Design Issues for Dynamics of High Speed Railway Bridges

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ABSTRACT: This work discusses the influence of issues arising from the dynamic resonant response on the design of railway bridges. These issues are related to dynamic analysis methods, composition of trains, and type of bridges. Some considerations are expressed on the occurrence of resonance, as well as on the influence of the above issues on the calculation and mitigation of dynamic effects. The evaluation of different response magnitudes is also addressed, showing that for calculation of bending moments or support reactions a much larger number of modes is needed in the model. Finally, a case study is presented with dynamic calculations for a large three-dimensional model of a special bridge.

1INTRODUCTION

Dynamic response of railway bridges is a major factor for design and maintenance, especially in new high speed railway lines. The main concern is the risk of resonance from periodic action of moving train loads. In cases when such risk is relevant (e.g. for speeds above 200 km/h) a dynamic analysis is mandatory.

The new engineering codes [EN1991-2 (2003) , EN1990-A2 (2004), IAPF (2003), FS (1997)] take into account these issues and define the conditions under which dynamic analysis must be performed, providing guidelines for models, types of trains to be considered, and limits of acceptance [Goicolea et al (2004)].

Resonance for a train of periodically spaced loads may occur when these are applied serially to the fundamental mode of vibration of a bridge and they all occur with the same phase, thus accumulating the vibration energy from the action of each axle. If the train speed is v, the spacing of the loads D and the fundamental frequency f_0, defining the *excitation wavelength* as $\lambda = v/f_0$, the condition for critical resonant speeds is expressed as [EN1991-2 (2003)]:

$$\lambda=\frac{D}{i},\ i=1,\ldots 4.$$

From a technical point of view a number of methods for dynamic analysis are available. However from a practical design perspective the issues which will either help to improve or modify a design in a desired direction for dynamic performance are often not clearly understood. Engineers find it difficult to comprehend the implications of design decisions in dynamic performance, being usually more at ease with static structural reasoning and models. At most, there is a tendency to employ dynamic calculation models on a pure trial and error basis.

The purpose of this paper is to comment a few selected topics arising from dynamics of railway bridges relevant to design, attempting to provide increased understanding. These issues are related to methods for analysis (section 2), characteristics of trains (section 3) and of bridges (section 4). Finally, a representative case study is presented in section 5.

2METHODS FOR DYNAMIC ANALYSIS

2.1*Static envelopes with impact factor*

The basic method employed up to now in the engineering codes for railway bridges has been that of the impact factor, generally represented as Φ. The impact factor is applied to the effects obtained for the static calculation with the nominal train type of load model LM71 (also called UIC71): $\Phi \cdot \mathrm{LM71} \rightarrow \Phi \cdot E_{\mathrm{sta,LM71}} \geq E_{\mathrm{dyn,real}}$. 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, being much heavier than modern high speed passenger trains (2 to 4 times).

This factor Φ represents the dynamic effect of a single moving load, but *does not include resonant dynamic effects*. As a consequence, applicability is subject to some restrictions, mainly for a maximum train speed of 200 km/h [EN1991-2 (2003)], as well as some other conditions such as bounds for the fundamental frequency f_0. Otherwise, dynamic calculations must be carried out.

2.2*Dynamic analysis with moving loads*

Dynamic analysis may be carried out by direct application of moving loads, with each axle represented by a load F_i travelling at the train speed v (Figure 1). This may be performed by finite element or similar programs, commercial or academic [ROBOT (2002), FEAP (2005)]. The main specific feature which is necessary in practice is a facility for definition of load histories [Gabaldón (2004)]. Dynamic calculation is generally carried out taking advantage of modal analysis, which reduces greatly the degrees of freedom to be integrated. A direct integration of the complete model is also possible, albeit very costly for large three-dimensional models.

