Double composite hollow section for high speed rail way bridges. Study of dynamic behavior.

Francisco Tolentino

Sptember 2011

Department of Civil Engineering, Instituto Superior Técnico, Universidade Técnica de Lisboa, Portugal

Absract

The main objective of this dissertation is to study an alternative constructive solution to the traditional high speed rail bridges (pre-stressed concrete), with main focus on the analysis of dynamic behavior. This study involved a systematization of the main regulatory aspects to be considered in the design of railway bridges, in particular as regards the quantification of actions related to traffic and the associated dynamic effects, the checks to be made taking into account the aspects related to structural safety (dynamic amplifications), with the road safety and passenger comfort. For the methodologies for dynamic analysis of train-bridge system has been presented by way of example methods of numerical analysis, simplified and empirical. Among the methods of numerical analysis methods have been described for no train-bridge interaction. For that was described as its implementation and validation in commercial software SAP2000 v14.2. In the main application of this dissertation was carried out to study the dynamic behavior of a railway bridge into the European TGV, it's a composite bridge, under the action of high speed rail traffic. The dynamic tests were made for the passage of several trains in circulation into the European high-speed trains for the High Speed Load Model A (HSLM-A).The response of the bridge was assessed in terms of structural safety, the safety of track and passenger comfort. The nature and relevance of dynamic effects in railway bridges is discussed, particularly with respect to high-speed traffic actions. The impact phenomenon due to a moving load and the resonance for a train of loads is discussed. Following the available methods for dynamic analysis are discussed, including their consideration in the current engineering codes. Next some basic concepts which condition the dynamic response are discussed, including those resulting from the different types of trains as well as those related to the bridges themselves. Finally some representative applications to practical design situations are presented and discussed.

KEYWORDS: Dynamics of railway bridges, high-speed railways, dynamic signature, double composite action.

1. Introduction

The Portuguese high-speed network is currently in a primordial stage. The network building will be left in charge of public and private partnerships, responsible for the financial and technical management of the infra-structure. RAVE, as the regulating entity of the Portuguese high speed network must give out some project guidelines in order to better evaluate and control its performance.

The current work is based on this specific task, which its sole objective is to clearly and concisely expose the main problems and design procedures related to the works of art, ones to be built or re-used in the high-speed network.

2. Double composite girders vs Pres-stressed girders

The use of precast elements was associated for a long time to a poor construction and, thus, it wasn't seen as a good choice. However, the construction of viaducts and bridges with precast/pre-stressed girders has some advantages that leads to increasing solutions in this field. The good quality of pieces and reduction of work time are some of the advantages of this technique. The construction of bridges with precast/pre-stressed girders has values for slenderness that depends on his type. Railway bridges usually present values smaller than 16 [6] while road bridges present values smaller than 20. are usually adopted “I” and “U” shape girders. “U” girders present more flexural stiffness due to his inferior flange and better torsion behavior after casting the slab.

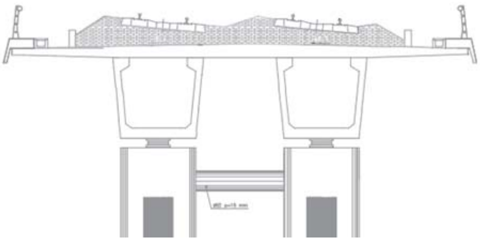

Fig. 1 – Cross section of pre-casted concrete section [1]

For the negative aspects of this solution, compared to a conventional mixed solution, it is noted the increase in dead weight of the structure that could burden the support elements, the pillars and their foundations. The total cost of the solution may be possible as another important drawback. As shown, this work aims to study an alternative to these traditional methods.

Fig. 2 – Cross section of double composite section

Over the years, a large number of bridges and overpasses have been built with pre stressed concrete decks. At the same time, bridges with all steel decks have always been very well accepted as a solution, for railway decks and for very long suspension and cable stayed decks. Over the past 50 years, the number of bridges built with composite steel concrete decks, composed of a concrete slab interacting with a steel structure, has been gradually increasing in several countries. The combined use of steel and concrete in bridge decks aims to take advantage of the best features of each material. This means combining the high compressive strength of concrete and the good tensile performance of steel. The choice between a pre-stressed concrete deck or a composite steel concrete deck is influenced by factors such as spans, the building process, geotechnical conditions, economic aspects (construction and maintenance costs), construction period, aesthetics and landscaping. A composite steel concrete bridge generally has the following main advantages over a pre-stressed concrete bridge.

- Lower self-weigth of deck;
- Simpler construction methods;
- Faster rate of construction.

Some important advantages of this method.

- Higher initial cost;
- Higher maintenance cost;
- Superior building technology.

