

DYNAMIC ANALYSIS OF HYPERSTATIC STRUCTURES UNDER HIGH SPEED TRAIN LOADS



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ABSTRACT

The construction of new high speed railway infrastructure in many European countries is actually a great civil engineering effort. One of the main design issues is related to the fact that trains at speeds above 220 km/h may induce resonance effects in structures as bridges, subway frames, etc. In consequence, the design of such structures requires dynamic calculations.

Several methods are available for the dynamic calculations of structures under high speed train loads. The simplest ones are based on sums of harmonic terms, providing upper bounds for the dynamic response, and with application limited to isostatic bridges. Alternatively, for statically indeterminate structures some models based on the direct time integration of the equations of motion are available. These methodologies suitable for hyperstatic structures, performed on full or reduced models, are discussed in this paper.

Additionally, some recent results for high speed traffic loads on bridges are presented. The object of these studies is diverse: simplified torsion analysis, a proposal of a simplified method for dynamic analysis of portal frames, and several practical results obtained for concrete bridges, composite steel-concrete bridges and pre-casted concrete bridges.

1. INTRODUCTION AND STATE OF THE ART

The construction of new transport infrastructure constitutes since the last years one of the major civil engineering efforts in many European countries. In Spain, the main part of these investments is dedicated to funding the construction of new railway lines for high speed trains,

being this item the most important too in other countries as France. These new railway lines are a very competitive alternative to other transport modes for medium distances. At this moment in Spain there are two high speed railways in operation: Madrid–Sevilla and Madrid–Lérida, pertaining the last one to the railway line Madrid–Barcelona–France. The GIF authority have in project or construction several railway lines as Córdoba–Málaga, Madrid–Valencia–Murcia and Madrid–Segovia–Valladolid, being this one considered in the frame of the international high speed railway system Portugal–Spain–Rest of Europe.

These activities have remarked out an important engineering aspect joined specifically to the design of high speed railway structures: the dynamical effects associated to the train moving loads, for which basic solutions have been described by Timoshenko and Young [18] and discussed fully in [6,7]. Also remarkable are the contributions performed in Spain by Alarcón [1,2].

Most engineering design codes for railway bridges have followed the approach of the dynamic factor proposed by UIC [19], which takes into account the dynamic effect of a single moving load and yields a maximum dynamic increment of $\varphi'=132\%$ for an ideal track without irregularities. The irregularities are taking into account with another parameter φ'' , leading to the dynamic factor $\Phi=1+\varphi'+\varphi''$. This approach cover the dynamic effects associated to a single moving load but does not include the possibility of resonant effects due to the periodicity of the moving loads, as this phenomenon does not appear for train speeds below 200 km/h. In this fact, for common structural eigenfrequencies and the distance between the axles in real trains, resonance is all too real for high speed railways, and its effects may surpass largely that of a single moving load.

Really, for velocities upper than 200 or 220 km/h and distances of axles between 13 m and 20 m. resonant effects may appear. An illustrative example showing experimental resonant measurements and the corresponding computational results is showed in [4] for the Spanish AVE train crossing at 219 km/h the Tagus bridge.

New European codes include the need for dynamic calculations covering resonant behaviour [5,11,14]. The calculation procedures foreseen in these codes are easy to apply to isostatic structures, as simply supported decks, because there is a fundamental eigenmode of oscillation. The simplest ones are those based on sums of harmonic terms, which provided bounds for the dynamic response [8]. Nevertheless for hyperstatic structures, like continuous deck bridges and portal frames for railway underpasses, more sophisticated methods of analysis are required because the participation of several eigenmodes in the dynamic response.

Generally these methods are based in the direct integration of the structural response for moving loads modelling the axles of the train. The calculations may be performed on full or reduced models with or without vehicle-bridge interaction. If vehicle-bridge interaction is considered the complexity of the model is increased and a major computational effort is required. Although these kind of simulation are very interesting from a research point of view they are not useful for standard design calculations except for unusual situations. Anyway, the structural model may be analysed either by the complete discretised system with N degrees of freedom and a time integrator like the Newmark method, or by a prior modal analysis and a posteriori time integration of the n significant eigenmodes.

2. MODELS BASED ON DIRECT INTEGRATION WITH MOVING LOADS

These class of methods are based on the direct time integration of the dynamic equations of the structure, under the actions corresponding to a train of moving loads of fixed values which values are representative of each axle of the train. The structural model may be analysed through the integration of the complete discrete system with N degrees of freedom, or through a reduction of the number of degrees of freedom via a previous modal analysis which reduces substantially the number of equations. This modal analysis can be performed through an approximate numerical procedure in order to obtain de eigenfrequencies and eigenmodes of vibration. This kind of procedures are available in the majority of finite element codes, and for certain cases of simple structures the spectral analysis can be achieved by analytical methods.

Finite element methods are applicable generally to arbitrary structures, even when non linear effects must be considered. A spatial semidiscretisation of the structure is performed into subdomains called “finite elements” leading a discrete N -d.o.f. system of equations:

$$\mathbf{M}\ddot{\mathbf{d}} + \mathbf{C}\dot{\mathbf{d}} + \mathbf{K}\mathbf{d} = \mathbf{f}(t) \quad (1)$$

where \mathbf{M} is the mass matrix, \mathbf{C} is the damping matrix, \mathbf{K} is the stiffness matrix, $\mathbf{f}(t)$ is the vector of external loads, and \mathbf{d} is the unknown vector of nodal displacements. In order to integrate in time this system of equations, a direct integration of the model solves the complete system (1) for each time step, and because the equations are generally coupled they must be solved simultaneously. This procedure is valid even when nonlinear effects must be taken into account. In such case the elastic internal forces and viscous damping should be replaced by a general nonlinear term $\mathbf{F}^{\text{int}}(\mathbf{d}, \dot{\mathbf{d}})$.