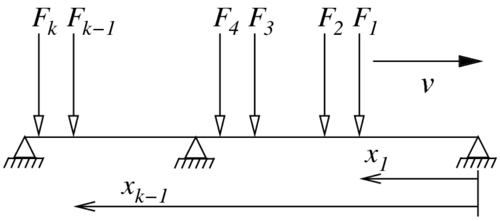

Figure 1: Load model for a train of moving loads

In principle, each response magnitude to be checked should be evaluated independently in the dynamic analysis; however, this may not be practical for engineering calculations. A common simplification is to perform a dynamic calculation to compute a single overall impact factor measuring a characteristic magnitude E, such as the displacement at mid-span. This factor is later assumed to apply for all the response magnitudes to be checked. In such way, a *real* impact factor may be computed from the dynamic analysis [IAPF (2003)]:

$$\Phi_{\mathrm{real}} = \frac{E_{\mathrm{dyn,real}}}{E_{\mathrm{sta,LM71}}}$$

The factor to be considered finally for design will be the largest of this Φ_{real} and the *envelope* impact factor Φ discussed in section 2.1. As has been said before, due to HS passenger trains being much lighter than the LM71 model, only for severely resonant situations will Φ_{real} be larger than the envelope impact factor Φ.

It is important to consider also ELS dynamic limits [EN1990-A2 (2004), Nasarre (2004)] (maximum acceleration, rotations and deflections, etc.), which are often the most critical design issues in practice. Accelerations must be independently obtained in the dynamic analysis. In the example shown in Figure 3 both maximum displacements and accelerations are obtained independently and checked against their nominal (LM71) or limit values respectively.

2.3*Dynamic analysis with bridge-train interaction*

The consideration of the vibration 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 masses, bodies and springs which represent wheels, bogies and coaches. A general model for a conventional coach on two bogies is shown in Figure 2a, 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). Corresponding models may be constructed for articulated or regular trains.

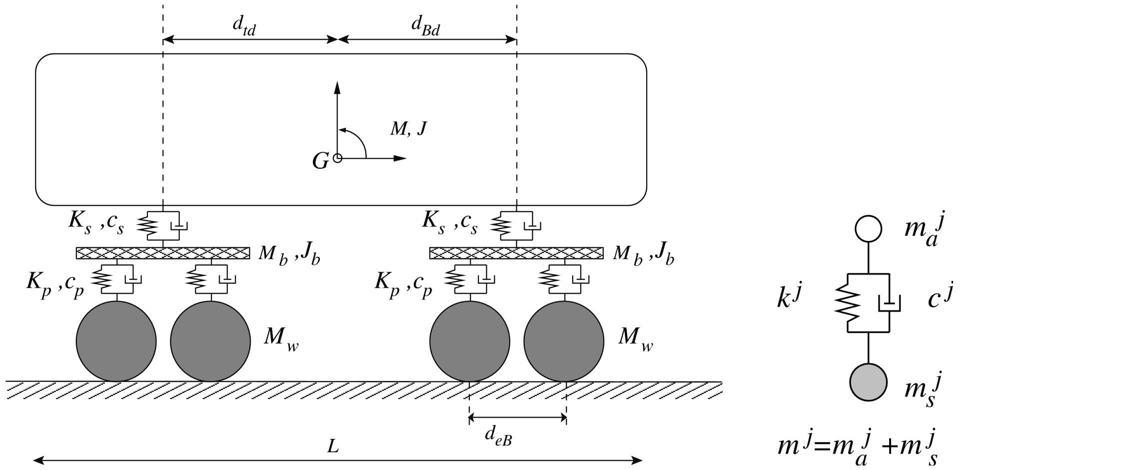

Figure 2: Vehicle–structure interaction models: (a) full interaction model; (b) simplified interaction model