The double composite action seeks to improve the structural behavior of a steel concrete composite bridge, particularly in the sections over the interior supports. In these sections, the applied bending moments (negative by convention) produce stress in the materials, steel and concrete, such that they are structurally less efficient. In fact, the negative bending moments cause tensile stress in the upper concrete slab, which usually tends to crack prematurely under the action of dead loads; these forces need be absorbed by the reinforcement and the upper steel flange. However, the compression that is developed in the lower steel flange is usually more of a problem. In conventional composite solutions, this compression can lead to phenomena of lateral instability, which require the use of very thick flanges and / or stiffeners and bracing in the deck sections near the internal supports.

In keeping with the concept of placing each material where it is more efficient in structural terms, the concept of double composite action is the addition of a second concrete slab at the bottom flange of the deck cross section near the supports, working with the bottom steel flange to resist the negative bending moments. At the same time the lower concrete slab gives the required stability to the bottom flange. It is intended, with this solution, to achieve the following improvements, relative to a conventional composite solution.

A double composite bridge generally has the following main advantages over a conventional composite bridge.

- Increased resistance to negative bending moments;
- Reduction of bottom steel flange thickness;
- Reduction (or elimination) of instability at the bottom flange and web of the steel section;
- Fewer stiffeners needed;
- Better torsional behavior;
- Better fatigue behavior;
- Less deck deformation.

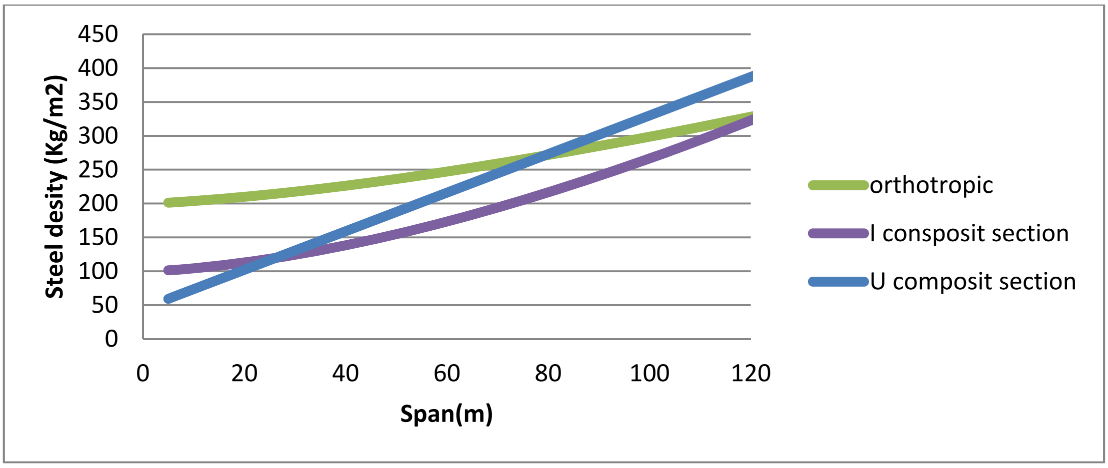

Fig. 3 - – Comparison between different structural sections

It is important to note that these figure concern to sections with conventional simple composite sections , for double composite sections the quantity of steel values tend to decrease, because this is exactly one of the main objectives of this type of structure.

3. Current documentation

The construction of a high-speed railway infrastructure is not new in Europe. Parallel to this experience, a set of norms and recommendations have already been written in order to alert the designer to the actions and dimensioning conditions. Among others, the TSI, CEN, CENELEC and UIC rules are also included.

TSI – Technical Specifications for Interoperability – these documents, of regulatory character, have the purpose of guiding the technical options of the competent authorities and to establish a common base. To every subsystem there is a corresponding article of the L245 publication of the European Communities Journal. About the bridges and viaducts, document L245/143 [5] demands that the evaluation of the infrastructural system must verify the safety of the vertical loads (static and dynamic effects), the transversal horizontal loads as well as the longitudinal loads throughout the project and the submission tests before putting into service.

CEN rules – European Committee for Standardization – these documents, of recommendatory status, surpass the civil engineering domain. In fact, among countless CEN norms, only some actually concern the matter of high- speed railway traffic. So the emphasizing rules would be the EN 199x specifically the EN 1991-2 [2], which defines the traffic actions on bridges.

The circulation of high speed railway vehicles provokes dynamic effects on bridges that had been studied for a long time. The increase of velocity that witnessed nowadays, has originated vertical displacements and accelerations corresponding to resonance phenomena that tends to appear for speeds higher than 200km/h

4. Methods for dynamic analysis

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): Φ.LM71 $\longrightarrow$ Φ.Esta,LM71 > Edin,real.

Note that the 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).

The 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], as well as some other conditions such as bounds for the fundamental frequency f0. Otherwise, dynamic calculations must be carried out.

4.1 Analysis vehicles

The load models of high-speed train consists of two universal HSLM-A and HSLM-B, and the first is indicated for the analysis of spans more than 7 meters and is divided into 10 vehicles and the second for spans less than 7 meters.