If as usual the structural behaviour is linear, a modal analysis can be performed resulting in another system with a remarkable reduction of degrees of freedom. In a first stage, the eigenvalue problem corresponding to the undamped system is performed solving the generalised eigenproblem corresponding to the structural discrete system:

$$\left(-\omega^2 \mathbf{M} + \mathbf{K}\right) \mathbf{a} = 0 \quad (2)$$

obtaining the more significant n eigenfrequencies ω_i $i = 1 \dots n$, and the corresponding normal vibration modes \mathbf{a}_n , being generally $n \ll N$. In a second stage, the vibration modes \mathbf{a}_i oscillating with their respective frequencies ω_i are integrated in time. With this procedure the equations are decouple, and the modal response of each mode is obtained from the dynamic equation of a system with a single degree of freedom.

The simplest procedure to define the train loads is applying load histories in each node. For time step t_i and an axle load F , a nodal load F_J is assigned to the node J if the axle is above an element that contains node J . The magnitude of F_J depends linearly on the distance from the axle to the node. This procedure is outlined in Figure 1 for a single load. This scheme has been implemented in the finite element program FEAP [17] both for the ten HSLM-A (High Speed Load Model) trains defined in the Eurocode [11,14] and for the seven real trains defined in the Spanish code IAPF [14]. All the results described in this paper have been obtained with

the methodology described in this section, using a direct time integration of the significant eigenmodes.

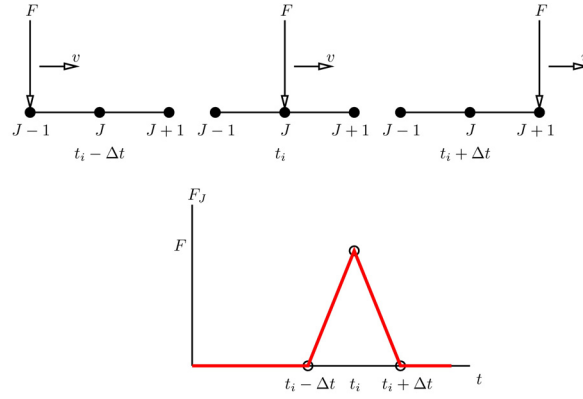


Figure 1: Nodal force time history definition for an axle load F moving at velocity v

3. DYNAMIC RESPONSE OF PORTAL FRAMES

3.1 Analytical methods

Although for an isostatic structure the eigenfrequencies and the eigenmodes can be obtained in analytical closed form [3], it is not possible in general to perform an analytical extraction of frequencies and vibration modes for statically redundant structures. Nevertheless, in some cases a closed form solution may be obtained such as intraslational portal frames and continuous beams with two or three spans [7]. Anyway, when possible, the dynamical calculations in closed form for hyperstatic structures are more complex than those available for simple supported beams. For example in the procedure corresponding to portal frames [13], the expression of the frequency for the first eigenmode (see Figure 2) is:

$$\omega_1 = \left(\frac{b}{l_d} \right) \sqrt{\frac{E_d I_d}{\bar{m}_d}} \quad (3)$$

where l_d is the span of the deck, $E_d I_d$ its bending stiffness and \bar{m}_d its mass per unit length. The parameter b is the solution of the following non-linear equation:

$$\frac{k_p (1 - \cosh(k_i b) \cos(k_i b))}{\cosh(k_i b) \sin(k_i b) - \sinh(k_i b) \cos(k_i b)} + \frac{1 - \cosh b \cos b}{(\cosh b + 1) \sin b - (\cos b + 1) \sinh b} = 0 \quad (4)$$

where:

$$k_p = 4 \sqrt{\frac{I_d^3 \bar{m}_d}{I_h^3 \bar{m}_h}}, \quad k_i = \frac{l_h}{l_d} 4 \sqrt{\frac{I_d \bar{m}_h}{I_h \bar{m}_d}} \quad (5)$$

and being l_h , I_h and \bar{m}_h the height, moment of inertia and mass per unit length of the vertical walls of the portal frame.

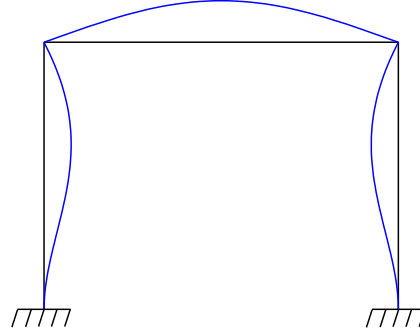


Figure 2: First vibration mode of a underpass modelled as a portal frame.

Once the vibration modes are known, it is necessary to integrate the dynamic equations. To this end, the basic solution is the response to a single moving load (see Figure 3). The differential equation for a load crossing the portal deck at a constant speed v , considering the i -th mode, is:

$$M_i \ddot{y}_i + 2\xi_i \omega_i M_i \dot{y}_i + \Omega_i^2 M_i y_i = F \langle \phi_i(vt) \rangle \quad (6)$$

where $\phi_i(x)$ is the modal shape, M_i the modal mass, ω_i the eigenfrequency, y_i the modal amplitude, ξ_i the critical damping fraction, and $\langle \phi(\cdot) \rangle$ is the bracket:

$$\langle \phi(x) \rangle = \begin{cases} \phi(x) & \text{if } 0 < x < l_d \\ 0 & \text{otherwise} \end{cases} \quad (7)$$

