INSTITUTO SUPERIOR TÉCNICO
Universidade Técnica de Lisboa

Dynamic behaviour of high-speed railway bridges with ballastless track

Hugo Baleia Casal

Civil engineering Department, Instituto Superior Técnico, Universidade Técnica de Lisboa, Portugal

Abstract

The ballastless track is already a credible solution in Railway Engineering, showing great competitiveness against the more usual ballasted track. Actually, the major advantages of the ballastless track, such as low maintenance, high availability or even reduced structure weight and height, have been the reason to tend more and more towards the application of this type of track on high speed railway lines and on high speed railway bridges. This trend is part of the reaction to the great new demands imposed by high-speed, particularly in terms of evaluation and control of train-induced vibrations effects, which is why this dissertation aims to study the dynamic behaviour of bridges with this type of track.

Under this scope, the present dissertation includes a hypothetical case study in order to analyse the dynamic behaviour of a high speed railway viaduct with Rheda 2000® ballastless track. Thus, by using numerical models of finite elements to represent the viaduct with the track, it is important to assess the degree of reliability of such models and how it affects the dynamic response obtained. In order to do so, two different alternatives were considered to include the ballastless track in the finite element model of the viaduct: the track included through its equivalent weight and the track represented by means of a finite element model. Furthermore, regarding the finite element model of the track, some of its components were subjected to a parametric study in order to assess their influence in the dynamic behaviour of the structure. Finally, a complementary study was performed with the purpose of providing a comparison between the effects of the two types of track – ballasted and ballastless – in the dynamic behaviour of the structure. The dynamic analyses performed with the support of the European normative documents provided the necessary results to make possible these studies.

Keywords: Rheda 2000® ballastless track, high-speed railway bridge, dynamic analysis, moving loads, numerical models of finite elements, ballasted track.

1. Introduction

The emergence of high speed railway lines has imposed great new demands on the entire railway system, i.e., the infrastructure, the rolling stock and the logistics. With regard to infrastructure and in response to these growing demands, a new type of track appears in the 1970s: the ballastless track. This innovative type of railway track would quickly evolve into a wide range of different solutions developed by different countries.

The ballastless track stands out mainly for its performance, leading to substantial reductions in maintenance costs and also in maintenance work like tamping, ballast cleaning or track lining. Despite the high initial construction costs, these expenses may be offseted over the service life of the track, creating a more economical and competitive solution when one assesses a broader dimension of time.

Additionally, the problem with drag forces at ballast due to the passage of high speed trains is no more a reason for concern. Indeed, this feature, together with others, has increasingly been providing the application of ballastless tracks on high speed railway lines

Recently, the application of ballastless tracks on high speed railway bridges has also been a growing trend. Actually, consideration must be given to problems caused by this type of application. However, one cannot ignore some immediate benefits, such as the substantial reduction in the weight of the track and the relief of problems associated with the dynamic effects due to the passage of vehicles, which sometimes can lead to the shedding of ballast in traditional railway tracks. A direct consequence of the latter issue is a larger tolerance in limit checks on the vertical accelerations of the deck, present in several design normative documents. These matters, along with the wide range of benefits associated with ballastless tracks, can certainly be enough to justify the use of this kind of railway track on bridges.

High-speed has also imposed new demands on the structure of railway bridges. These demands are, in part, a reaction to the dynamic effects due to the passage of vehicles. In fact, the whole design of high speed railway bridges gets a new attention after the research carried out by the D214 committee under the scope of European Rail Research Institute (ERRI) [4]. The committee considers that the resonance phenomenon is likely to occur for the passage of trains with speeds above 200 km/h. As a consequence, the study of the dynamic behaviour of railway bridges subjected to moving vehicles gained a substantial importance in the design of such structures.

Thus, it is crucial to question the assumptions under which all the study is based, which will largely determine its degree of reliability. It should be noted, for example, the degree of representation and detail desired for a possible numerical model of a bridge with ballastless track. A better representation of the ballastless track through finite elements may have influence on the dynamic response of the structure. And given this representation, it is interesting to see how some of the components of the track interfere in the dynamic behaviour.

The use of finite element models for the preparation of such studies raises a host of questions to clarify in order to promote more efficient and reliable procedures for the determination of the dynamic effects in high-speed railway bridges.

Even the ballastless track effect on the dynamic behaviour of the bridge is itself a matter of interest. Therefore and as a complement, it is important to assess how the type of railway track, i.e. ballasted track or ballastless track, influences such dynamic behaviour.

It is within this scope that is based this document, which was prepared on the assumption of providing a clear study about the dynamic behaviour of a high speed railway viaduct.

Initially, a general approach on the ballastless track is provided. Soon after, a case study which forms the basis for the preparation of this document is shown. Finally, the presentation and discussion of the various results is performed.

2. Overview of ballastless railway tracks

The ballastless track is already at the forefront of the Railway Engineering, presenting a wide range of advantages when compared to the conventional ballasted track. The high demands imposed by high speed railway traffic have provided conditions to invest in a new kind of track with a revolutionary performance. Actually, it is especially at high speed that one takes the biggest advantages of using ballastless tracks, which essentially boil down to:

- A significant reduction in the required maintenance during the service life of the track, which leads to a higher availability and to a substantial reduction in costs of about 20-30 % when compared to traditional ballasted track;
- A considerable increase in the service life of the track;
- No drag forces at ballast with the passing of high-speed trains;
- Better control of structural behaviour of the track in terms of stiffness and stability. Besides, higher lateral resistance is achieved;
- A substantial reduction of weight and height of the track, making this type of solution more desirable on bridges or tunnels.

However, the ballastless track still exhibits some limitations when compared to the ballasted track, still very competitive. The following highlight the main disadvantages:

- High construction costs;
- Large modifications or repairs in track can only be made possible by substantial amounts of work and cost;
- Extra attention and care are required when ballastless track is applied on earth work, as it is necessary to avoid possible settlements that may damage the structure of the track. This problem usually does not apply on bridges, in which the structure itself assures all the support required;
- High airborne noise reflection.

