arbitrary structures. Adequate procedures for preprocessing (definition of load histories for all individual axles) and postprocessing 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 3, 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.

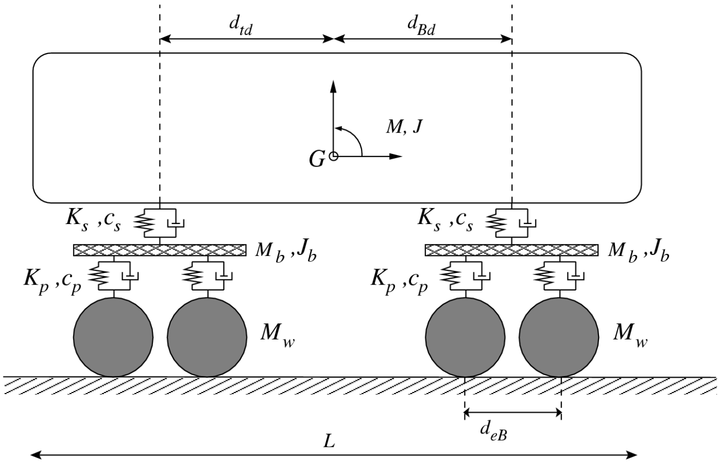

Figure 3: Complete vehicle–structure interaction model.

The level of detail in the above model is not always necessary. In this work we employ a simplified model in which for each axle only the primary suspension, equivalent nonsprung and sprung masses are considered (Figure 4). In this model each axis is independent from the rest, thus neglecting the coupling provided by the bogies and vehicle box. Further details of the model are described in [11].

For a train of *k* loads, each axle is represented by an interaction element (Figure 4).

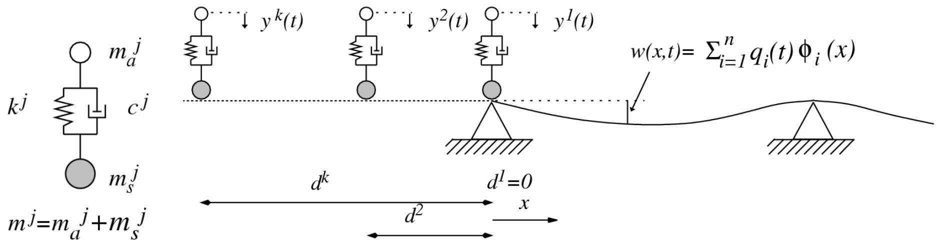

Figure 4: Crossing of a train of loads, according to the vehicle-structure interaction simplified model: a) interaction element; b) geometric definition of variables

The model thus obtained considers a degree of freedom for each mode of the structure and an extra one for each interaction element. The equation for each mode (i=1,...n) is

$$M_i\,\ddot{q}_i + C_i\,\dot{q}_i + K_i\,q_i = \sum_{j=1}^{k}\langle\phi_i(d^j_{\mathrm{rel}})\rangle\left(g\,m^j + m^j_a\,\ddot{y}^j\right). \tag{12}$$

For each interaction element (j=1,...k):

$$m_a^j\ddot{y}^j + k^j\left[y^j - \sum_{i=1}^{n} q_i\langle\phi_i(d^j_{\mathrm{rel}})\rangle\right] + c^j\left[\dot{y}^j - \sum_{i=1}^{n}\dot{q}_i\langle\phi_i(d^j_{\mathrm{rel}})\rangle - \sum_{i=1}^{n} q_i v\langle\phi_i'(d^j_{\mathrm{rel}})\rangle\right] = 0 \tag{13}$$

In the above equations the notation $\langle\phi(\bullet)\rangle$ defined previously (11) has been employed. Additionally, d_{rel}^{j} represents the relative position on the bridge for each element j:

$$d_{\mathrm{rel}}^{j} = vt - d^{j} \tag{14}$$

Finally, theses equations may be solved in time by standard numerical integration schemes in structural dynamics, such as trapezoidal or HHT rules.

3. HIGH SPEED REAL TRAINS AND INTEROPERABILITY

It is highly desirable from a social and economical point of view that 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.

European high speed trains may be classified into three different types (Figure 5):

1. *Articulated trains:* each two coaches share one bogie between them. This type includes Thalys, AVE and Eurostar.
2. *Conventional trains:* each coach has two bogies. This includes Ice2, 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.