After obtaining the response for a single moving load, the response for a load train may be assembled as the superposition of the responses for each one of the point loads defining the train. In this case the differential equation corresponding to mode i is:

$$M_i \ddot{y}_i + 2\xi_i \omega_i M_i \dot{y}_i + \omega_i^2 M_i y_i = \sum_{j=1}^{n_{axes}} F_j \langle \phi_i(vt - d_j) \rangle \quad (8)$$

3.2 Simplified models

Portal frames are hyperstatic structures and therefore it is necessary a direct time integration method, including several eigenmodes, in order to evaluate their dynamic response. Besides, as they are often embedded in an embankment, they may have earth close to piers and even on the deck. The detailed analyses of the frame considering this fact is fairly complex, being

usual to consider the earth as added masses without structural effects. From a practical point of view, the conclusion is that portal frames are very simple structures, commonly encountered as railway underpasses, with low budget for calculations, but requiring a relatively great computation effort.

With this motivation the objective of the research work reported here, and detailed in a previous technical report [13], was to establish a adequate model based on an equivalent isostatic beam which dynamic response covers the corresponding one to a portal frame. This equivalent beam, with fictitious mass, length and stiffness, should have similar dynamic characteristics as the frame deck, but exhibiting a dynamic envelope similar or greater than the real frame one. Four representative portal frame underpasses constructed [16] in the Spanish high speed railway line Córdoba-Málaga, with l_d deck spans equal to 8.5, 8.7, 9.8 and 15 m, have been considered.

In order to establish the most adequate equivalent beam, four beams have been considered for each frame with deck span l_d . Therefore a total of 20 beams or frames structures have been analysed. The length l_{eq} considered for each one of these equivalent beams is l_d , $0.95l_d$, $0.90l_d$ and $0.85l_d$. The mass of the equivalent beams is defined with the same value that the mass of the deck: $\bar{m}_{eq} = \bar{m}_d l_d / l_{eq}$. The equivalent beam bending stiffness $(EI)_{eq}$ is adjusted such that the first eigenfrequencies are coincident:

$$(EI)_{eq} = \frac{\omega_{frame}^2 \bar{m}_{eq} l_{eq}^4}{\pi^4} \quad (9)$$

The same damping rate has been considered for the frame and beam models, and the contribution of earth cover has been neglected. This last assumption leads to conservative results because additional masses would decrease the maximum displacements and accelerations.

The computational analyses have been carried out in accord to the methodology described in previous section, using a enhanced version of the FEAP code [17]. The calculations are performed for a range of velocities of 120--420 km/h every $\Delta v=5$ km/h, considering the seven European high speed trains defined in the Spanish IAPF code [14]: AVE, EUROSTAR 373/1, ETR-Y, ICE-2, TALGO-AV, THALYS and VIRGIN.

The results obtained comprise displacements and accelerations. As representative examples, in Figures from 4 to 5 the envelopes of maxima acceleration and displacement obtained with the TALGO AV train for the four frames considered are showed.

The criterion for selection of the most appropriate beam model was the greatest similarity between its acceleration envelope and the frame one, since this is a critical aspect in this kind of structures. Attending to this criterion, the equivalent beam of equivalent length l_d was selected. Hence the properties of the equivalent isostatic beam are $l_{eq} = l_d$, $\bar{m}_{eq} = \bar{m}_d$, damping rate ξ_{eq} as the portal frame deck, and the bending stiffness $(EI)_{eq}$ obtained from expression (9). Other remarkable conclusions obtained from this study were:

- It is possible to define an equivalent isostatic beam for the dynamic analysis of usual portal frames in railroad underpasses, which conserves the form of envelopes of accelerations, displacements and impact coefficients.
- For non critical speeds (those velocities for which the results obtained are lower than the maximum ones) the results obtained for the equivalent beam are almost always greater than the results for the portal frame. Nevertheless it cannot be stated with absolute generality that the results obtained for an equivalent beam are always an envelope for the portal frame ones.

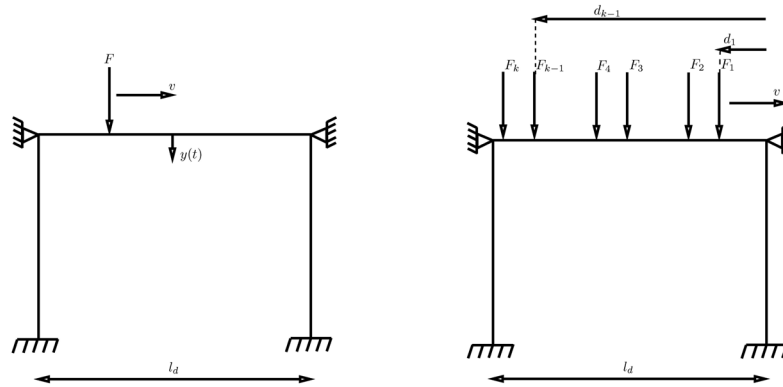


Figure 3: Response of a portal frame for a single moving load and for a load train.