Some constraints are also involved in the application of ballastless tracks on bridges, a reality that assumes a special status in Civil Engineering. Indeed, this subject stands out for its innovation. The entire risk assumed by choosing a path that could undermine an entire design for lack of experience or even knowledge, can be explained primarily on the intention of the designer, which seeks to harness all the advantages of ballastless track as set out above. Regarding this type of application, some problems can be presented, as follows:

- The independence between the structure of the track and the structure of the bridge which is, to some extent, a hindrance to optimum compatibility of the two structures. This happens because these structures hold characteristics quite different from each other like the service life for which they are designed or even the response of each one when loaded by moving vehicles.
- The track-bridge interaction. This kind of track is not very tolerant to variations in the bridge length due to changes in temperature, braking and acceleration of trains or even creep or shrinkage of concrete. Often, if a more desirable solution is impossible to achieve, the only alternative is to use rail expansion joints, which is a more expensive and avoidable option;
- The track-bridge interaction on transition areas such as abutment-bridge deck and bridge deck-bridge deck on piers. Once again, this kind of track is not very tolerant to deformations and lifting forces that the rail is subjected, as a result of deformations of the bridge. To solve these inconveniences the construction costs are likely to increase by having to apply alternative solutions, using, for example, transition slabs.
- The transition areas between the track on railway bridges and the track on earth works, where is likely to occur discontinuities caused mainly by settlements at

Table 1- An overview of different designs of ballastless track systems according to some groups with similar characteristics among themselves.

Ballastless track systems					
Discrete rail support				Continuous rail support	
With sleepers or blocks		Without sleepers		Embedded rails	Clamped rails
Sleepers or blocks embedded in concrete	Non embedded sleepers	Prefabricated concrete slabs	Direct rail fastenings		
- Rheda® (from Rheda Classic to Rheda 2000) - Züblin® - LVT-SONNEVILLE® - SBV®*	- FTR®* - SATO®* - FFYS®* - WALTER®* - GETRAC®* - ATD®* - BTD®	- Shinkansen® - Bögl®	- Civil structures - Monolithic in-situ slabs	- Embedded rail structure (ERS) EDILON corkelast®	- Cocon® - ERL® - KES®

*asphalt in use

earth works. Again, the ballastless track is not tolerant to localized variations on the support, so some improvements on the mechanical properties of the soil should be made.

Currently there is a large variety of different designs of ballastless track systems, mostly thanks to investments made by countries such as Germany, Japan or the Netherlands

The continuous search for improvement of construction methods and structural performance led to this large variety of solutions, which contributes more and more to the credibility and competitiveness of ballastless track.

Table 1 shows this set of different solutions, organized into groups with similar characteristics among themselves in order to provide a quick interpretation of the more relevant differences.

The current multiplicity of solutions already developed for ballastless track could possibly hinder the selection process by the designer because there is a high technical, economical and even functional ambiguity between those available solutions.

However, the Rheda® system may be considered a benchmark for proving its performance and efficiency over the years, having already presented a wide range of applications since its origin. The latest version of this system, the Rheda 2000® system, is duly represented in Figure 1:

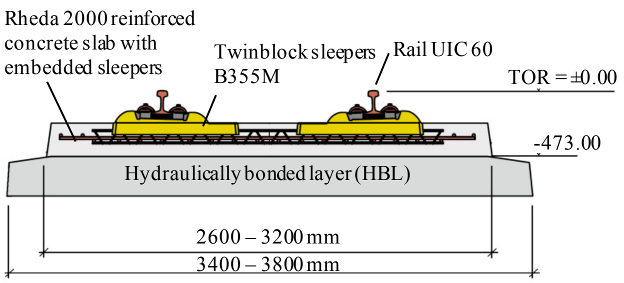


Figure 1 – Schematic representation of Rheda 2000® system. Adapted from [12].

The Rheda 2000 system consists of a concrete supportive slab, where the reinforcing bars are applied at the centre of the slab to provide adequate control of cracking and transmission of lateral forces. Twinblock sleepers are embedded in this slab in order to contribute to the monolithic nature of this support layer.

3. Presentation of the case study

In this Chapter, a case study, that is the basis for the entire study developed under the scope of the dynamic behaviour of a high-speed railway bridge with ballastless track, is presented. Simply, one considers a case study for the bridge and a case study for the track. In both, one wishes to numerically model the real structure of the bridge and the real structure of the track.

3.1. Presentation of the bridge

The case study for the bridge was based on the São Martinho Viaduct, located in the Natural Reserve of the Sado estuary. This structure is designed to accommodate rail traffic on ballasted track, with a design speed of 220 km/h.

From the structural point of view, the Viaduct is composed of 8 continuous segments, 7 with 4 spans of 28.4 m length each, giving a total length of 113.6 m, and another segment with 56.8 m, consisting of 2 spans of 28.4 m. Only the structural portion of 113.6 m in length will be modelled to study its dynamic behaviour.

The Viaduct cross-section (Figure 2) consists of a prestressed concrete deck, composed of two main girders connected by the railway platform concrete slab. At supports there is a diaphragm with 0.60 m width.

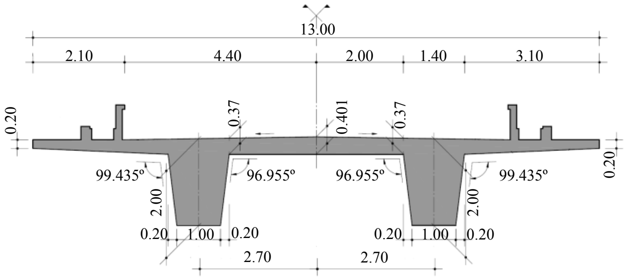


Figure 2 – São Martinho Viaduct cross-section (in meters). Adapted from [13].

The deck is composed by concrete of C40/50 grade. Table 2 presents the design values for actions, e.g. the values for permanent loads, with respect to the self weight of the deck, and the values for other permanent loads.

Table 2 – Design values for actions.