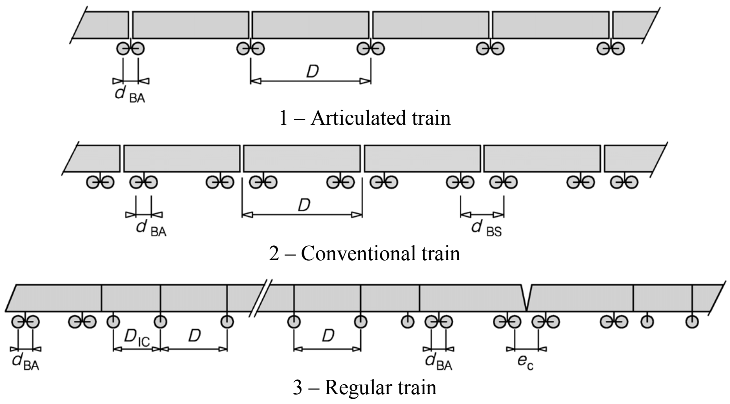

Figure 5: Different types of high-speed trains, according to Eurocode 1 [10]

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 interperability conditions.

The concept of train signature (section 2.2) is very useful for the purpose of obtaining a dynamic envelope. Figure 6 shows the dynamic signature (DER method) obtained for the most common current European high-speed trains. An envelope of these signatures may be easily obtained, as shown in Figure 7.

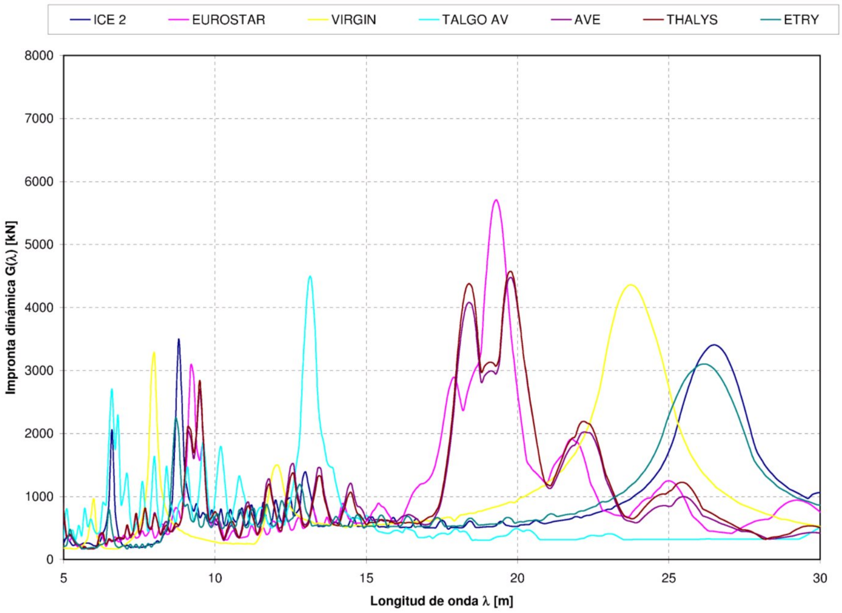

Figure 6: Dynamic signatures (zero damping) for European high-speed trains.