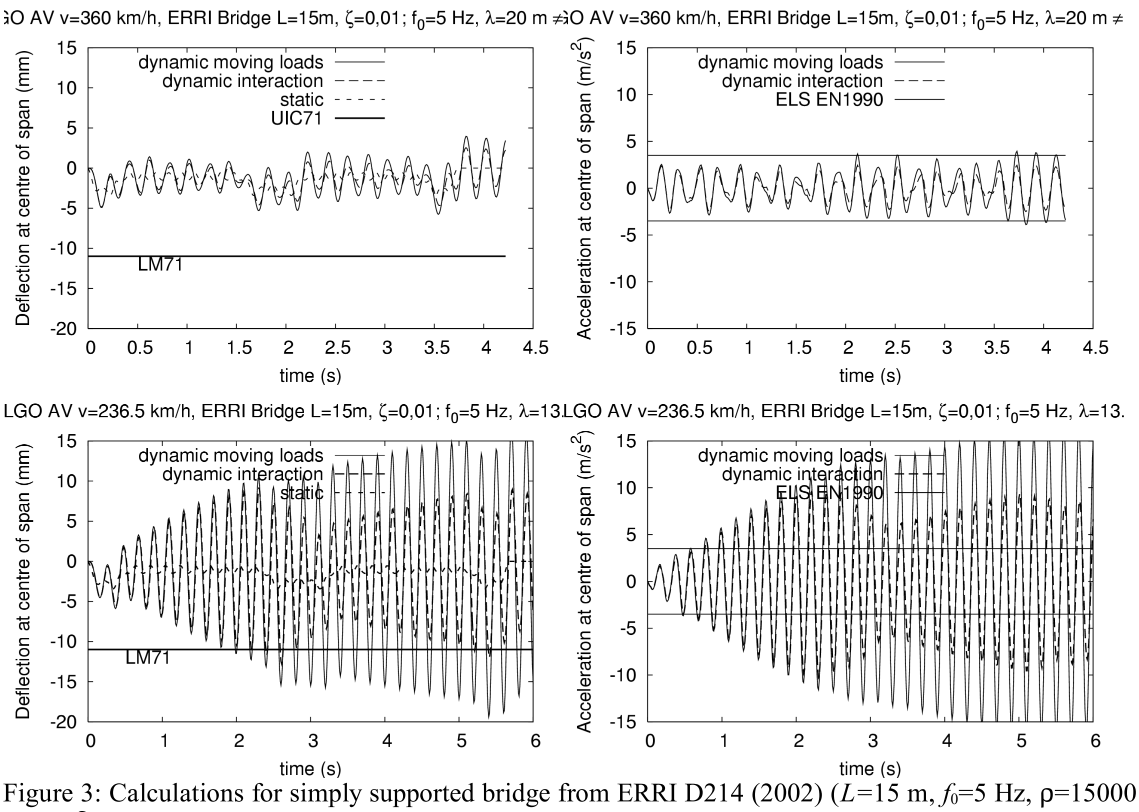

Figure 3: Calculations for simply supported bridge from ERRI D214 (2002) (L=15 m, f_0=5 Hz, ρ=15000 kg/m, δ_{LM71}=11 mm), with TALGO AV2 train, for non-resonant (360 km/h, top) and resonant (236.5 km/h, bottom) speeds, considering dynamic analysis with moving loads and with train-bridge interaction. Note that the response at the higher speed (360 km/h) is considerably smaller than for the critical speed of 236.5 km/h. The graphs at left show displacements, comparing with the quasi-static response of the real train and the LM71 model, and those at right accelerations, compared with the limit of 3.5 m/s^2 [EN1990 (2004)].

The detail of the above model is not always necessary, and often simplified models may be employed considering for each axle only the primary suspension (k^j and c^j) and an equivalent sprung mass m_a^j (Figure 2b). The remaining mass m_s^j corresponds mainly to the coach body on softer springs and is considered to be decoupled dynamically. This model also neglects the coupling provided by the bogies and vehicle box, as well as the rocking motion of the vehicle. Further details of these models are described in Domínguez (2001).

An application of dynamic calculations using moving loads and simplified interaction models is shown in Figure 3. A considerable reduction of vibration is obtained in short span bridges under resonance by using interaction models. This may be explained considering that part of the energy from the vibration is be transmitted from the bridge to the vehicles. However, only a modest reduction is obtained for non-resonant speeds. Further, in longer spans or in continuous deck bridges the advantage gained by employing interaction models will generally be very small. This is exemplified in Figure 6, showing results of sweeps of dynamic calculations for three bridges of different spans. As a consequence it is not generally considered necessary to perform dynamic analysis with interaction for design purposes.

2.4 *Evaluation of Displacements and other Dynamic Response Magnitudes*

In some situations specific dynamic response magnitudes are required directly from the analysis model. This situation arises when a more precise evaluation is required than what would be obtained by using an overall factor Φ_{real} computed from say a displacement response. Here we would like to call the attention to the fact that the model to be employed, for instance the number of modes considered in the integration, need not be the same for all cases.