<table>
<tr><th colspan="2">Permanent loads [kN/m]</th><th>Total [kN/m]</th><th></th></tr>
<tr><td>Self-weight of the deck</td><td>204.0</td><td>204.0</td><td rowspan="7">247.8</td></tr>
<tr><th colspan="2">Other permanent loads [kN/m]</th><td rowspan="6">43.8</td></tr>
<tr><td>Footpath</td><td>20.3</td></tr>
<tr><td>Side walls</td><td>6.9</td></tr>
<tr><td>Metallic vehicle parapet</td><td>1.0</td></tr>
<tr><td>Cornice</td><td>7.2</td></tr>
<tr><td>Regularization and waterproofing</td><td>8.4</td></tr>
</table>

3.2. Numerical modelling of the bridge

São Martinho Viaduct was represented using 4 different numerical models based on finite elements. These 4 models have undergone a process of evaluation and comparison to indentify which is the best suited for the present study. The models created were:

- A numerical model with solid finite elements;
- A bar numerical model with frame finite elements;
- A numerical model with shell finite elements;
- A grid numerical model with frame finite elements.

the last two were the result of work developed by Neves [10]

3.2.1. Model with solid elements

The model with solid finite elements was entirely developed in ANSYS, a commercial FE program of structural analysis.

For this purpose, solid elements with 8 nodes each, named SOLID185 in ANSYS, were used to model all the components. Each node has 3 degrees of freedom of translation, providing a total of 24 degrees of freedom per element. The size of these finite elements was cleared to ensure a high degree of discretization. In total, 60 530 nodes and 37 540 solid finite elements were used.

This model was considered a reference for the calibration of the remaining models, given its high degree of reliability and given the impossibility of conducting an experimental campaign. For such calibration, two static analyses and one modal analysis were performed. Figure 3 and Figure 4 show two fundamental mode shapes obtained from modal analysis.

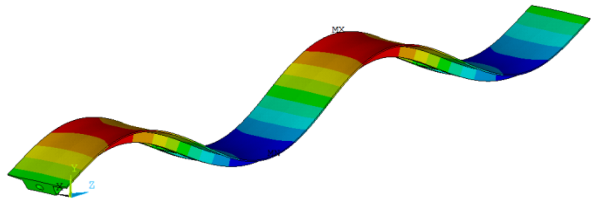


Figure 3 – Graphical representation of the vertical fundamental mode shape (f=4.0274 Hz).

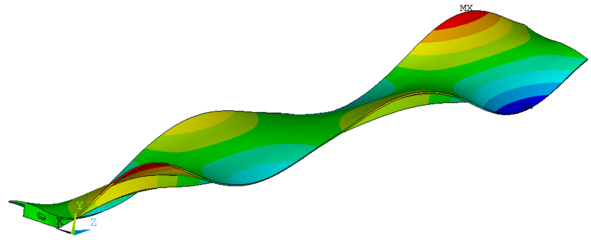
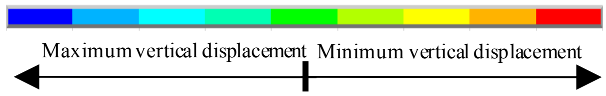


Figure 4 – Graphical representation of the torsional fundamental mode shape (f=7.0454 Hz).



This model was not selected to perform the planned dynamic analyses because of the unreasonable calculation times that frustrate the task. This situation would remain even if a coarser discretization was used.

3.2.2. Bar model

The numerical modelling of São Martinho Viaduct with a bar model was entirely developed in SAP2000, a commercial FE program of structural analysis.

The deck was modelled making use of frame finite elements longitudinally aligned and with an average length of 0.3 m. In order to represent deck areas on piers and on half span, two different cross sections for frame finite elements were defined. In total, 400 nodes and 399 frame finite elements were used.

Figure 5 represents graphically the vertical fundamental mode shape obtained from modal analysis:

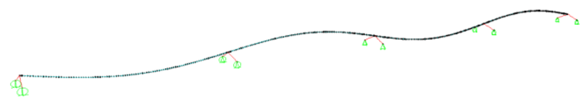


Figure 5 – Graphical representation of the vertical fundamental mode shape (f=4.1361 Hz).

The bar model was not used to perform dynamic analyses because it is completely unable to effectively represent the torsional behaviour of the Viaduct and, therefore, its torsional mode shapes. This model is not appropriate for the concerned dynamic study since it is not possible to represent, in a reliable way, the effect of the eccentricity of rails.

3.2.3. Shell model

The numerical modelling of São Martinho Viaduct with a shell model was entirely developed in SAP2000.

The Viaduct was modelled in a three-dimensional configuration with shell finite elements with 4 nodes per element.

The beams are represented making use of two longitudinal alignments of shell finite elements classified as “thick” and with a non-constant thickness in order to considerer the differences between the bottom and the top of the beam. The top slab between beams and the cantilevers were modelled in a similar way using shell finite elements classified as “thin” and with a non-constant thickness. The diaphragms are represented with shell elements classified as “thin” and with a constant thickness.

In total, the model contains 6825 nodes and 6554 shell finite elements.

Figure 6 and Figure 7 represents graphically two fundamental mode shapes obtained from modal analysis:

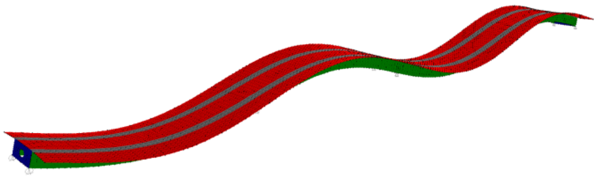


Figure 6 – Graphical representation of the vertical fundamental mode shape (f=4.0220 Hz).

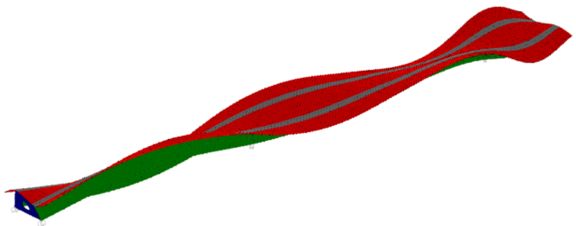


Figure 7 – Graphical representation of the torsional fundamental mode shape (f=7.2839 Hz).

The shell model could be a viable option, if the grid model did not exist, thanks to the high degree of reliability of its performance, very close to the model with solid elements. The criteria used for comparison showed few parameters with deviations exceeding 5%. Moreover there’s a clear similarity between the first 9 mode shapes and those obtained from the reference model. This option was not used to perform the desired dynamic analyses since it could cause high calculation times. Indeed, as one may know, using shell elements could lead sometimes to a more complex model.

3.2.4. Grid model

The numerical modelling of São Martinho Viaduct with a grid model was entirely developed in SAP2000.

The Viaduct was modelled in a three-dimensional configuration with frame finite elements.