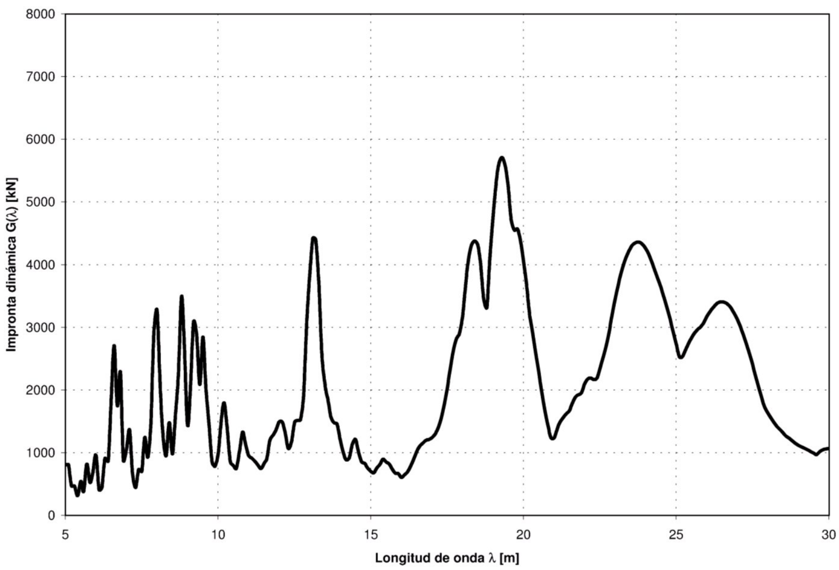

Figure 7: Envelope of dynamic signatures (zero damping) for European high-speed trains.

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. Following, a family of fictitious articulated trains *(Universal trains)* was devised ensuring that their signature envelope effectively covered the signatures of all real trains. Table 1 summarises the characteristics of HSLM-A family of universal trains. A further family HSLM-B must be used for bridges with span L<7 m.

Table 1: Characteristics of HSLM-A universal trains. ([14], [10], [9])

	HSLM-A
Type	articulated
Total length	≈ 400 m
Coach length D	18 m – 27 m
Axle point load	170 kN – 210 kN
Bogie axle spacing d	2.0 m – 3.5 m
Head and tail locomotives	yes

Dynamic analysis with HSLM model hence requires the analysis for the 10 fictitious universal trains at all possible speeds, up to the 1.2× the maximum permitted speed.

Further to the above envelopes, a procedure is also defined in [14] and [10] for simple bridges which allows to determine the critical universal train and associated speed. In such cases the dynamic analysis is simplified.

4. EVALUATION OF THE DYNAMIC VEHICLE–STRUCTURE INTERACTION

The object of this application is to evaluate the effective reduction which is obtained, with respect to the dynamic analysis made without considering the vehicle-structure interaction, such as the models based on series of harmonics or models of moving loads, more common in engineering practise.

The calculations are based on a modal analysis considering only first mode of vibration, without shear deformation. A model of moving loads is compared to the model with interaction proposed in section 2.4. Time integration has been carried out using the trapezoidal rule. A family of simply supported bridges of spans (L) between 10 and 40 m have been considered, with characteristics following the catalogue of isostatic bridges from [13]. The speed sweep is (120-420) km/h, with Δv=2.5 km/h. The trains used are the Ice2, Eurostar and Talgo AV, defined in [9], with damping rates ζ=0.5%, 1%, 1.5%, 2.0%. Further technical details of the numerical model are contained in [11].

The analysis results, as was predictable, show a significant reduction of the maximum displacements and accelerations for models with interaction. Some of the results obtained are included in Table 2.

Table 2: Percent reduction for maximum acceleration and displacement for vehicle-structure interaction model as compared to moving load model. V_{max}^{line}=220 and 375 km/h

220 km/h	ζ=0.5%		ζ=1.0%		ζ=2.0%	
L (m)	disp.	accel.	disp.	accel.	disp.	accel.
5	-25%	-35%	-15%	-25%	-10%	-20%
10	-30%	-35%	-20%	-25%	-10%	-15%
15	-25%	-45%	-15%	-35%	-5%	-20%
20	-10%	-20%	-5%	-15%	0%	-10%
25	-10%	-35%	-5%	-25%	0%	-10%
30	0%	-15%	0%	-5%	0%	-0%
40	0%	-10%	0%	-5%	0%	-5%

375 km/h	ζ=0.5%		ζ=1.0%		ζ=2.0%	
L (m)	disp.	accel.	disp.	accel.	disp.	accel.
5	-25%	-35%	-15%	-25%	-10%	-20%
10	-30%	-35%	-25%	-25%	-15%	-15%
15	-30%	-45%	-20%	-35%	-10%	-20%
20	-20%	-20%	-15%	-20%	-10%	-15%
25	-20%	-35%	-15%	-25%	-5%	-15%
30	-10%	-15%	-5%	-15%	-5%	-10%
40	-5%	-10%	0%	-10%	0%	-5%

In view of the results shown, one may conclude in first place that the moving load models clearly overestimate, in general terms, the resonant response in accelerations and displacements of an isostatic structure; in comparative terms, the interaction models can reduce the maximum acceleration values in isostatic bridges up to 45% respect to acceleration obtained with moving load models.