Family of 10 (fictitious) articulated trains, which are dynamic envelope of the real (and foreseen) high speed trains:

Table 1 – Family HSLM-A [4]

Parameter	HSLM-A
Type	articulated
Total length	~ 400m
Coach length D	18m – 27m
Axle point load	170KN – 210KN
Bogie axle spacing D	2.0 m – 3,5m
Head and tail locomotives	yes

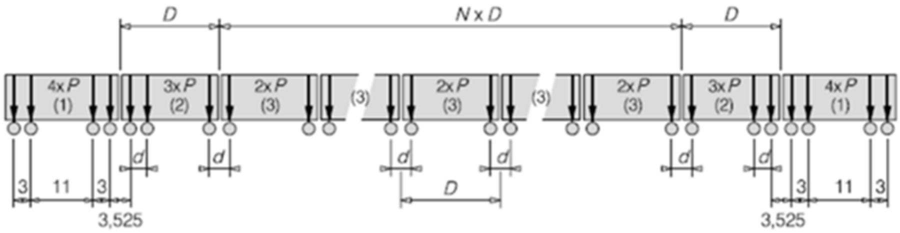

Fig. 4 – HSLM-A model [2]

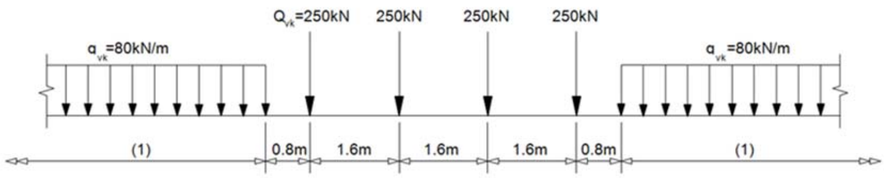

Fig. 5 – LM71 with characteristic values [2]

5. Analytic methods

Moving load models: Basis

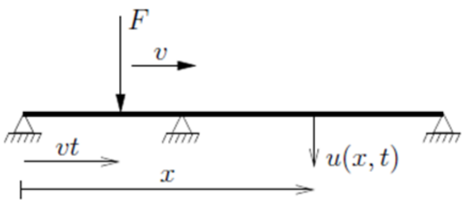

Fig. 6 – Moving load [4]

Dynamic equation of elastic:

$$\rho\ddot{u} = (EIu")" = p(x,t) \qquad 1$$

Hypotheses:

- Simple example: Bernoulli beam, straight, no torsion
- Constant loads, point or distributed loads

Solution Procedures

- Discretization (e.g. finite elements) and direct time integration of complete model
- Computation of normal modes of vibration and time integration of selected modes
- Numerical computer models or analytical calculations

<u>Moving load models: modal analysis</u>

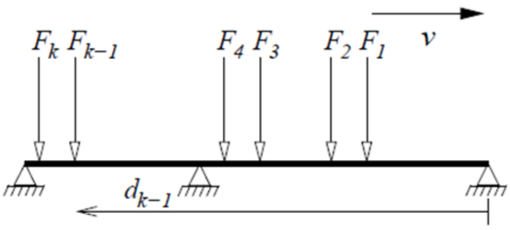

Fig. 7 – Multiple moving loads [4]

- Modal analysis: $\{\omega_i, \phi_i\}$ $(only\ modes\ \omega_i < 30Hz)$[2]
- Decoupled equations, one per mode $\phi_i(x)$

$$M_i\ddot{y}_i + 2\xi_i\omega_i M_i\dot{y}_i + \omega_i{}^2 M_i y_i = \sum_{k=1}^{n} F_k\langle\phi_i(vt - d_k)\rangle \qquad 2$$

$$\langle\phi_i(x)\rangle = \begin{cases} \phi(x) & if\ 0 < x < l \\ 0 & otherwise \end{cases} \qquad 3$$

6. Resonance phenomena

The resonance effect of a vehicle-structure system can be divided in the following categories according with the mechanisms that generate them:

- Bridge resonance induced by periodically loading of moving load series. For the resonance analysis on the bridge, the series of loads must consist not only in the vertical forces on the axles from the weight of the train, but also the lateral forces transmitted in the contact points due to the centrifugal acceleration or wind pressure acting on the vehicle. To this type of resonance there is a corresponding critical speed, given to us by:

$$V_{cr} = \frac{D \times f_0}{i} \qquad i = 1,2,3 \ldots. \qquad 4$$

7. Bridge descriptions

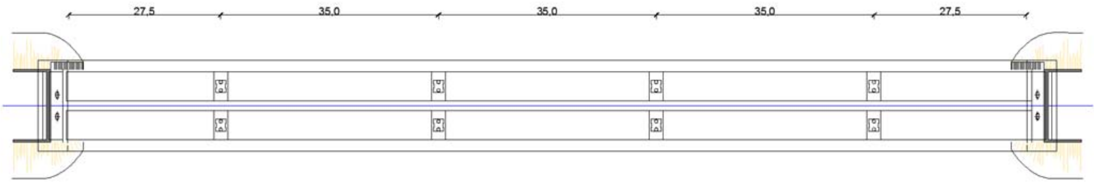

Fig. 8 - The longitudinal layout of the bridge has three spans of 27,5 + 35 + 35 + 35 + 27,5.