The two beams were modelled making use of two longitudinal alignments of frame elements with an appropriate cross-section identical to the beam. However, their characteristics were slightly modified as a result of the calibration work. The frame elements representing the cantilevers and the top slab between beams were calibrated and transversely aligned, being replicated to a constant value of 0.65 m. Mention should be made to the rigid bending nature of finite elements representing the cantilevers since it will avoid excessive local mode shapes.

Finally, the diaphragms were modelled resorting to the use of frame elements with 0.60 m thick and with rigid characteristics to enable their connection to other elements. The number of frame elements was found out after calibration.

In total, 2787 nodes and 3014 frame finite elements were used.

Figure 8 and Figure 9 represents graphically two fundamental mode shapes obtained from modal analysis:

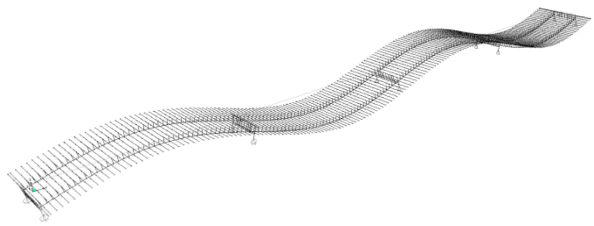


Figure 8 – Graphical representation of the vertical fundamental mode shape (f=4.0271 Hz).

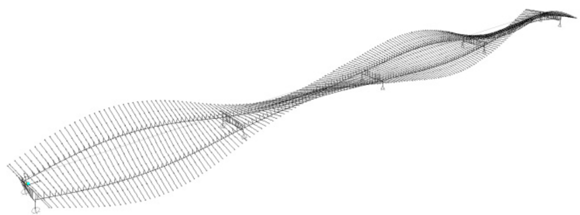


Figure 9 – Graphical representation of the torsional fundamental mode shape (f=7.0443 Hz).

The grid model was selected to perform the desired dynamic study and subsequent dynamic analyses. With a performance very similar to the shell model, regarding the comparison criteria used, this model proved to be a strong option for the realization of dynamic analyses. In addition, the calculation times of this model were likely to be lower than those from the shell model, which did not show other significant advantages. The grid model is the most appropriate, among the four models, to prepare the desired dynamic study.

3.3. Presentation of the track

The case study for the track was based on the Rheda 2000 system ballastless track, being adopted the entire structural configuration used on the Hollandsch Diep Bridge, in the Netherlands. This civil structure is a rare case of a high-speed railway bridge using this kind of track and with traffic up to a limit of 300 km/h.

This bridge over the Hollandsch Diep River is 2 km long and is part of the HSL-Zuid railway line, a Dutch portion of 100 km length integrated in the European high speed network.

The cross-section of Rheda 2000 ballastless track used on the Hollandsch Diep Bridge is depicted in Figure 10:

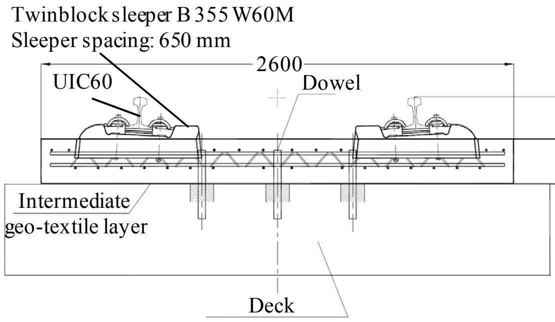


Figure 10 – Cross-section of Rheda 2000 ballastless track used on the Hollandsch Diep Bridge (in millimetres). Adapted from [14]

The Rheda 2000 slab is anchored to the bridge structure in pre-designated free-drilling zones by means of high quality stainless steel dowels with a diameter of 40 mm [14]. An important feature of this connexion is its capacity to allow movements in the longitudinal direction. Thus, the overloading of dowels, as a result of expansion or shrinkage of deck or even track's concrete, can be avoided. [7].

The ballastless track is composed by segmented slabs with a length of 3.50 to 6.40 m.

The Rheda 2000 slab used concrete of C35/45 grade for its construction. Some specific values regarding the weight and mass of the track can be seen in Table 3:

Table 3 – Mass and weight of the ballastless track considered in the created models.

Component of the track	Mass per meter length [kg/m]	Weight per meter length [kN/m]
Rail UIC60	60.34 [5]	0.59
Rheda 2000 slab	1679.32	16.46
Ballastless track	1800.00 [14]	17.64

3.4. Consideration of railway ballastless track

The ballastless track was considered according to two different approaches:

- consideration of the track by means of linear distributed forces;
- consideration of the track by means of a finite element model, which reproduces the characteristics of several components of the track.

The first procedure was very simple and easy to perform. The inclusion of the ballastless track in the grid model through its equivalent weight (Table 3) was the basis for this type of representation.

The second procedure, i.e. the numerical modelling, was inspired by the double beam model, a type of representation suggested by several authors [5], [1] and [15] and depicted in Figure 11:

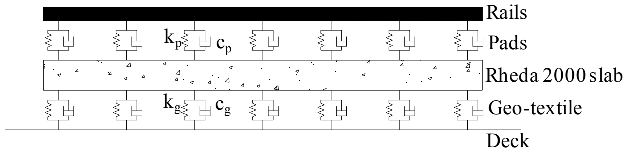


Figure 11 – Schematic representation of the ballastless track model.

The UIC60 rails and the Rheda 2000 slab were modelled using frame elements, while the remainder components were modelled using finite elements that reflect the properties of a viscoelastic material characterized by stiffness and damping values.

The properties of the frame elements are shown in the Table below:

Table 4 – Properties of the frame elements of the track model.

	Property		Value
2 UIC60 rails	Specific weight	γ_r	76.930 kN/m^3
	Area	A_r	1.537 x 10^{-2} m^2
	Elasticity modulus	E_r	210.00 GPa
	Poisson's ratio	ν_r	0.3
	Vertical bending inertia	I_{xx}	6.110 x 10^{-5} m^4
	Lateral bending inertia	I_{yy}	7.925 x 10^{-3} m^4
Rheda2000 slab	Specific weight	γ_s	26.374 kN/m^3
	Área	A_s	0.624 m^2
	Width x height	[-]	2.60m x 0.240m
	Elasticity modulus	E_s	34.00 GPa
	Poisson's ratio	ν_s	0.2

The longitudinal alignment of the frame elements of the Rheda 2000 slab is not continuous but segmented, with segments of 6.5 m length.