Additionally, the dynamic response reduction, for the same hypothesis of span and damping, is greater for accelerations than for displacements, and the reduction increases as the line design speed is increased. Finally, it is also observed that the reduction of the response decreases when damping rate or the bridge span increases.

5. DYNAMIC UPLIFT

We discuss here some recent results for evaluating dynamic uplift effects. Some of these results have been considered for the code [9]. Under some circunstances these may be relevant from a structural point of view. A typical example is the verification of bridge piers in a continuous deck bridge, for which the limiting case is the minimum vertical loads simultaneously with maximum horizontal loads (centrifugal and wind mainly). This aspect is not addressed directly in [10], where an *unloaded train* is proposed for these scenarios. The results shown here summarise thos described with greater detail in [17].

As a result of interpreting the dynamic response as oscillations around a quasi-static state it is possible to obtain bounds for maxima and minima, computed from the said static response and the amplitude of oscillation. Figure 8 shows the vertical reaction in a pier between two (simply supported) spans in a real bridge (Tajo river), computed for three different cases with the Eurostar train. Details of the structure and of analysis model may be found in [16]. Two of these cases are dynamic results for a speed of 225 km/h which was shown to produce resonance, with a moving load model and with a vehicle-structure interaction model. Additionally, the quasi-static low-speed (20 km/h) results are superposed on to the previous cases (these are previously scaled in pseudo-time in order to correspond with the dynamic cases).

The above results show that the dynamic vibration may be interpreted as a dynamic effect $\pm\Delta E_{din}$ which is superposed on the quasi-static one, E_{sta}. The maximum dynamic effects obtained would be $E_{max}=S_{sta}+\Delta E_{dyn}$, whereas the minima would result $E_{max}=S_{sta}-\Delta E_{dyn}$. The time instant in the figure for which the level E_{min} shown ceases to be a lower bound corresponds to a moment at which the train has already exited the first span, which then remains in free vibration. The minimum dynamic effects correspond to unloadings, that is upward reactions. Although these are significant, they would not effectively produce a lifting of the deck from the pier which would prescribe an anchorage, due to the permanent self-weight loads. However, their consideration may be necessary for some design features such as those governed by horizontal loads.

A further feature which may be observed in Figure 8 is that the model with interaction predicts results which are slightly below those of the moving load model. This was expected in a resonant scenario.

A complete set of analyses of this type has been carried out for a set of simply supported and hyperestatic (continuous deck) bridges, reported in [16]. In Figure 9 a representative result is shown for a continuous deck viaduct with 17 spans over river Cabra. The case shown here is for the bending moment at the centre of the first span, produced by Eurostar train at 420 km/h. This result differs in several important aspects from the previous one. Firstly, although the speed selected is that for which maximum dynamic response was obtained, it does not correspond to a resonant scenario. (This is common in hyperestatic bridges for which resonant peaks are not so pronounced or may not be significant, as numerous competing vibration modes take part in the response at a given point.) As a consequence, the dynamic response has a lower relative importance relative to the quasi-static response. In other words, the quasi-static part of the response represents a greater fraction of the total maximum or minimum dynamic response. Secondly, the result for the model with interaction is also shown here. In this case the result predicted by the model with interaction turns out to be greater than the one with moving loads. This is also due to the fact mentioned above that the situation is not dominated by resonance, contrary to the results shown in section 4.