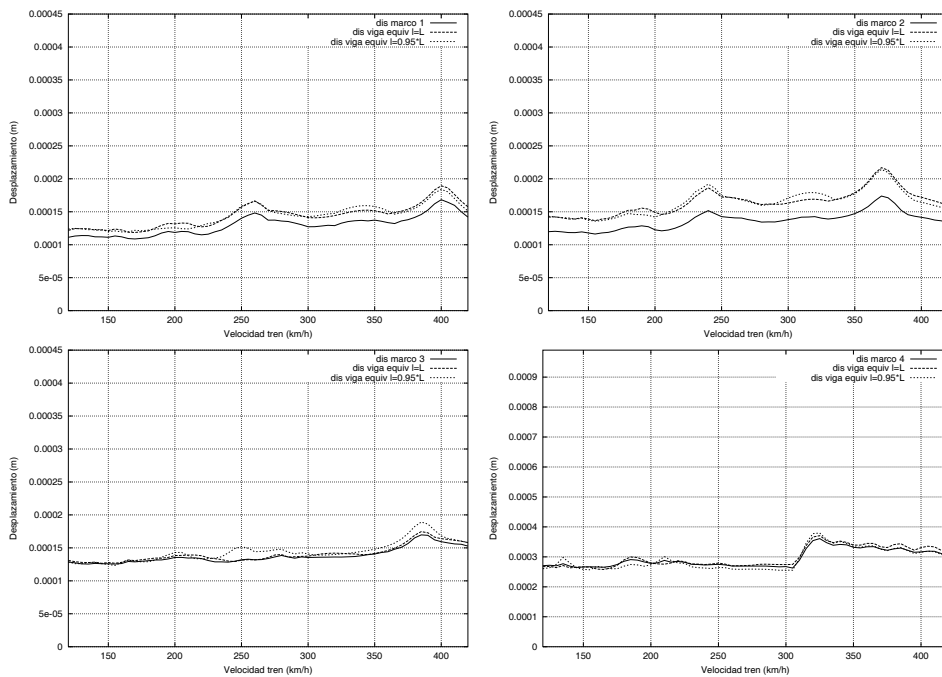


Figure 4: Envelopes of maximum displacements for portal frames and their equivalent beams of lengths l and $0.95l$ for TALGO AV train.

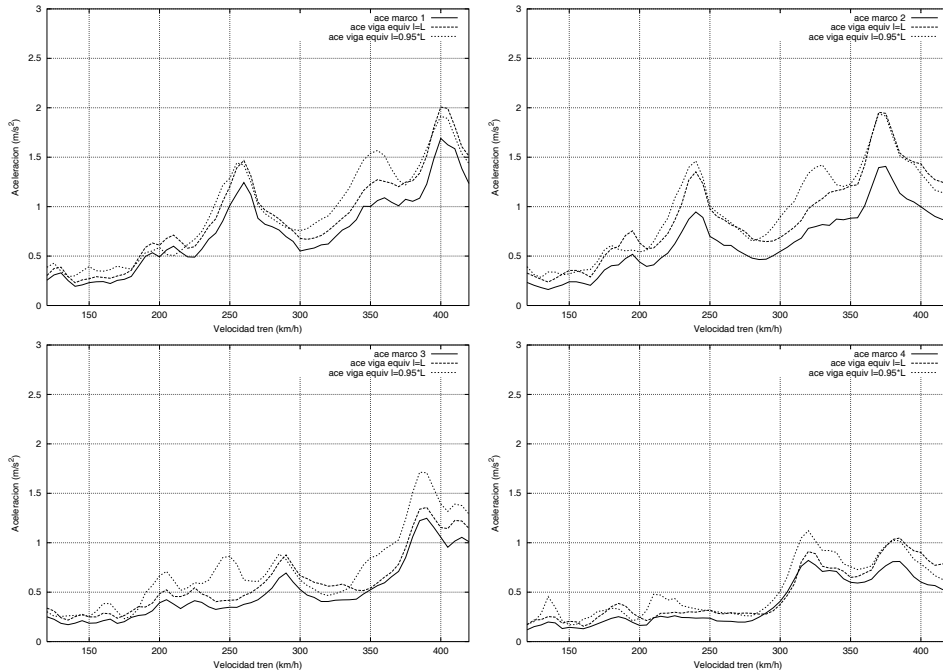


Figure 5: Envelopes of maximum accelerations for portal frames and their equivalent beams of lengths l and $0.95l$ for TALGO AV train.

4. EXAMPLES

4.1 Methodology for the analyses of hyperstatic bridges

In this section some dynamic analyses of representative real hyperstatic bridges in the Spanish high speed lines are showed. The methodology followed for the analyses, described in general in section 2, is explained in this subsection in the frame of the Spanish IAPF 2003 code [14] and the Eurocode [11].

The structure is modelled with three dimensional linear frame elements based on the Euler-Bernoulli beam theory, in order to consider in a coupled way the bending and torsion effects. At a first step a modal analyses of the structure is considered in order to know the eigenfrequencies and eigenmodes of the structure. In a second step the eigenmodes up to 30 Hz (in accord to the Spanish IAPF code [14]) are integrated in time. The loads considered are the corresponding to the European trains defined in IAPF 2003 code [14] or, preferably in order to asses the European interoperability, the HSLM-A (High Speed Load Model) trains defined in IAPF 2003 [14] and Eurocode [11]. The calculations are performed for a range of velocities from 120 km/h to 420 km/h (the maximum velocity in the analyses is a 20% higher than the maximum velocity of the bridge design [14]). The mass per unit length of the elements is defined taking into account the proper mass of the structure plus the corresponding to the rest of the permanent loads: track, ballast, sleepers, rails, etc. The damping is defined as Rayleigh

damping [3] considering a fraction of critical damping rate ξ equal to 2% for concrete structures and 0.5% for steel structures [14].