With regard to the elements that represent the pads and the geo-textile, several values of stiffness and damping were defined in order to enable a parametric study Thus, one may assess how these properties affect the dynamic behaviour of the structure.

4. Dynamic analyses

This section presents a comparative study that makes use of dynamic analysis with moving loads. This study aims at evaluating the dynamic behaviour of the Viaduct taking into account the type of representation chosen for the ballastless track. Some parametric analyses will be performed in order to facilitate such study and to understand how the track model is affected by the properties of the pads and the geo-textile.

At the end of this section, a comparative study that aims to assess the influence of the type of railway track (ballasted or ballastless) in the dynamic behaviour of the structure will be presented. In fact, this works as a complement, since the initial study was conducted on the assumption that the Viaduct holds a ballastless track. The dynamic analyses were performed according to the following assumptions:

- The method of modal superposition was applied, considering a number of frequencies and associated mode shapes on the analysis up to the greater of 30 HZ, as recommended in EN1990-A2 [2];
- The structural damping was simulated using the Rayleigh damping;
- The used time step was $\Delta t = 0.005$ s;
- The range of speeds used was 140 km/h to 420 km/h, being considered a speed step of Δv=10 km/h;
- Only the 4th span of the Viaduct was assessed.

4.1. Grid model without the ballastless track model

The envelope of accelerations and displacements, depicted in the Figures below, were obtained for all the ten HSLM-A defined in EN1991-2 [3] and making use of the grid model without the track model:

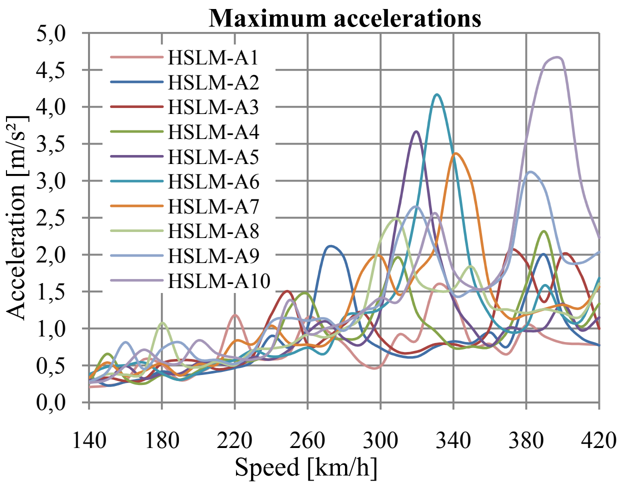


Figure 12 – Envelope of accelerations for the passage of the HSLM-A.

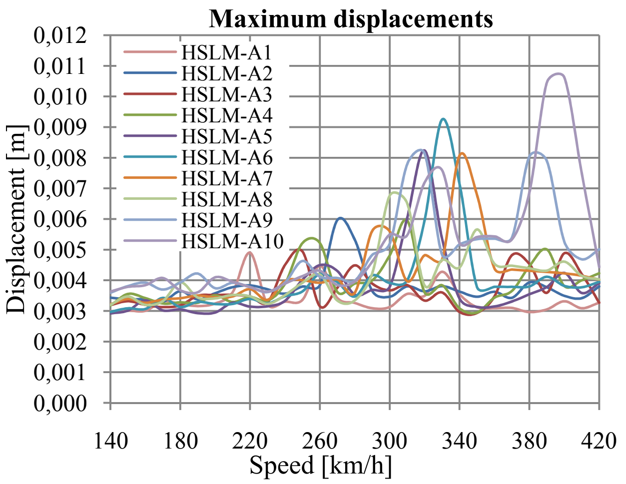


Figure 13 – Envelope of displacements for the passage of the HSLM-A.

As it can be observed, the maximum acceleration and displacement occur for the HSLM-A10, which induces dynamic responses with quite significant resonance peaks. A maximum acceleration of 4.613 m/s^2 and a maximum displacement of 0.01062 m were computed.

The serviceability limit of 5 m/s^2 for the acceleration of ballastless tracks, defined in EN1990-A2 [2], was never exceeded.

As expected, the HSLM-A results are always higher than those calculated with real trains.

4.2. Grid model with the ballastless track model

Regarding the grid model with the track model, the properties of the pads and geo-textile, such as the dynamic stiffness ($k_{d,p}$ and $k_{d,g}$) and the damping (c_p and c_g), were firstly assigned with the reference values present in Figure 14. The next Figures show, for these reference values, the results obtained from the dynamic analyses performed for the HSLM-A10:

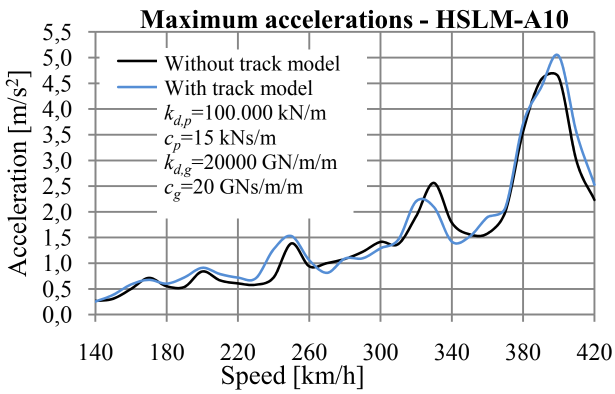


Figure 14 – Envelope of accelerations for the passage of the HSLM-A10.

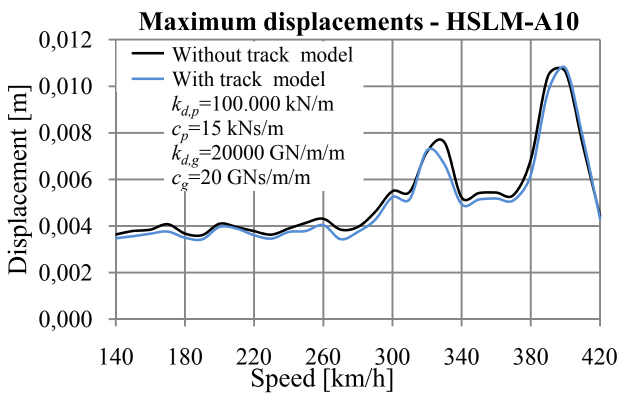


Figure 15 – Envelope of displacements for the passage of the HSLM-A10.