To illustrate this we develop a model problem, a sudden step load *P* at the centre of a simply supported span. A closed form solution may be obtained for the response of each mode, obtaining the total magnitude as sum of a series. For instance, the displacement and bending moments at *x=L/2* result:

$$\delta\left(\frac{L}{2},t\right)=\frac{2PL^3}{\pi^4 EI}\left\{\sum_{n=1}^{\infty}\frac{1}{(2\mathrm{n}-1)^4}-\right.$$
$$\left.-\sum_{n=1}^{\infty}\left[\frac{\cos\left(\omega_{2\mathrm{n}-1}\sqrt{1-\varsigma_{2\mathrm{n}-1}^2}\,t\right)-\frac{\varsigma_{2\mathrm{n}-1}}{\sqrt{1-\varsigma_{2\mathrm{n}-1}^2 t}}\sin\left(\omega_{2\mathrm{n}-1}\sqrt{1-\varsigma_{2\mathrm{n}-1}^2}\,t\right)}{(2\mathrm{n}-1)^4}\mathrm{e}^{-\varsigma_{2\mathrm{n}-1}\omega_{2\mathrm{n}-1}t}\right]\right\}$$

$$M\left(\frac{L}{2},t\right)=-\frac{2PL}{\pi^2}\left\{\sum_{n=1}^{\infty}\left[\frac{1}{(2\mathrm{n}-1)^2}\right]-\right.$$
$$\left.-\sum_{n=1}^{\infty}\left[\frac{\cos\left(\omega_{2\mathrm{n}-1}\sqrt{1-\varsigma_{2\mathrm{n}-1}^2}\,t\right)-\frac{\varsigma_{2\mathrm{n}-1}}{\sqrt{1-\varsigma_{2\mathrm{n}-1}^2 t}}\sin\left(\omega_{2\mathrm{n}-1}\sqrt{1-\varsigma_{2\mathrm{n}-1}^2}\,t\right)}{(2\mathrm{n}-1)^2}\mathrm{e}^{-\varsigma_{2\mathrm{n}-1}\omega_{2\mathrm{n}-1}t}\right]\right\}$$

In the steady-state limit we recover the static values expected,

$$\delta\left(\frac{L}{2},t\to\infty\right)=\frac{2PL^3}{\pi^4 EI}\sum_{n=1}^{\infty}\left[\frac{1}{(2\mathrm{n}-1)^4}\right]=\frac{2PL^3}{\pi^4 EI}\frac{\pi^4}{96}=\frac{PL^3}{48EI}$$

$$M\left(\frac{L}{2},t\to\infty\right)=-\frac{2PL}{\pi^2}\sum_{n=1}^{\infty}\left[\frac{1}{(2\mathrm{n}-1)^2}\right]=-\frac{2PL}{\pi^2}\frac{\pi^2}{8}=\frac{PL}{4}$$

Similar developments may be obtained for shear forces [Goicolea et al (2003)] which are not shown here for lack of space. The results for a typical railway bridge are shown in Figure 4. One may see that for displacements at centre span only the first mode gives an excellent approximation. However, for the bending moment 10 modes must be considered for similar precision.

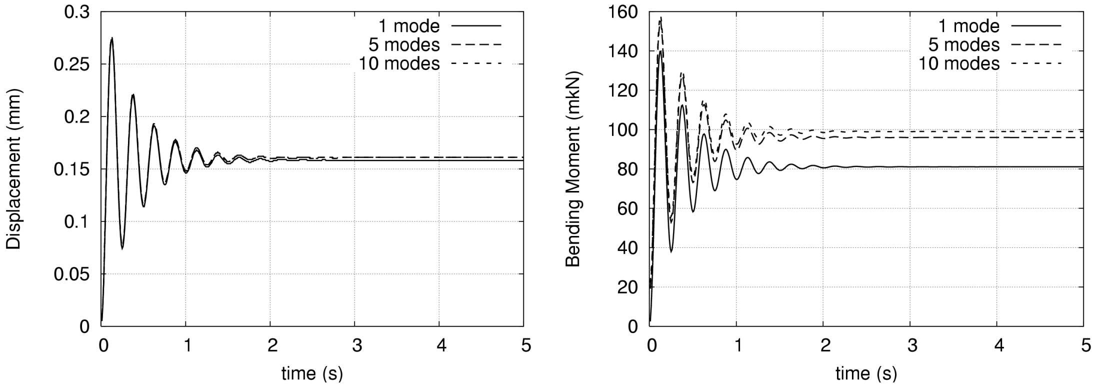

Figure 4. Response of simply supported bridge (L=20 m, f_0=4 Hz, ρ=20000 kg/m) under step load P=20 kN, with damping ζ=10%. Results for displacement and for bending moment at centre of span as a function of the number of modes considered in model [Goicolea et al (2003)].