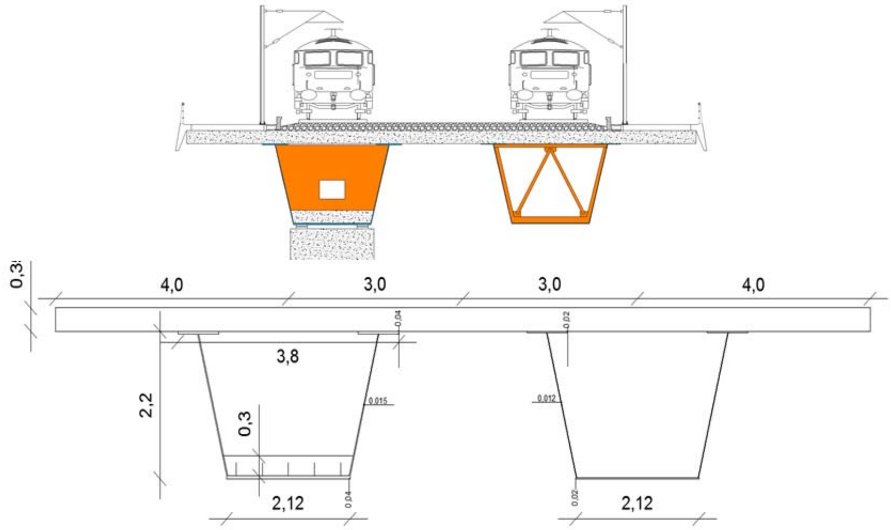

Fig. 9 - The transverse layout (m)

- The concrete is class C30/37, the reinforcing steel is A500NR and the steel is S355 NL.
- The superstructure-infrastructure connection is made through neoprene bearings with free rotation and longitudinal displacement (except for one of the right abutment).
- In supports will be adopted steel diaphragms with a thickness of 1.2 cm with a hollow in the center of 0.8 x0, 8 (m). Over the span were adopted diaphragms to prevent buckling and to introduce more torsional stiffness in the launching process. These diaphragms are arranged with a spacing of 7.0 m.

8. Analysis of a high speed railway viaduct

8.1 Static verification

The strength and in-service behavior was checked and presented on this chapter. The actions and safety verifications were based on current regulations and the Eurocodes.

I. Verification of stresses in serviceability limit state:

Table 2 – Verification SLS

	Fiber	Stress SLS (MPa)	Stress limit EC0 (MPa)
Span	Top flage	98,3	355
	Bottom flange	248,4	355
	Top concrete slab	11,3	16
Support	Top flage	-185,8	355
	Bottom flange	-149,3	355
	Top rebar	-137,2	435
	Bottom rebar	-117,7	435
	Bottom slab concrete	-12,9	16

II. Vertical deformation of the deck

- The limit for traffic safety according to EN 1991.2 is L/600 (35/600=5,8cm)
- The limit SLS checks according to EN 1991.2 is L/1750x0,9 (35/1575=2,2cm)