As one can notice, the envelope of accelerations and displacements display significant deviations when compared to those obtained from the grid model without the track model. However, the configuration of these envelopes still exhibits an obvious similarity.

Changing the properties of the pads for the ones presented in Table 5, the dynamic analyses for the HSLM-A10 were performed once again.

Table 5 – List of models tested with distinct properties for the pads.

Grid model	Pads	
	Dynamic stiffness $k_{d,p}$ [kN/m]	Damping c_p [kNs/m]
With track model 1	100000	15
With track model 2	100000	75
With track model 3	100000	500
With track model 4	30000	15
With track model 5	30000	75
With track model 6	30000	500
With track model 7	∞	—

The next Figures show the results obtained from the dynamic analyses performed for the HSLM-A10:

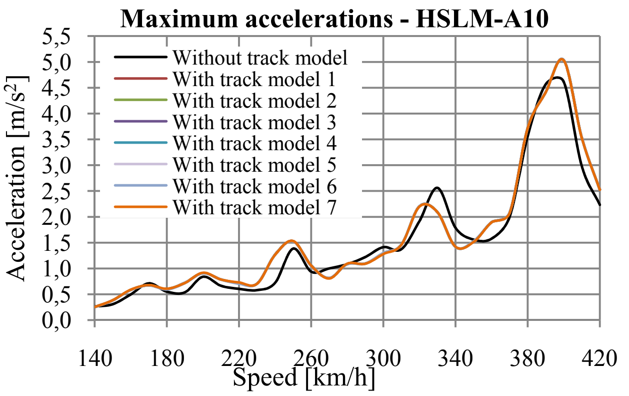


Figure 16 – Envelope of accelerations for the passage of the HSLM-A10, obtained from the grid model with track model 1-7 and from the grid model without the track model.

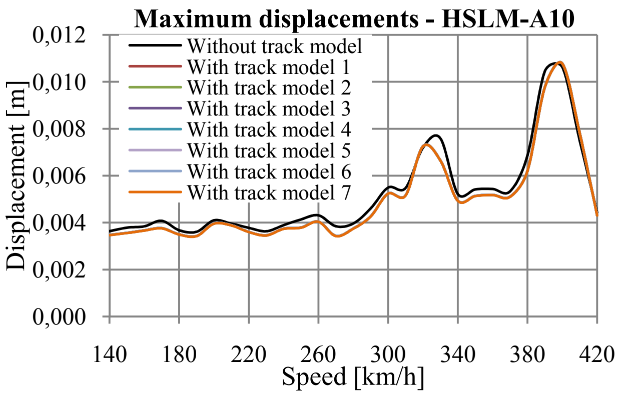


Figure 17 – Envelope of displacements for the passage of the HSLM-A10, obtained from the grid model with track model 1-7 and from the grid model without the track model.

As it can be observed, and regarding the 7 models tested, the graphical representations of the envelopes are indistinguishable from each other. The variation of the properties of the pads between the different proposed values does not cause any substantial change in the response of the global structure.

Finally, the properties of the geo-textile assigned with the following values:

Table 6 – List of models tested with distinct properties for the geo-textile.

Grid model	Geo-textile	
	Dynamic stiffness $k_{d,g}$ [GN/m/m]	Damping c_g [GNs/m/m]
With track model A	20000	20
With track model B	2000	2
With track model C	200	0.2
With track model D	20000	200
With track model E	∞	—

The next Figures show the results obtained from the dynamic analyses performed for the HSLM-A10:

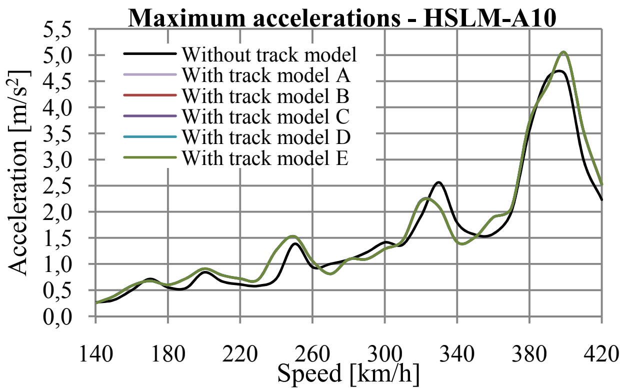


Figure 18 – Envelope of accelerations for the passage of the HSLM-A10, obtained from the grid model with track model A-E and from the grid model without the track model.

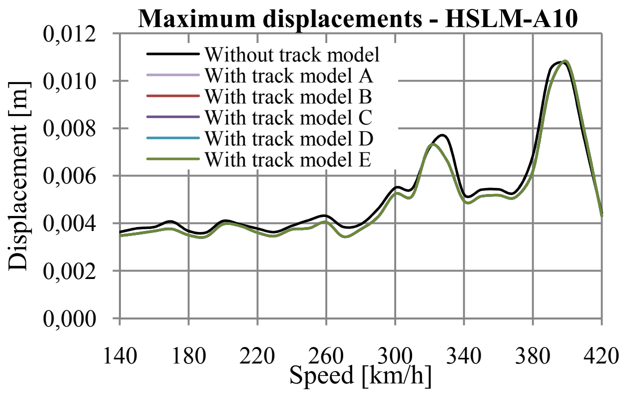


Figure 19 – Envelope of displacements for the passage of the HSLM-A10, obtained from the grid model with track model 1-7 and from the grid model without the track model.

Once more, and regarding the 5 models tested, the graphical representations of the envelopes are indistinguishable from each other. The variation of the properties of the geo-textile between the different proposed values does not cause any substantial change in the response of the global structure.

By comparing the dynamic responses of the grid model without the track model and the grid model with the track model, it can be concluded that the main differences recorded are due to the frame elements used to represent the rails and the Rheda 2000 slab. In fact, these elements add an extra stiffness in the structure. Naturally, such stiffness is not taken into account in the grid model without the track model. This extra stiffness will appreciably change the dynamic response of the structure since these elements absorb some of the load of the vehicles.