From the analyses described in the later paragraph the maximum movements and accelerations (for displacements and rotations) are obtained for the points of interest, for each velocity. In accord to the Spanish codes, the verification of the ELU (Ultimate Limit State) and ELS (Usage Limit State) states must be assured. The ELU one is verified through the definition of the impact coefficient [14]:

$$\Phi = \max_i \frac{\delta_{\text{din,real}}^i}{\delta_{\text{est,tipo}}^i} \quad (10)$$

In expression (10) the parameter $\delta_{\text{est,tipo}}^i$ is the static deflection obtained from train UIC71. The parameter $\delta_{\text{din,real}}^i$ is the “real” displacement obtained for the i train. This value is related with the ideal displacement $\delta_{\text{din,ideal}}^i$ which is obtained from the dynamic analysis considering the i train, and being the highest value for all the velocities considered. This relation is (for structures with good maintenance):

$$\delta_{\text{din,real}}^i = \delta_{\text{din,ideal}}^i (1 + \varphi' + 0.5\varphi'') \quad (10)$$

being φ' a coefficient related to the equivalent length of the bridge [14] and to its fundamental bending frequency, and φ'' a coefficient related to the tracking irregularities [9].

In relation with the dynamic effects, the verification of the ELS state comprises the checking of several conditions [15]:

- The value of accelerations must be lower than 0.35g for bridges with ballast, and lower than 0.5g for tracks without ballast.
- The value of torsion warping between two sections with a distance equal to 3 m is limited both in the Spanish IAPF code [14] and the Eurocode [11], being more restrictive the latter one.
- The bending rotation in the end supports and the relative bending rotation in the intermediate supports (for bridges with isostatic decks) have a limited value. This value is $\theta \leq 3.5 \cdot 10^{-3}$ for the transition tie bar–deck and $\theta \leq 5 \cdot 10^{-3}$ for the transition between adjacent decks. These values are modified for tracks without ballast and for bridges with only one track [14].
- In order to avoid possible transversal resonant effects in the vehicles, the eigenfrequency corresponding to transversal bending must be higher than 1.2 Hz.
- In order to guarantee the comfort of the passengers the vertical deformation of the bridges are limited depending on the span and the speed of the train.

In the following paragraphs several results obtained from the dynamical calculations of different types of real bridges are showed.

4.2 “In situ” concrete bridges

The Río Cabra viaduct is in Córdoba-Málaga High Speed line. It is a continuous deck viaduct with seventeen spans and hollow slab transversal section. The length of the spans are 20 m for the end ones and 25 m. For the intermediate ones, resulting in a total length of 415 m. In Figure 6 a image of this viaduct at the ending stage of construction is showed. In Figure 7 the envelopes of impact coefficient Φ , computed in accord to expression (10), and acceleration obtained from the dynamic calculations are showed, corresponding to the mid point of the side span. For the acceleration envelope the influence of the track irregularities is showed.

The data of this viaduct, together with the corresponding to one with box slab section, was used in order to analyse the simplified method proposed for torsion analysis by ERRI D214 committee [10]. The object of this study, detailed in [13], was to compare the cited simplified methodology with the application of a complete three dimensional analyses coupling the bending and torsion effects. To this end the following process was followed:

- Finite element dynamic analysis to evaluate the effects due to longitudinal bending
- Dynamic analysis to evaluate the effects due only to torsion, obtaining them by two different ways:
 1. From a bending analysis of an isostatic beam, applying the existing proportionality between the maximum acceleration curves associated to torsion and bending.
 2. From the finite element model used for torsion results.
- Finite element complete dynamic analysis coupling bending and torsion effects.



Figure 6: Río Cabra viaduct (Courtesy of NECSO).

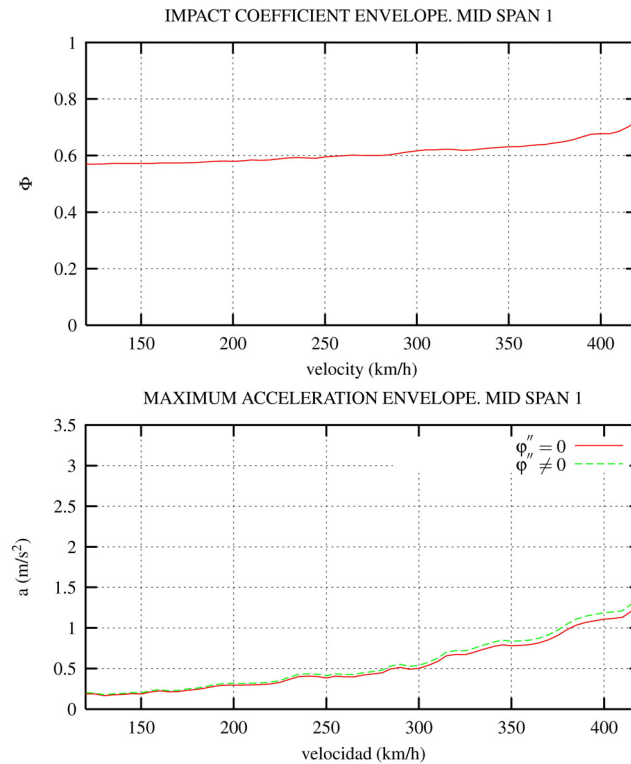


Figure 7: Impact coefficient and acceleration envelopes computed for Río Cabra viaduct.

A comparison criterion was established to relate the results obtained with the simplified method (direct sum of the absolute maximum of response obtained for the *only bending* analysis plus the maximum obtained from the *only torsion* analysis), with the maximum corresponding to the complete analysis and with the SRSS (Square Root of the Sum of Squares) combination. From this study, the following conclusions were extracted:

- The simplified method proposed in [10] is valid, but in certain situations is too conservative. The simplified method modified with the uses of SRSS combination is not always on the safety side, always the deviation with these examples was small.
- For structural sections with large torsional stiffness GJ_t , the deviation of results obtained with the simplified method and those obtained with the coupled model of bending-torsion is almost insignificant.
- For sections with reduced torsional stiffness (for example, hollow slabs or open sections) the deviation of results between simplified models and those of interaction bending-torsion is more significant (specially for isostatic beams).