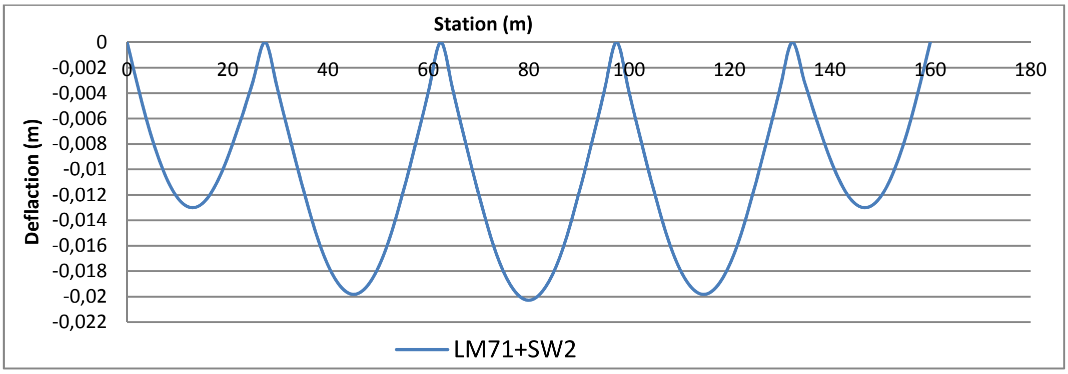

Fig. 10 – Verification for for traffic safety

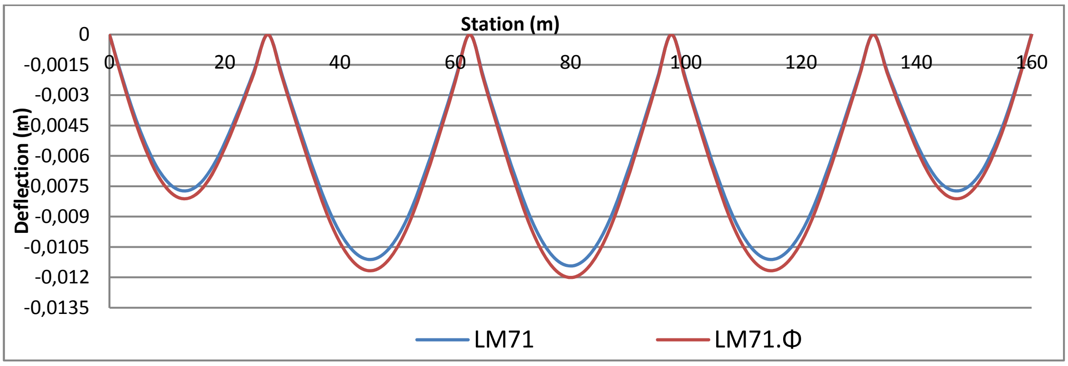

Fig. 11 – Verification for SLS

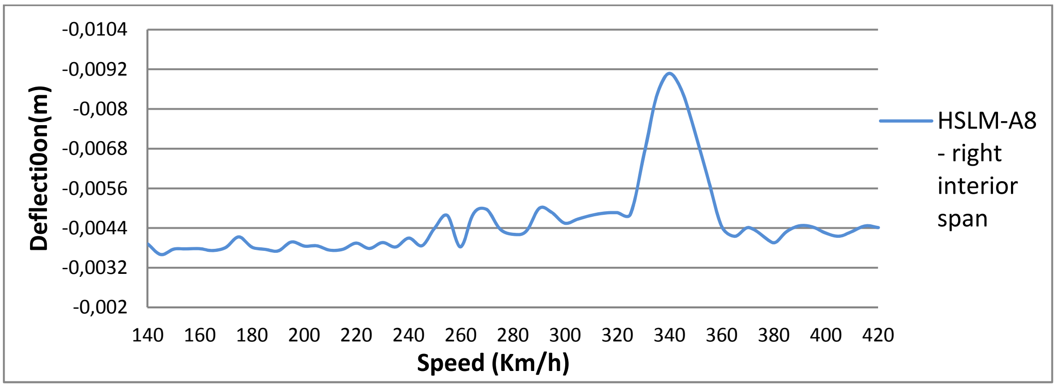

Fig. 12 – Maximum deflections for HSL-A8 (v=335,16 Km/h)

III. Verification of ultimate limit state:

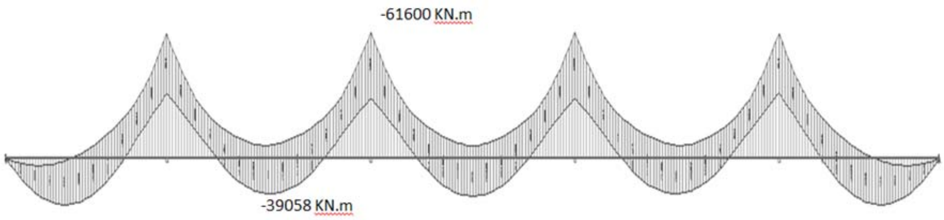

Fig. 13 – Ultimate limit estate M3

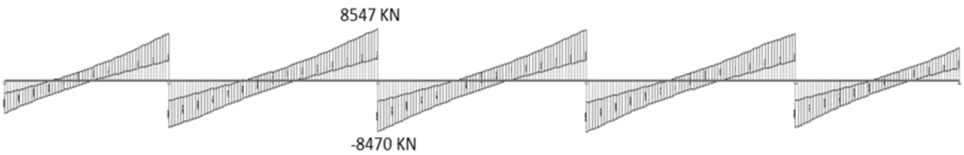

Fig. 14 – Ultimate limit state V2

Table 3 – Verification Msd

	Span	Interior pier
Msd	39058	-61600
Mrd	63878	-92000
Mrd/Msd	1,63	1,5

Table 4 - Verification Vsd

	Interior pier
Vsd	8547
Vrd	13383
Vrd/Vsd	1,57

8.2 Dynamic verifications

To realize the dynamic analyses of the viaduct, two finite elements models in SAP2000 were considered. The first model uses shell elements – shell model, while the second one uses frame elements.

Models are also analyzed by comparing their modal and dynamic behavior. The young's modulus is affected by a factor related to the fact that the action of trains circulating over the bridge is a fast load [3]. This way, to the slab is adopted 34,1GPa.

The modal analysis was made only to compare the frequencies of both models.