Consequently, it is useful to assess the degree of stiffness of the ballastless track since such stiffness might influence substantially the dynamic response of the structure.

4.3. Influence of the type of railway track in the dynamic behaviour of the structure

So far, all the presented studies were conducted on the assumption that the Viaduct holds a ballastless railway track. Therefore, and as a complement, it will be assessed the influence of the type of railway track on the dynamic behaviour of the structure. The highlights are the ballasted track (studies developed by Neves [10]) and the ballastless track. To accomplish this task, it will be taken into account the influence of the weight of the tracks, the influence of the modelling of the tracks and the comfort.

To assess the influence of the weight of the tracks, two grid models without the ballasted track model (BT1 and BT2 with different weights presented in Table 7) and the grid model without the ballastless track model (BLT), were considered.

Table 7 – Different weights of the ballasted tracks considered.

Grid model with ballasted track	Specific weight of ballast [kN/m^3]	Weight per linear meter of track [kN/m]
Model BT1	17.0	47.5
Model BT2	20.0	56.0

The next Figures show the results obtained from the dynamic analyses performed for the HSLM-A10:

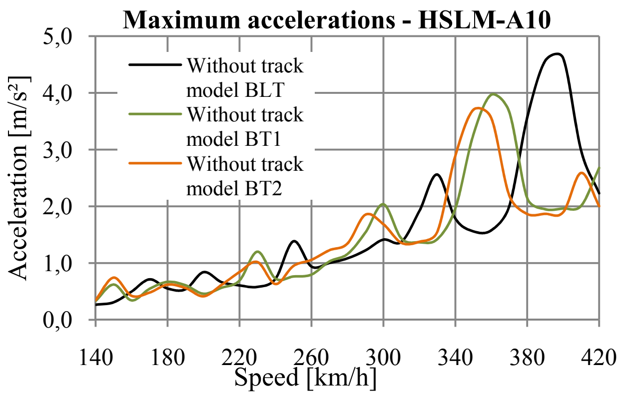


Figure 20 – Envelope of accelerations for the passage of the HSLM-A10, obtained from the grid model without ballastless track model (BLT) and from the grid models without ballasted track model (BT1 and BT2) [10].

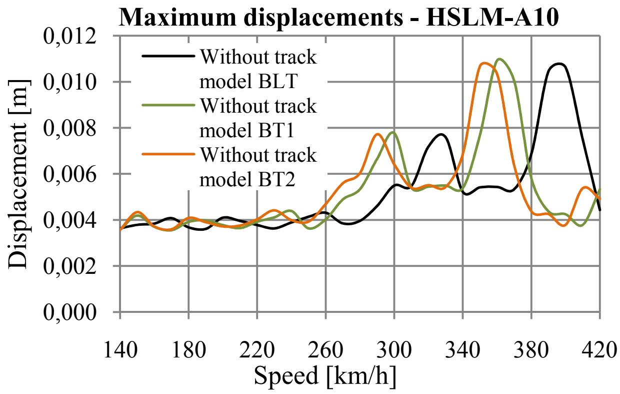


Figure 21 – Envelope of displacements for the passage of the HSLM-A10, obtained from the grid model without ballastless track model (BLT) and from the grid models without ballasted track model (BT1 and BT2) [10].

As one can observe, the envelopes of accelerations and displacements are lagged, despite the obvious similarity in their configurations. Thus, the lighter the track is, the more to the right the resonance peaks will be shifted. Furthermore, the lighter the track is, the higher the accelerations peaks will be. In structures with a lighter track one might avoid resonance peaks recorded in structures with heavier tracks if such peaks occur outside the range of speeds analysed.

To assess the influence of the modelling of the tracks, the model with non-vibrating ballast and the model with vibrating ballast [10] (Figure 22 and Figure 23) were adopted:

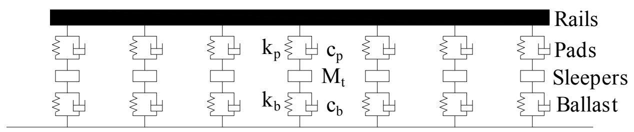


Figure 22 – Schematic representation of the model with non-vibrating ballast for the modelling of the ballasted track [10].

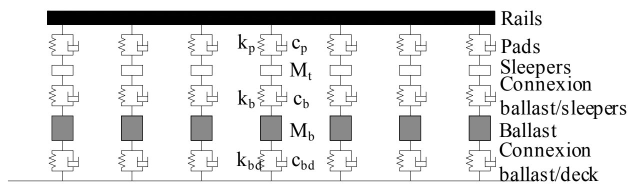


Figure 23 – Graphical representation of the model with vibrating ballast for the modelling of the ballasted track [10].

The next Figures show the results obtained from the dynamic analyses performed for the HSLM-A8:

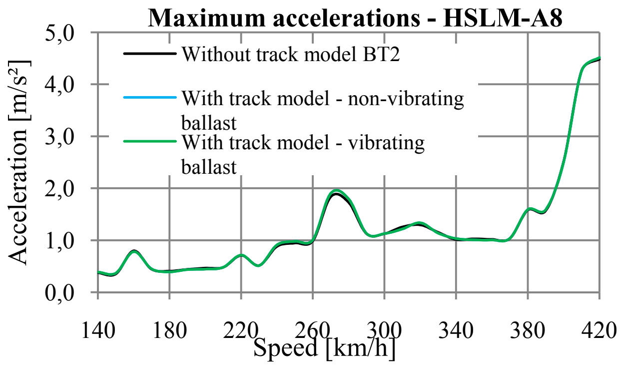


Figure 24 – Envelope of accelerations for the HSLM-A8, considering 3 different possibilities for the inclusion of the ballasted track.

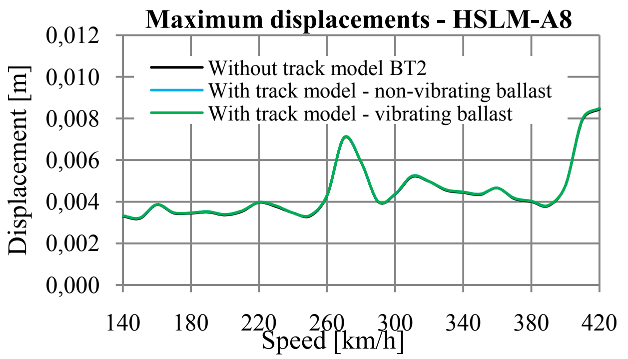


Figure 25 – Envelope of displacements for the HSLM-A8, considering 3 different possibilities for the inclusion of the ballasted track.

