The influence of torsional resistance of the deck on the dynamic response of a high-speed railway bridge

Case study: Ulla River Viaduct

CLAUDIA SANROMAN CERVERO
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Abstract

Understanding how different parameters affect the dynamic response of high-speed railway bridges is crucial to selecting an efficient structural form. Despite existing numerous publications within this field, only few address the importance of torsional deformations.

The main objective of this thesis is to investigate the influence of the torsional resistance of the deck on the dynamic response of an existing bridge. Ulla River Viaduct is presented as a case study, allowing to analyse some aspects of its design and what their alteration entails. To this end, 6 different 3D FE models are compared, 5 of which show a modification from the original configuration. In addition, several positions of the train are considered to contrast the effects when the torsional modes are excited. The performed dynamic calculations are based on the implicit direct integration procedure.

The analysis of the case study demonstrates the benefit of closing the torsional circuit of the deck. The results also evidence the need of including torsional effects in its dynamic assessment when low values of torsional rigidity are considered. All this is not easy when simplified 2D or 3D beam models are used. As a final remark, the original design of the Ulla River Viaduct is found highly efficient from a dynamical point of view.

Keywords: Dynamics, High-speed railway bridges, Torsional rigidity of the deck, FE modelling, 3D Model, Ulla River Viaduct
Resumen

Entender cómo diversos parámetros afectan el comportamiento dinámico de puentes de ferrocarril de alta velocidad resulta crucial en la elección de una configuración estructural eficiente. A pesar de la existencia de numerosas publicaciones en este ámbito, pocas abordan la importancia de las deformaciones torsionales.

La finalidad de este trabajo es investigar la influencia de la resistencia torsional del tablero de un puente existente en su comportamiento dinámico. El Viaducto sobre el Río Ulla se presenta como caso de estudio donde algunos aspectos relativos a su diseño son estudiados, así como lo que su alteración conlleva. Para ello, se comparan 6 modelos de elementos finitos en 3D, 5 de los cuales presentan una modificación del diseño original. Asimismo, para contrastar los efectos provocados al excitar los modos torsionales se han considerado varias posiciones de tren. Los cálculos dinámicos se han realizado por integración directa.

El análisis del caso de estudio demuestra que un tablero con circuito torsional cerrado resulta beneficioso. Los resultados, además, evidencian la necesidad de incluir los modos de torsión en la evaluación del puente cuando se consideran bajas rigideces torsionales. Todo esto no resulta sencillo cuando se utilizan modelos tipo viga simplificados, en 2D o 3D. Finalmente, cabe destacar que el diseño original del Viaducto sobre el Río Ulla es altamente eficiente desde el punto de vista dinámico.

**Palabras clave:** Cálculo dinámico, Puentes de ferrocarril de alta velocidad, Rigidez torsional de tablero, Modelo FE, Modelo 3D, Viaducto Río Ulla
Preface

This master thesis was conducted at the Royal Institute of Technology (KTH), department of Civil and Architectural engineering, in collaboration with TYPSA. The study was developed under the supervision of José Javier Veganzones, José Luis Sanchez and Sonia Alonso to whom I wish to express my most sincere gratitude for their guidance, advice, support and continuous feedback, but especially for letting me do research in this field. My profound gratitude to the examiner, Prof. Raid Karoumi who always found time to help me. Special thanks to John Leander and Andreas Andersson for their valuable advice with my ABAQUS model, and to Heydar Beygi and Jing Yang for their help with the moving loads simulation. I would additionally wish to thank Joan Ramon Casas for his guidance every time I needed it. Finally, huge gratitude to Jose M. Goicolea and Miguel Otega for sharing information that I found precious and essential for my study.