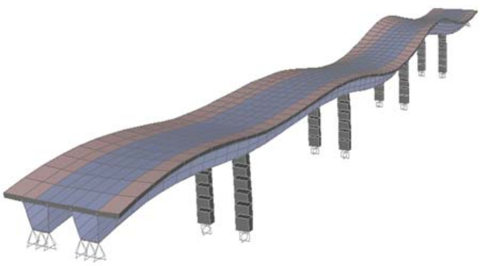

Fig. 15 – Shell model

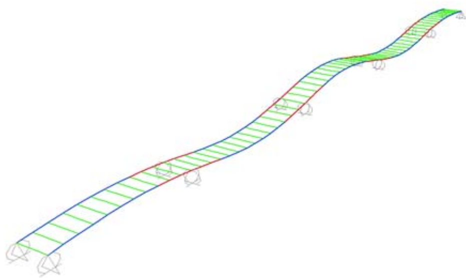

Fig. 16 – Frame model

Tabela 1 – Frame model

Mode	Freq. Hz	Longitudinal	Lateral	Vertical
1	0,796	0,00%	79,49%	0,00%
2	2,301	0,00%	0,67%	0,00%
3	2,960	0,00%	0,00%	3,02%
4	3,760	0,00%	0,00%	0,00%
5	4,328	0,00%	10,93%	0,00%
6	4,386	82,58%	0,00%	0,00%
7	4,802	0,00%	0,00%	4,0%
8	5,015	0,00%	0,00%	0,00%
9	5,418	0,00%	0,00%	0,00%
10	5,832	0,00%	0,00%	0,00%
11	5,989	0,00%	0,00%	0,00%
12	6,238	0,00%	0,00%	77,56%

Tabela 2 – Shell model

Mode	Freq. Hz	Longitudinal	Lateral	Vertical
1	0,814	0,00%	81,14%	0,00%
2	2,868	0,00%	0,04%	0,00%
3	3,060	36,88%	0,00%	3,60%
4	3,413	35,95%	0,00%	4,00%
5	3,527	0,00%	0,00%	0,00%
6	4,130	0,00%	0,00%	0,00%
7	4,177	9,98%	0,00%	0,57%
8	4,883	0,00%	0,04%	0,00%
9	5,074	2,31%	0,00%	3,97%
10	5,536	0,00%	0,22%	0,00%
11	5,869	0,32%	0,00%	20,48%
12	5,929	0,00%	6,14%	0,00%