3TRAINS

The consideration of traffic loads must be done for all the possible trains on the line, each at all possible speeds, as it has been shown that the largest effects are not generally obtained at maximum speed. Further, interoperable lines in Europe should be prepared to accept any HS train from another network. Although estimates of critical speeds may be evaluated by eqn. , in practice it is advisable to perform velocity sweeps (see e.g. Figure 6) for each train in order to estimate better the significance of resonance.

The existing trains in Europe are defined in EN1991-2 (2003), IAPF (2003), and classified into conventional (ICE, ETR-Y, VIRGIN), articulated (THALYS, AVE, EUROSTAR) and regular (TALGO). Variations of these trains which satisfy interoperability criteria have been shown to covered by the dynamic effects of the High Speed Load Model (HSLM), a set of universal fictitious trains proposed by ERRI D214 (2002). The use of this new load model is highly recommended for all new railway lines, and incorporated into codes EN1991-2 (2003) and IAPF (2003).

A useful way to compare the action of different trains and to evaluate the performance of HSLM as an envelope is to employ the so-called *dynamic train signature* models. These 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. Their basic description may be found in [ERRI D214 (2002)]. They furnish an analytical evaluation of an upper bound for the dynamic response of a given bridge. For a bridge of span L, with fundamental frequency f_0 and zero damping (ζ=0%),the maximum acceleration Γ is obtained as:

$$\Gamma = \frac{1}{M} \cdot A(K) \cdot G(\lambda),$$

$$A(K) = \frac{K}{1-K^2}\sqrt{2\left(1+\cos(\pi/K)\right)},$$

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

In these expressions M is the total mass of the deck, $K=\lambda/(2L)$, and $\delta_i = x_i/\lambda$ with x_i the distance of each one of the N axle loads F_i to the first axle of the train (Figure 1). The result is expressed as a product of three terms: a constant term $1/M$, the *dynamic influence line* of the bridge $A(K)$, and the *dynamic signature* of the train $G(\lambda)$. This function depends only on the distribution of the train axle loads. Each train has its own dynamic signature, which is independent of the characteristics of the bridge. The above expressions have been applied in Figure 5 to represent the dynamic signature of three HS trains, one of each class, together with the envelope of HSLM.

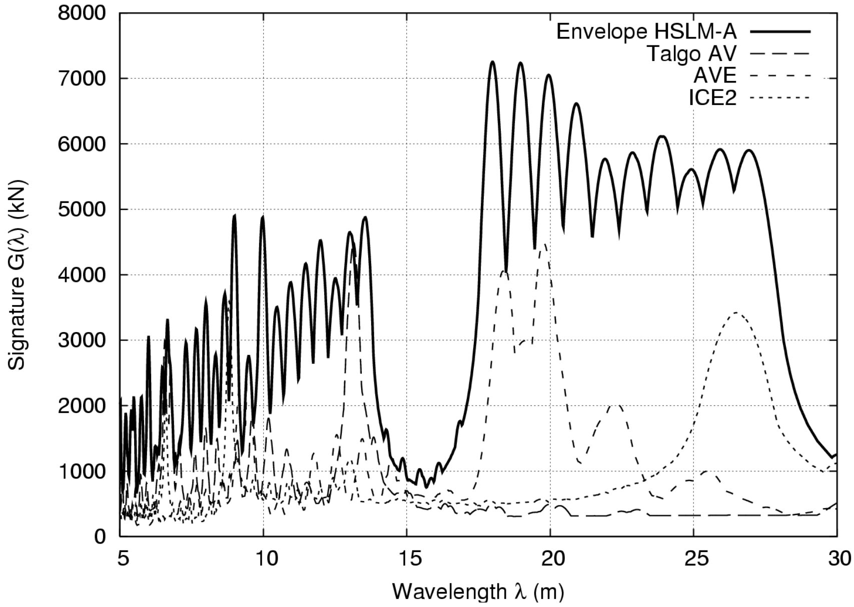

Figure 5. Dynamic signatures for three types of HS trains (conventional type ICE2, articulated type AVE and regular type TALGO AV2), together with the envelope of signatures for High Speed Load Model HSLM-A, showing the adequacy of this load model for dynamic analysis. It may be remarked that the critical wavelengths for each train coincides approximately with the coach lengths (13.14 m for TALGO AV2, 18.7 m for AVE and 26.44 m for ICE2)