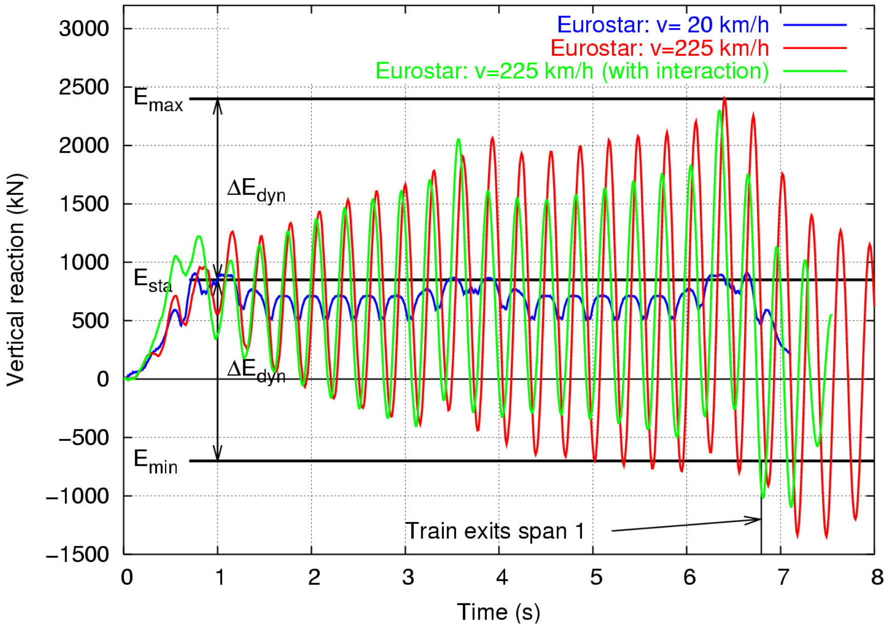

Figure 8: Time history of vertical reactions at a pier of Tajo river viaduct (simply supported spans), for Eurostar train at a speed of v=225 km/h (resonant speed).

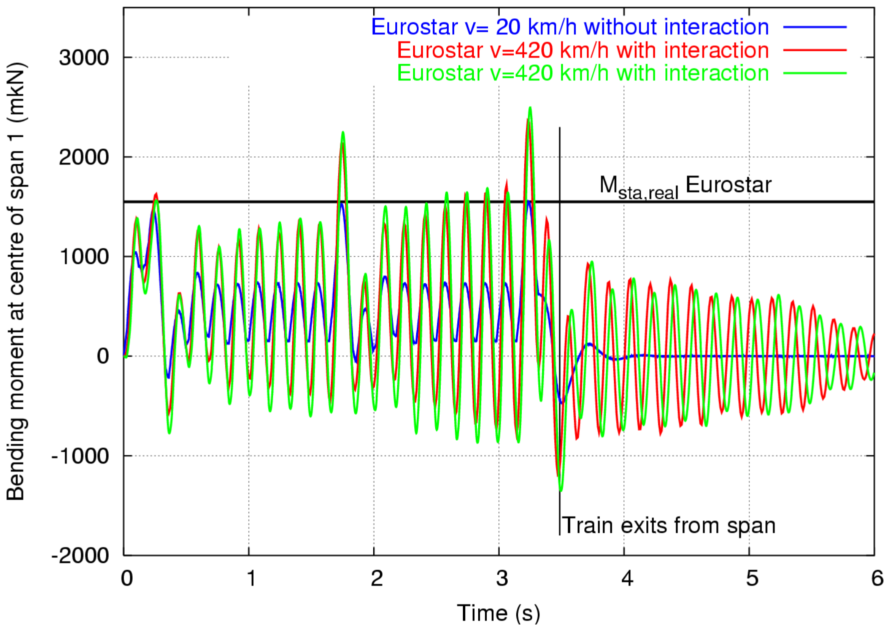

Figure 9: Time history of bending moment at the centre of the first span of the continuous deck viaduct over Cabra river, for Eurostar train at a speed of v=420 km/h (speed for maximum dynamic effects).

From the above results and the consideration of the complete set of results in a set of representative cases [16], [17], a proposal was drafted for a design envelope of uplift effects:

$$\Phi_{\min} = 2f_e - \Phi_r; \quad \Phi_{\min} \geq 0, \tag{15}$$

where $f_e = E_{sta,real}/E_{sta,LMd}$ is the ratio between the static response for real trains and that of the design static load model (LM71×α), and Φ_r is the real impact coefficient, defined by $E_{max} = \Phi_r E_{sta,LMd}$. The loads for the design static load model are considerably larger than those of the much lighter passenger high speed trains, and as a result f_e normally lies between 0.25 and 0.35. Consequently, the coefficient Φ_{min} may end up having negative values, which would represent a net dynamic uplift due to the traffic induced structural vibration. We must take into account that this net uplift must be superposed to the generally greater effects of the permanent self-weight loads, hence the deck would not really lift up from the piers.

6. 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 applciability. 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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