Claudia Sanroman Cervero
Notations

\( \alpha \) Classification coefficient
\( \beta \) Stiffness-proportional Rayleigh damping coefficient [sec]
\( \delta \) Displacements [m]
\( \lambda \) Wavelength [m]
\( \mu \) Mass-proportional Rayleigh damping coefficient [sec\(^{-1}\)]
\( \xi \) Damping ratio [%]
\( \rho \) Density [kg/m\(^3\)]
\( \Phi \) Dynamic factor or impact coefficient
\( \varphi', \varphi'' \) Coefficients for dynamic enhancement and track irregularities respectively
\( \omega \) Circular frequency [rad/s]
\( a \) Vertical acceleration [m/s\(^2\)]
\( C \) Number of intermediate coaches
\( d \) Bogie axle separation [m]
\( D \) Coach length [m]
\( e \) Eccentricity [m]
\( E \) Young Modulus [MPa]
\( f \) Frequency [Hz]
\( \mathbf{f} \) External force vector
\( I \) Second moment of inertia [m\(^4\)]
\( K \) Stiffness matrix
\( l \) Span length [m]
\( L_\Phi \) Determinant length [m]
\( m \) Mass [kg]
\( \mathbf{M} \) Mass matrix
\( p \) Load [kN]
\( S_n \) Number of samples
\( t \) Time [s]
\( T \) Total time [s]
\( \mathbf{u} \) Nodal displacement vector
\( v \) Speed of the train [km/h]
## Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Definition</th>
</tr>
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<tbody>
<tr>
<td>2D</td>
<td>Two-dimensional</td>
</tr>
<tr>
<td>3D</td>
<td>Three-dimensional</td>
</tr>
<tr>
<td>DOF</td>
<td>Degree of freedom</td>
</tr>
<tr>
<td>ERRI</td>
<td>European Rail Research Institute</td>
</tr>
<tr>
<td>FE</td>
<td>Finite element</td>
</tr>
<tr>
<td>FFT</td>
<td>Fast Fourier Transform</td>
</tr>
<tr>
<td>HSLM</td>
<td>High-Speed Load Model</td>
</tr>
<tr>
<td>HSR</td>
<td>High-speed railway</td>
</tr>
<tr>
<td>LM</td>
<td>Load Model</td>
</tr>
<tr>
<td>SRSS</td>
<td>Square root of the sum of squares procedure</td>
</tr>
</tbody>
</table>
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Chapter 1

Introduction

1.1 Motivation

During the last few decades, high-speed railway (HSR) traffic has become an interesting alternative for connecting medium and long distances. Not only is it a sustainable and comfortable way of travelling, but also a powerful tool of economic and social development for the regions involved.

After Japan built their first passenger dedicated HSR line, it rapidly started its expansion in Europe and East Asia. Nowadays, China boasts the longest HSR network, reaching almost 13000 km. Spain covers about 3100 km coming to the second place in the world and resulting in the first one in Europe. The entire world high-speed network extends over 30000 km but it is forecasted to grow around 30% within the future ten years (Figure 1.1). Detailed historic development has been addressed in the literature [1].

![Figure 1.1: Development of the world high-speed network](image)

Bridges help to overcome all the obstacles arising from the difficult orographic conditions and to avoid altering existing infrastructures, becoming an indispensable asset in the construction and operation of new HSR lines (Figure 1.2). For instance, in Spain it was necessary to build around 1200 bridges to this end [3].
These types of HSR bridges present some specific requirements clearly different from those dedicated to conventional railway traffic. Apart from ensuring safety conditions, a good comfort for the users has to be provided. The most relevant aspects in the design of these structures are the magnitude of the live loads and the dynamics effects of passing trains that may cause amplifications of forces and deformations. Hence, such effects are to be thoroughly studied in order to avoid resonant phenomena that can result in non-admissible forces and deformations, and consequently, altering the traffic conditions and causing excessive vibrations inside the vehicles. However, their estimation usually requires a great demand of computational resources and a deep knowledge of the structure itself [4].

The selection of an efficient structural form is the basic goal of bridge design. Thus, understanding how different parameters influence on their dynamic response is a crucial concern. The effect of some parameters such as span length, number of spans, damping, train speed, etc. on the said dynamic response has been the subject of numerous researchers over the last decades [5, 3, 6, 7].

On the other hand, in many practical applications, design engineers usually consider simplified 2D models for dynamic analysis of bridges, leading to a considerably large gain in terms of time and computational sources. This is the main reason why a significant amount of publications within this field focuses on linear and simple structures.

However, one may note that the vast majority of current bridges dedicated to HSR traffic are composed of double railway-track, which torsional stiffness should be high enough to cope with the torsional flows caused by the eccentric loads of only one train passing. Moreover, steel-concrete composite bridges are becoming more and more popular to this end [8]. Some of their main characteristics, in terms of dynamics, are their low values of torsional natural frequencies and the similar values of deflections of the deck due to the torsional and bending components, being necessary to consider both effects as coupled [9]. With all this in mind, it results indeed very striking that little of the research effort addresses the topic of the importance of torsional deformations.
1.2 Aim and scope

The present thesis aims to study the influence of torsional resistance of the deck on the dynamic behaviour of HSR bridges, more precisely, on the dynamic behaviour of a complex and hyperstatic worldwide-recognised structure selected as a case study: Ulla River Viaduct. This bridge uses an alternative solution for closing the torsional circuit of the deck with respect to the traditionally one employed in similar bridges typologies. More specifically, the objectives are to:

- Demonstrate if the aforementioned design is efficient and investigate how its behaviour is affected when it is altered, from a dynamical point of view.
- Determine the extent of the contribution to the dynamic response of the mass and stiffness of the piers.
- Compare the effects produced when several locations of the train are considered.