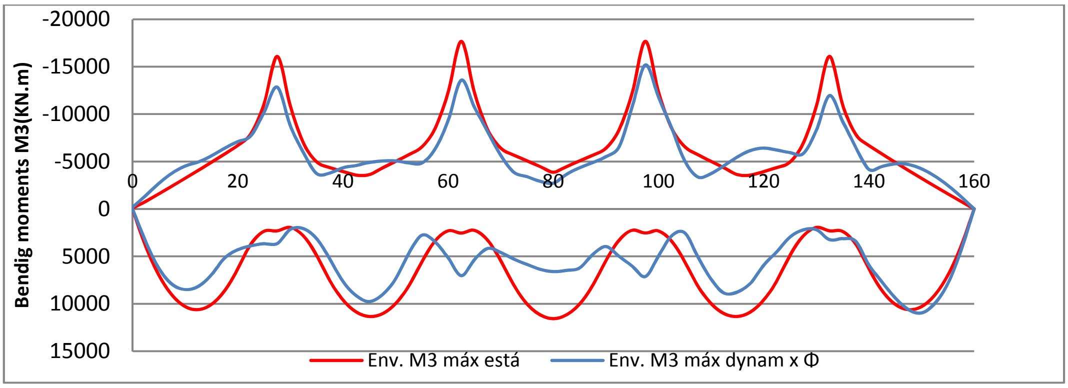

Fig. 17 – Envelope for M3

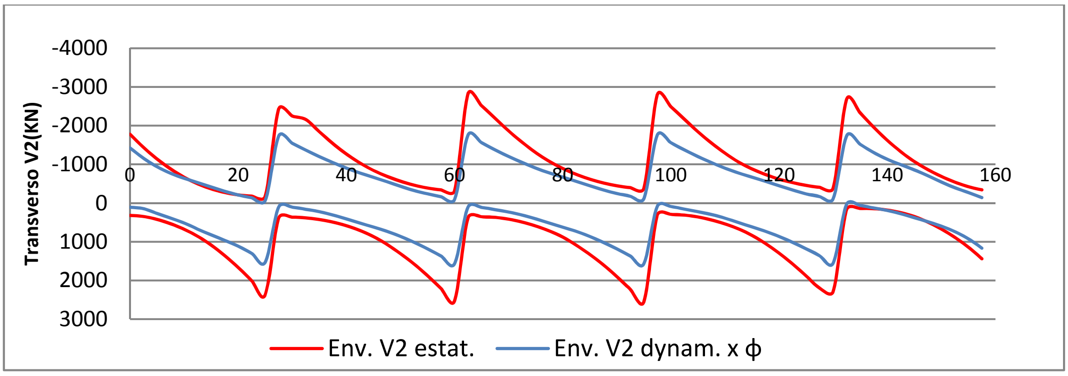

Fig. 18 – Envelope for V2

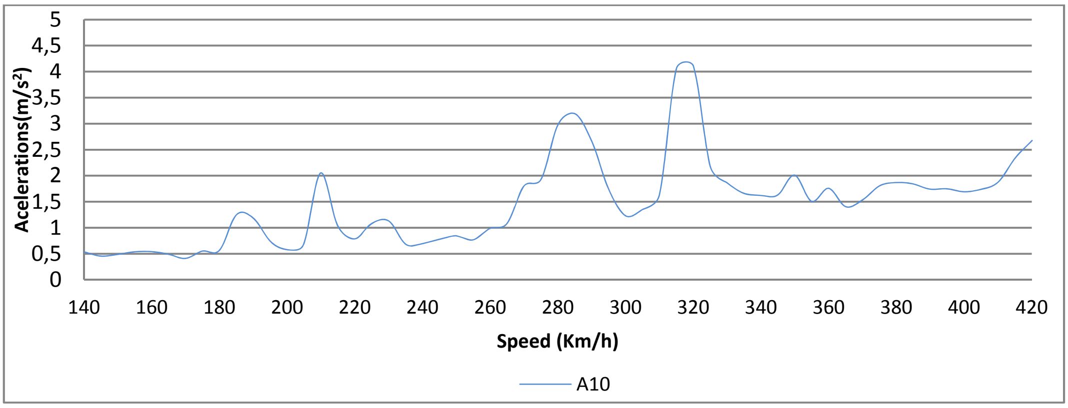

Fig. 19 – Maximum acceleration of HSLM-A10 v=310 Km/h (rigidity increased)

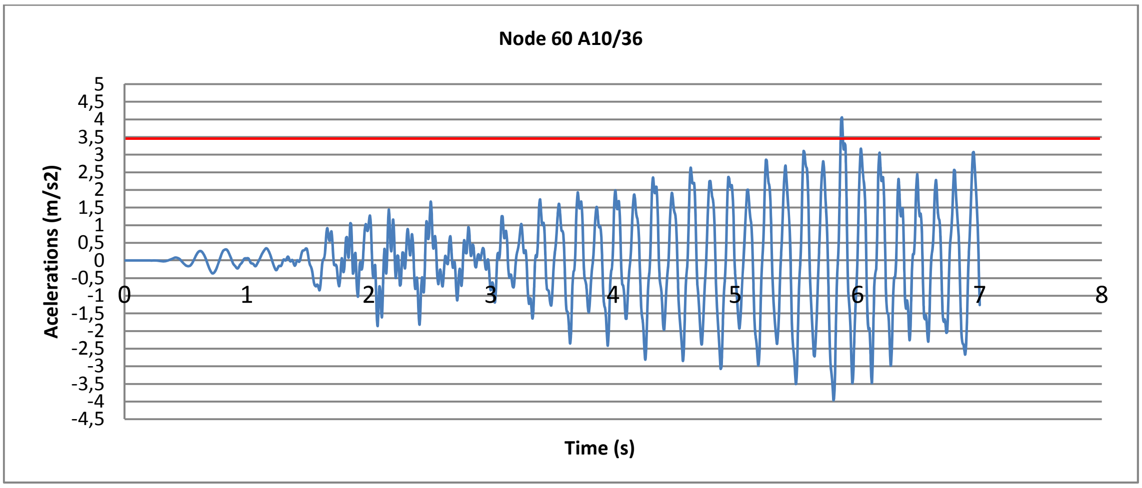

Fig. 20 – HSLM-A10 v=320 Km/h (rigidity increased)

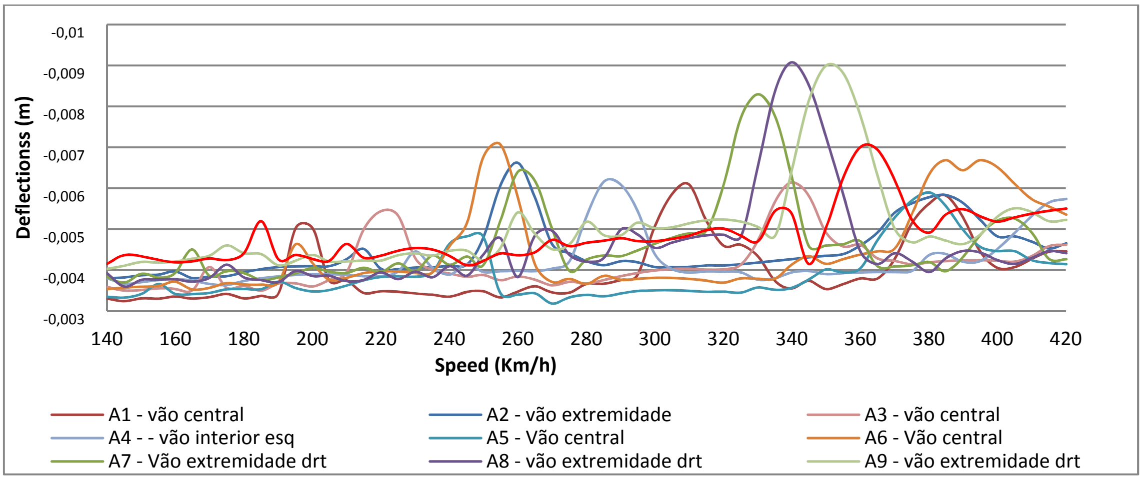

Fig. 21 – Acceleration for family HSLM-A (rigidity increased)