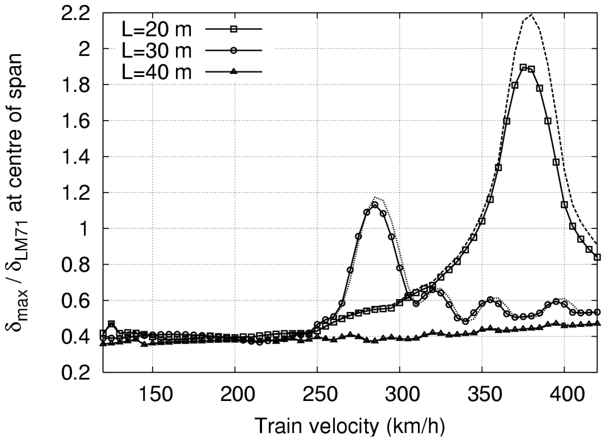

Figure 6. Normalised envelope of dynamic effects (displacement) for ICE2 high-speed train between 120 and 420 km/h on simply supported bridges of different spans (L=20 m, f_0=4 Hz, ρ=20000 kg/m, δ_{LM71}=11.79 mm, L=30 m, f_0=3 Hz, ρ=25000 kg/m, δ_{LM71}=15.07 mm and L=40 m, f_0=3 Hz, ρ=30000 kg/m, δ_{LM71}=11.81 mm). Dashed lines represent analysis with moving loads, solid lines with symbols models with interaction. Damping is ζ=2% in all cases.

4BRIDGES

Generally resonance may be much larger for short span bridges. As a representative example, Figure 6 shows the normalised displacement response envelopes obtained for ICE2 train in a velocity sweep between 120 and 420 km/h at intervals of 5 km/h. Calculations are performed for three different bridges, from short to moderate lengths (20 m, 30 m and 40 m). The maximum response obtained for the short length bridge is many times larger that the other. The physical reason is that for bridges longer than coach length at any given time several axles or bogies will be on the bridge with different phases, thus cancelling effects and impeding a clear resonance. We also remark that for lower speeds in all three cases the response is approximately 2.5 times lower than that of the much heavier nominal train LM71. Resonance increases this response by a factor of 5, thus surpassing by a factor of 2 LM71 response.

Another well-known effect which will not be followed here due to lack of space is the fact that dynamic effects in indeterminate structures, especially continuous deck beams, are generally much lower than isostatic structures [Domínguez (2001)]. The main reason for this is that for simply supported bridges only one fundamental mode dominates the response, whereas in continuous beams several modes have an effective participation, which cancel each other partially under the moving loads.

5CASE STUDIES: ANALYSIS OF A "PERGOLA"-TYPE BRIDGE

This bridge is currently under construction, in the Madrid–Valencia high speed line. It is a "pergola" type bridge, this meaning that the main structural girders are oriented in the direction transversal to the train rather than longitudinally, in order to obtain the correct geometry for a crossing. The deck is composed of 64 precast beams with box section, joined by a top slab. Beam length ranges between 35.35 m and 40.62 m, with a separation between axes of 4.5 or 6 m., leading to 326.75 m. of total width. The train crosses in this direction with a skew angle of 15º. Taking into account the special characteristics of the viaduct (Figure 8), it has been considered convenient to perform dynamic calculations a three dimensional finite element model.

The model included 552 eigenmodes in order to consider all frequencies lower than 30 Hz, one of the modes is shown in Figure 8. The traffic actions correspond to the ten trains of the HSLM-A model. The calculations are performed for the range of velocities of 120-300 km/h every Δv=5 km/h. The highest velocity is 20% higher than the design velocity v_{design} = 250 km/h. The impact coefficient Φ_{real} was evaluated from the basis of displacement amplitudes. Further checks were done for accelerations and other ELS magnitudes. A total of 370 calculations were performed for this sweep of velocities. In order to employ a reasonable lapse of time a cluster of 24 PENTIUM machines (2.6 GHz and 512 Mb RAM) was used, running in parallel processes.