4.3 Concrete–steel composite bridges

Typologies corresponding to concrete–steel composite bridges are usual in French high speed lines, but not so common in the Spanish ones. The Arroyo de las Piedras bridge, located in the

Córdoba–Málaga HS line and in construction stage at this moment, is the first composite bridge in a Spanish high speed line. It has a total length of 1208 m composed of 20 spans. The intermediate spans are 63.5 m in length, and the side spans are 50.4 m and 35 m. In Figure 8 a longitudinal profile of the bridge is showed.

One of the main characteristics of composite bridges, from a dynamical point of view, is the low values of eigenfrequencies associated to the torsional oscillation modes. This fact leads to the participation of a high number of eigenmodes in the computations. Besides, the values for bending deflections and torsional deflections of the deck are similar being necessary to compute both effects in a coupled way.

For the Arroyo de las Piedras viaduct several dynamic analyses have been carried out in order to analyse the influence on different design parameters. The seven real European trains defined in IAPF code [14] have been considered, for a range of velocities from 120 km/h to 420 km/h every 5 km/h. The computational analyses were performed using a modified version of FEAP code [17]. The following aspects were considered in the sensitivity analyses carried out:

1. Comparison of results obtained considering a damping ratio $\xi=0.5\%$ (specified in [14] for composite and metallic structures calculations) with those obtained considering $\xi=1\%$.
2. Comparison of results obtained considering encastred torsion supports and elastic torsion supports.
3. Sensitivity to the results corresponding to the evaluation of bending inertia under the hypothesis of total cracked section in the part of the deck with negative bending moments.
4. Variation of the results obtained considering open sections for the torsional response of the structure.

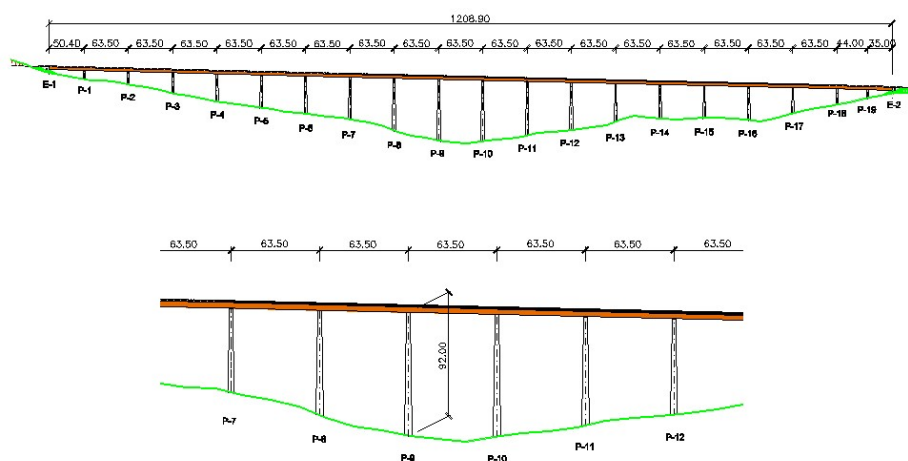


Figure 8: Arroyo de Las Piedras viaduct. Longitudinal profile (Courtesy of IDEAM).

The results obtained from these analyses are detailed in [12], being reported the following conclusions:

1. The maximum vertical accelerations increase a 33.3% from considering a damping fraction $\xi=1\%$ to $\xi=0.5\%$. For both values the Usage Limit State requirements are verified.
2. The values computed with and without encastred torsional supports are very similar.
3. The results corresponding to the hypothesis of cracked sections for negative bending moments don't show important differences with the standard ones.
4. The main differences appear considering open sections. The increment of the impact coefficient is important although it does not modify the design parameters of the viaduct. Nevertheless the values obtained for maximum accelerations are not admissible as can be seen in Figure 9.

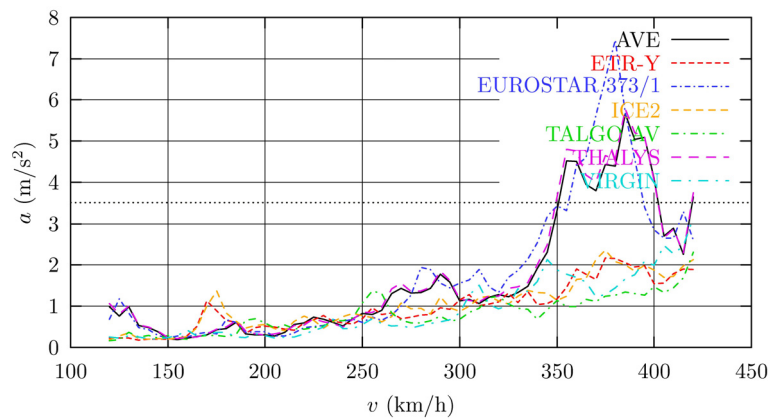


Figure 9: Envelope of maximum accelerations for open sections. Mid point of 35 m side span.

4.4 Precasted concrete bridges

About precasted concrete bridges, two typologies have been analysed: attached double box section bridges and double box section bridges (see Figure 10).