As can be seen, and contrarily to what occurs in the case of the ballastless track, the ballasted track considered by means of a track model with vibrating ballast or non-vibrating ballast does not cause significant changes in the dynamic response obtained from the grid model without the track model.

This happens because the models with vibrating and non-vibrating ballast are prone to vibrate for frequencies above the limit of 30 Hz, imposed by the EN1990-A2 [2] to determine the dynamic response. Furthermore, it is not considered the shear stiffness of the ballast. Therefore, the ballasted track models added just a small extra stiffness in structure. Such stiffness is related to the rails. This small stiffness increment causes virtually no effect in the dynamic response of the structure.

With regard to comfort, "very good" levels were reached for both types of railway track. Notwithstanding, one may indentify the ballastless track as the railway track with the highest values of L/δ, well above the limit of 0.9*L/δ=1402 defined in EN1990-A2 [2]. Besides, it was noted a considerable influence of the pads in the vertical deflection calculated along the rails.

5. Conclusions

The present work was developed in order to provide a study regarding the dynamic behaviour of a high-speed railway bridge with a ballastless track. Unsurprisingly, recent applications tend more and more towards the ballastless track, which stands out for its innovative character and its enhanced performance. On the one hand, this study aimed to evaluate how the representation of the railway track influences the dynamic behaviour of the global structure of a bridge, obtained from a numerical model of finite elements. On the other hand, and as a complement, it was also assessed the influence of the type of railway track in such dynamic behaviour.

Regarding the two different approaches for the consideration of the ballastless track, some conclusions can be drawn. In fact, the consideration of the track by means of a finite element model, which reproduces the characteristics of several components of the track, might be useful when compared to simple considerations of the track by means of equivalent forces. By analysing some components of the ballastless track duly represented in the finite element model, one can conclude the following:

- When the properties of the pads and the geo-textile are varying between the proposed values, no substantial changes are recorded in the response of the global structure. In principle, these elements are oriented to filter out the high frequency force components, well above the limit of 30 Hz imposed by the EN1990-A2 [2] to determine the dynamic response;
- The rails, and specially the Rheda 2000 slab, might add a significant stiffness in the global structure that might influence its dynamic response. Consequently, it is useful to assess the degree of stiffness of the ballastless track caused mainly by these elements.

By assessing the influence of the type of railway track in the dynamic response of the bridge, some conclusions can be drawn:

- When a lighter track is used, the resonance peaks will be shifted to the right, i.e. will occur at higher speeds. It is possible to avoid resonance peaks occurring in structures with heavier tracks if a lighter track is used and if such peaks occur outside the range of speeds analysed;
- The lighter the track is, the higher the resonance peaks will be;
- Modelling of the ballasted track does not significantly influence the dynamic response of the global structure when obtained from a grid model without a ballasted track model;
- Both tracks present "very good" levels of comfort according to specifications presented in EN1991-2 [3] and EN1990-A2 [2]. Furthermore, the pads considerably influence the vertical deflection calculated along the rails.

References

[1] De Man, Amnon P. *A survey of dynamic railway track properties and their quality*, *PhD thesis*. Technische Universiteit Delft, Delft, 2002.

[2] EN1990-A2. *Eurocode: Basis of structural design - Annex A2: Applications for bridges (Normative)*. European Committee for Standardization (CEN), Brussels, 2005.

[3] EN1991-2. *Eurocode 1: Actions on structures - Part2: Traffic loads on bridges*. European Committee for Standardization (CEN), Brussels, 2003.

[4] ERRI Specialists' Committee D214/RP 9. *Final Report - Rail Bridges for Speeds > 200 km/h*. European Rail Research Institute , Utrecht, 2001.

[5] Esveld, Coenraad. *Modern Railway Track*. MRT-Productions, Technische Universiteit Delft, 2001.

[6] Henriques, João F. *Dynamic Behaviour and Vibration Control of High-Speed Railway Bridges through Tuned Mass Dampers, dissertação de Mestrado Integrado em Engenharia Civil*. IST, Universidade Técnica de Lisboa, Lisboa, 2007.

[7] Infraspeed. *High Speed in the Low Lands*. Infraspeed, Zoetermeer, 2006.

[8] Kaewunruen, S. and Remennikov, A.M. An Alternative Rail Pad Tester for Measuring Dynamic Properties of Rail Pads Under Large Preloads. *Experimental Mechanics*, 48 (2007), 55–64.

[9] Maes, J., Sol, H., and Guillaume, P. Measurements of the dynamic railpad properties. *Journal of Sound and Vibration*, 293 (2006), 557–565.

[10] Neves, Marco. *Comportamento Dinâmico de Pontes Ferroviárias de Alta Velocidade com Via Balastrada, dissertação de Mestrado Integrado em Engenharia Civil.* IST, Lisboa, 2010.

[11] Nielsen, J.C.O. and Oscarsson, J. Simulation of dynamic train–track interaction with state-dependent track properties. *Journal of Sound and Vibration*, 275 (2003), 515–532.

[12] Rail.one Pfleiderer track systems. *Rheda 2000® - The ballastless track with concrete supporting layer*. Rail.one, Neumarkt, 2009.

[13] Reis, A. J., Lopes, N. T., and Ribeiro, D. Track-Structure Interaction in Long Railway Bridges. In *Workshop Track-Bridge Interaction on High-Speed Railways*. FEUP, Universidade do Porto, porto, 2007.

[14] Tünnissen, Joep T. F. M. Dynamic Aspects of the High-Speed railway Bridge Across the Hollandsch Diep. In *Workshop Track-Bridge Interaction on High-Speed Railways*. FEUP, Universidade do Porto, Porto, 2007.

[15] Varandas, José, Dimitrovová, Zuzana, Palácio, Alberto, and da Silva, Manuel. *Influência da rigidez da fundação no valor da velocidade crítica de uma carga móvel. Aplicação a vias ferroviárias*. UNIC Centro de Investigação em estruturas e Construção da UNL, 2007.