Table 5 – Maximum accelerations

Train	$Acel_{máx}(m/s^2)$	Speed (Km/h)
HSLM-1	2,93	380,16
HSLM-2	3,56	415,08
HSLM-3	4,15	415,08
HSLM-4	4,13	420,12
HSLM-5	3,02	375,12
HSLM-6	2,82	384,84
HSLM-7	3,18	285,12
HSLM-8	3,02	290,16
HSLM-9	2,90	420,12
HSLM-10	4,10	320,04

Table 6 – Resonance velocities according to (EN 1991-2)

	D(m)	i=1	i=2	i=3	i=4
A1	18	198,69	-	-	-
A2	19	209,73	-	-	-
A3	20	220,77	-	-	-
A4	21	231,80	-	-	-
A5	22	242,84	-	-	-
A6	23	253,88	-	-	-
A7	24	264,92	-	-	-
A8	25	275,96	-	-	-
A9	26	287,00	-	-	-
A10	27	298,03	149,02	-	-

For this dissertation was studied 3 different solutions, because the base model (Fig. 9) did not verified the limit of 10 consecutive cycles up to 3,5 m/s^2 for the cracked section analysis.

- Solution 1- Base solution (cracked and no-cracked)
- Solution 2- increase de height of beam to 2,35m (cracked)
- Solution 3- change the spans to 35,45,45,35 (clacked)

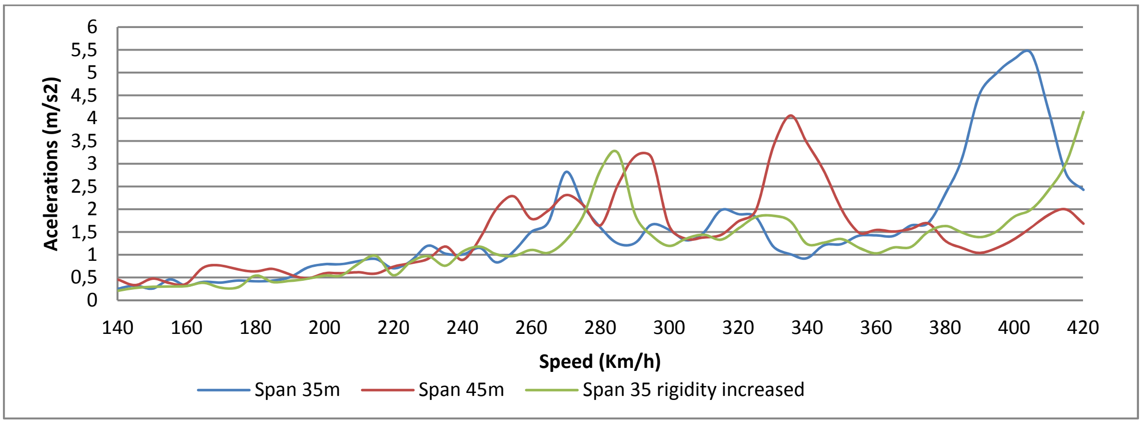

Fig. 22 – Different tested solutions

9. Concluding Remarks

The purpose of this work was to present the response of a double composite section in the analysis of high speed railway bridges.

To compute the dynamic behavior of the viaduct, a numerical model was developed in a finite element program. The dynamic analysis was based on train's circulation from 140km/h to 420 km/h. The conclusions of this analysis are presented: The worst train was the HSLM A3 reaching 4,15m/s^2 as peak value, but the criteria of 10 consecutive cycles [7] of acceleration up to 3,5 m/s^2 was verified for all the trains and for the solution 3. The resonant speeds of the frame model, is in some trains different that the resonant speed expected for the normative EN1991-2.

References

[1] Calçada R., Delgado.R & Campos e Matos A.: Bridges for High-Speed Railways. Department of Civil Engineering, Faculty of Engineering, University of Porto, Portugal

[2] European Committee for Standardization (CEN). Eurocode 1 – Actions on structures – Part 2: Traffic loads on bridges. CEN, 2002.

[3] ERRI D214/RP9: "Rail Bridges for speeds > 200km/h.", European Rail Research Institute, 2001.

[4] Giocolea, J.M.(2007 Novembro). Dynamic behaviour of high-speed railway bridges; Research at Escuela de Caminos, UPM Seminar at Instituto Superior Técnico, Lisboa

[5] L 245/143. Technical specification for interoperability relating to infrastructure subsystem, Official Journal of the European Communities, 2002.

[6] Rocha, Jorge Manuel Pereira Fernandes: "Pré-fabricação de tabuleiros de viadutos para comboios de alta velocidade" (in Portuguese), MSc Dissertation, IST, 2006.

[7] Reis J.A &. Pedro J.O. Composite Truss Bridges:new trends, design and research.