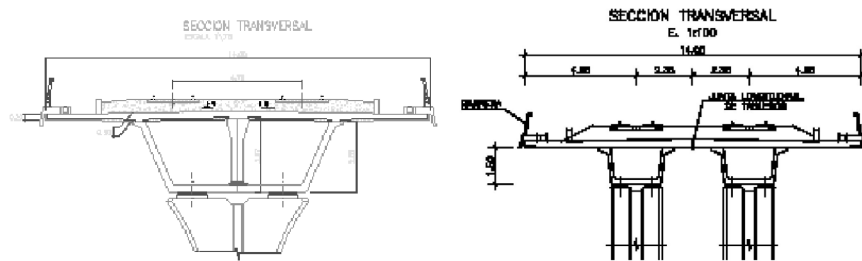


Figure 10: Attached double box and double box beam sections of precasted bridges (Courtesy of PRAINSA).

The Río Moros viaduct, located in the Spanish Valladolid–Segovia high speed railway line, corresponds to the attached double box section typology. This viaduct has a total length of 475 m, with 11 continuous spans. The two side spans are 35 m and the intermediate spans are 45 m, having non-uniform side near the piers. Figure 11 shows a construction stage image and the finished viaduct.



Figure 11: Río Moros viaduct (Courtesy of PRAINSA).

The dynamic analyses were carried out using a finite element model with 3D beam elements based on the Bernoulli theory, coupling the bending and torsion effects. In accord to the Spanish IAPF code [14] seventy three modes of oscillation with eigenfrequencies lower than 30 Hz were considered for the computational analyses. A range of velocities from 120 km/h. to 420 km/h every 5 km/h was considered for the analyses. The actions applied were the corresponding to the ten trains of the HSLM-A model (High Speed Load Model A) defined in [11,14], which are dynamic envelopes of the effects of the possible real trains. This load model assures the interoperability criteria, opening the possibility of interconnection with other European high speed lines.

For the double box section bridges two types can be distinguished corresponding to deck with and without longitudinal joint (see Figure 12). Two precasted viaducts of this type have been analysed under high speed train loads, describing the main aspects of the results in the following paragraphs.



Figure 12: Double box section bridge (Courtesy of PRAINSA).

The V2 viaduct is a double box section bridge with longitudinal joint in its deck. It will be in the Spanish Madrid–Cuenca–Valencia High Speed line. At this moment this HS railway line is in project stage. This viaduct has three spans. The side spans are 16.5 m and the main span is 22.5 m. The calculations are carried out with one track loaded, taking into account the assessment of torsion effects associated to a minimum excentricity due to possible transversal displacements of the track [14]. The loads considered was the corresponding to the ten universal trains defined in the HSLM-A model [14,11], in the range of velocities from 120 km/h to 420 km/h. The results were compared with those obtained for the seven European real trains. In Figures 13 and 14 are showed the envelopes of impact coefficient (see expression (10)) and accelerations in the mid point of the main span, respectively.

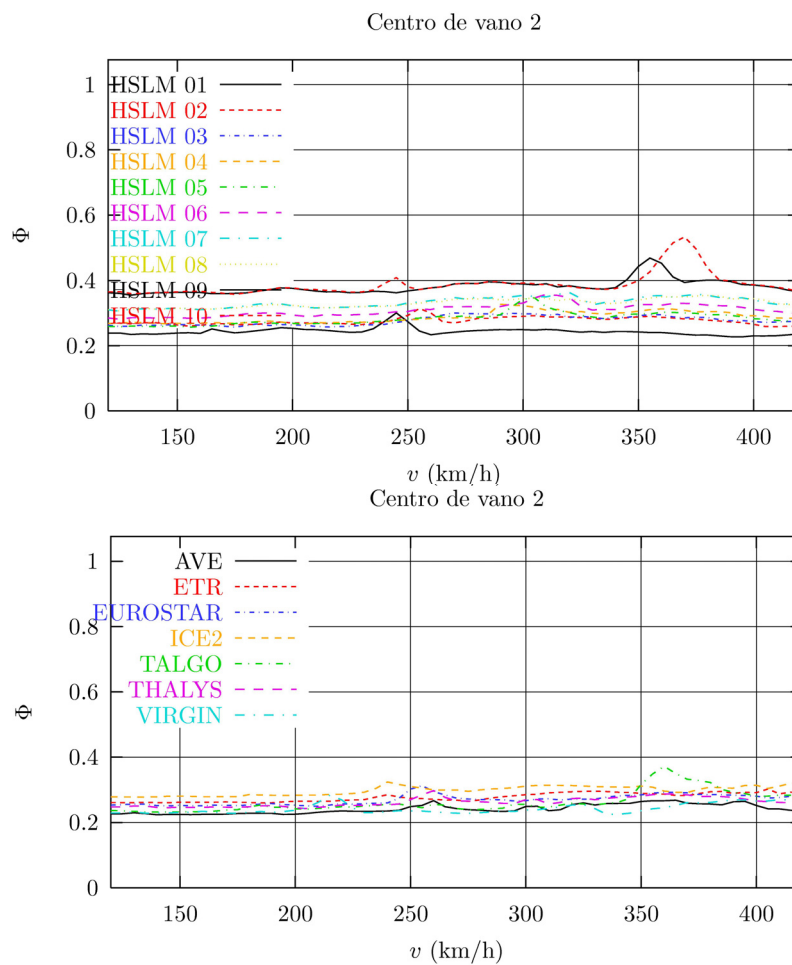


Figure 13: V2 viaduct. Impact coefficient envelopes for universal trains HSLM-A and European real trains

It can be observed that, although designed to be dynamical envelopes for isostatic structures, the universal trains are envelopes for the impact coefficient Φ in this structure too.