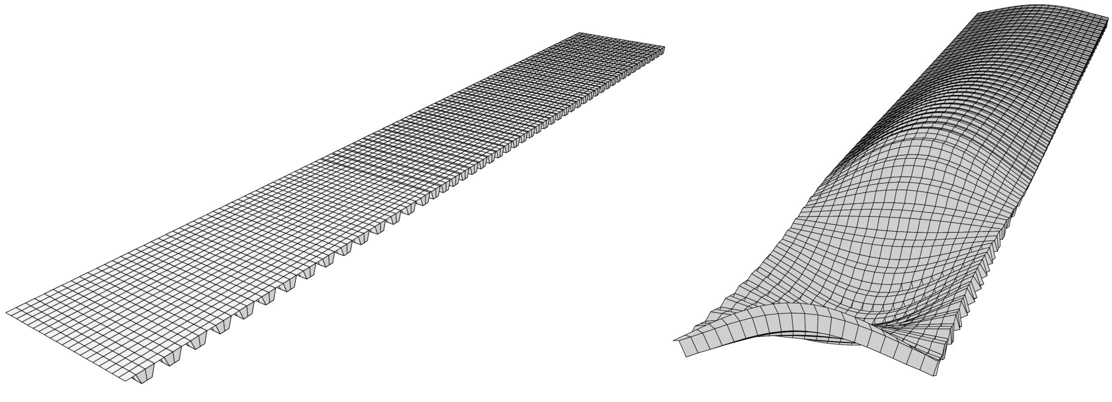

Figure 8. Finite element model of the "pergola" bridge and eigenmode #3, corresponding to frequency f_3 = 4.51 Hz

The results obtained were post-processed in order to obtain the maximum values of vertical displacement, vertical acceleration and "in plane" rotations. Figure 10 shows the impact coefficient Φ_{real} obtained for each velocity and each train. The values of Φ_{real} are always lower than unity, thus indicating that for the range of velocities considered dynamic effects will not surpass those of the static LM71 train.

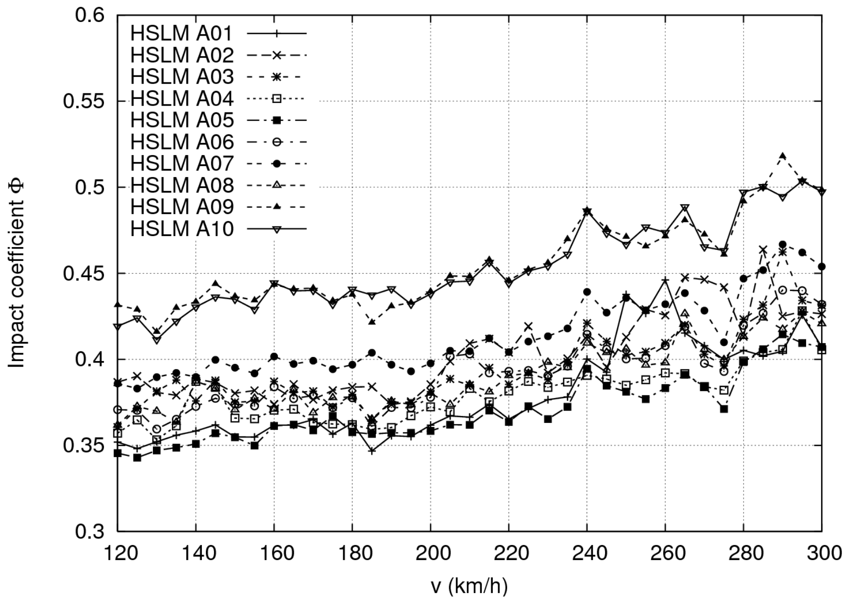

Figure 10. Pergola bridge. Envelopes of impact coefficient Φ_{real} computed from dynamic analysis at the midpoint of the track (the trains HSLM-A09 and HSLM-A10 produce the most critical values).

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