Nevertheless, it seems that the acceleration obtained for the Virgin train at 220 km/h is higher than those obtained for the universal ones. This aspect is not relevant because the values obtained with the Virgin and *HSLM-07* train are similar, and because this intermediate peak is not determinant for design. Anyway, it is interesting to remark this fact because it is not proved the envelope capabilities of universal trains for statically indeterminate structures.

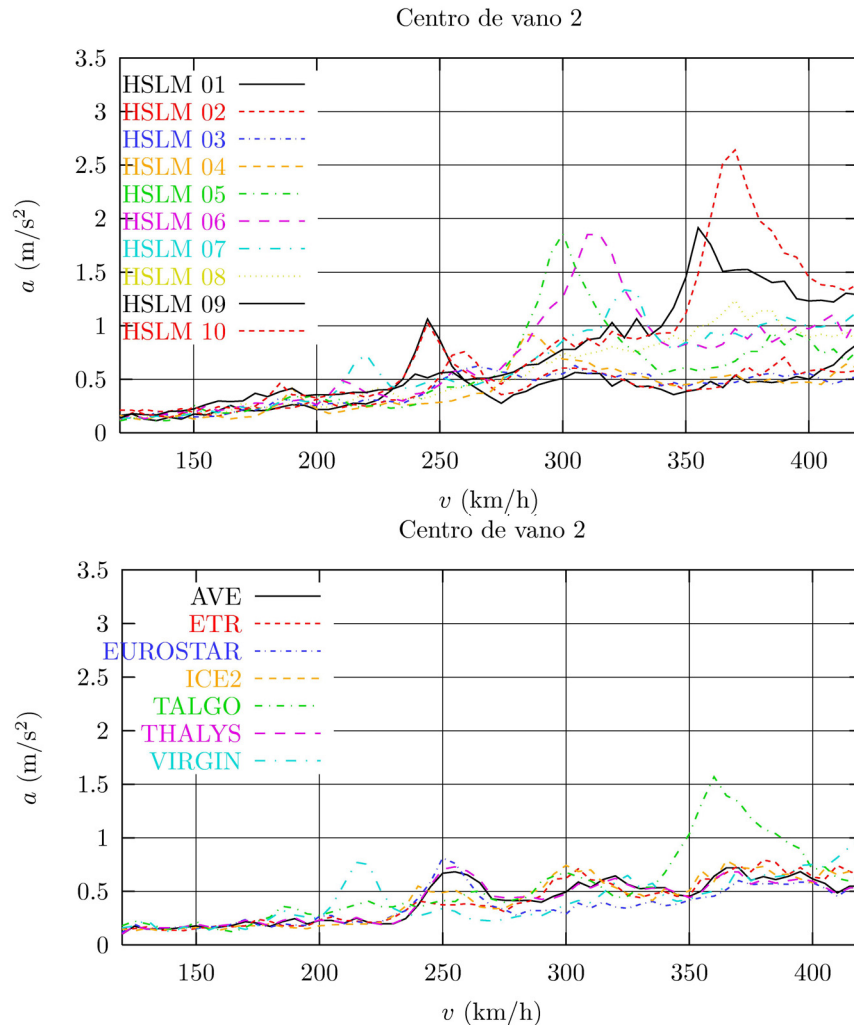


Figure 14: V2 viaduct. Maximum acceleration envelopes for universal trains HSLM-A and European real trains.

Within the group of bridges without longitudinal joint in the deck is the Río Milanillos viaduct. It is located in the Segovia–Garcillán HS line, and at this moment is in project stage. Because its two box beams are separated 7 m and joined with a relatively flexible deck, the hypothesis corresponding to the infinity stiffness of the transversal section may be not correct for dynamical computations: the torsional eigenfrequency would be higher and in consequence the

participation of this mode could be undervalued. In consequence, an equivalent torsional stiffness was assessed using to this end a 3D detailed finite element model. The FEM mesh has 1536 shell elements, resulting in a model with 10000 equations approximately. In order to impose a torsion moment two vertical point loads were defined in the axles of the tracks, and the warping of the supports and mid span was disabled. Figure 15 shows in the upper part the mesh with the contour of vertical displacements obtained and the point loads. In the lower part of this figure the deformed mesh is showed. Using an important magnification factor it can be shown that the hypothesis corresponding to the rigid solid movement of the transversal section is not suitable.

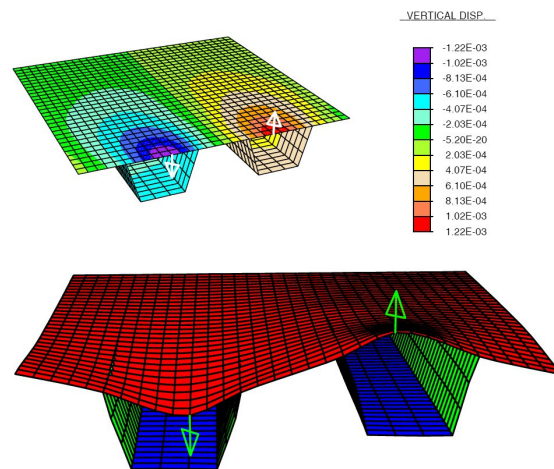


Figure 15: 3D FEM model of the Río Milanillos viaduct and deformed mesh with a magnification factor equal to 1000.

6. CONCLUSIONS

Nowadays the dynamic analysis of hyperstatic structures under high speed train loads is a requirement of the European design codes for railway bridges, because of the real possibility of resonance. To this end, several models based on direct integration with moving loads have been described in this paper. These models have been applied to some representative design problems, presenting a simplified methodology for the analysis of railway underpasses modelled as portal frames. Finally, some remarkable aspects corresponding to several dynamic analyses performed on real structures in the Spanish high speed railway lines have been discussed.

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