As it can be gathered from previous lines, this thesis limits its scope to study a particular bridge. Furthermore, due to the limited amount of time, it is also beyond the scope of this work the consideration of more than one train model. All the calculations were performed for high-speed load model A1, commonly denoted as HSLM–A1.

1.3 Methodology

First and foremost, an exhaustive literature review was carried out. Throughout this report, some of the most relevant state-of-the-art reviews are presented, with particular emphasis in those documents which subject of research focuses on structural dynamics of high-speed railway bridges, all the relevant design codes and tutorials.

After having identified the most appealing aspects, a particular bridge was selected as a case study: Ulla River Viaduct. Due to its complex typology, a full analysis of its dynamic behaviour was considered of great interest. Furthermore, the aforementioned alternative solution for closing its cross-section made it ideally suited for studying the influence of its torsional resistance of the deck.

To this end, a complete 3D finite element (FE) model was performed reproducing in detail the design drawings. Project information developed by IDEAM, courtesy of TYPSA as technical assistance engineering at construction.

The finite element (FE) method is arguably the most suitable way of analysing structures nowadays, both two- and three- dimensional. The commercial package available for the FE modelling was ABAQUS [10]. However, applying moving loads is not straightforward in this software, thus, the engineering tool MATLAB [11] was needed as a support.

For the dynamic analysis, the implicit integration scheme has been used. However, few checks in the frequency domain were performed as an endorsement for a better understanding of the obtained results.
The quality of the results is highly dependent of the underlying assumptions, especially when it comes to mass, stiffness and boundary conditions. Therefore, the refined quality assurance included static as well as dynamic checks/verifications to ensure that the model functioned adequately.

Finally, the core part of this work consisted of studying the influence of different aspects on the bridge dynamic response. Thereby, different FE models have been developed in which the following parts of the original bridge were altered:

- Non-connected prefabricated plates of the bottom slab.
- Torsional rigidity of the deck by considering open cross-sections.
- Mass and stiffness of the piers.
- Location of the railway track.

Subsequently, six different FE models were examined in which a high-speed train load model travelled over three different positions. The train speeds vary from 200 km/h to 420 km/h with a step of 20 km/h, yielding more than 120 dynamic calculations.

### 1.4 Assumptions and limitations

As it will be explained through the following chapters, the 3D FE model reproduced in detail the design drawings, seeking to simulate the real flexural and torsional rigidity of the deck and piers. Nevertheless, the following simplifications have been made:

- Only the central spans were modelled.
- No interaction vehicle-structure was contemplated.
- Each axle of the train was assumed to cause a concentrated load that moved along one straight axis (singly railway-track lane).
- No load distribution due to the existence of ballast or two rails was considered.
- Only HSLM–A1 was contemplated.

On the other hand, the dynamic calculations were based on direct integration within the time domain. This method requires of a very little time step if a realistic solution is sought, especially for structures with low damping such as the case of bridges, leading to a considerably large amount of resources in terms of computational memory and time. The several aforementioned simplifications helped to reduce these problems, but they could also diminish the accuracy of the results.

### 1.5 Outline of the thesis

Some knowledge of structural dynamics of HSR bridges is necessary beforehand to understand the overall study; therefore, the basic theory and pertinent design codes requirements are provided throughout chapters 2 and 3. Likewise, these chapters also present some of the most relevant state-of-the-art review.
Chapter 4 starts by describing in detail the Ulla River Viaduct, bridge selected as case study. This chapter then proceeds by explaining the FE models analysed and its validation by means of quality assurance checks/verifications.

All the results are shown and discussed in chapter 5. Finally, the 6th chapter includes the subsequent conclusions and suggest some aspects for future research in this field.
Chapter 2

High Speed Railway Bridges

2.1 Review of the special characteristics

Essentially five aspects may constitute the basic differences between bridges dedicated to conventional traffic and HSR lines, in terms of morphology and design [4]:

- The magnitude of vertical loads: railway traffic has approximately double value than conventional highway traffic.
- Location of loads: the position of the live loads may only occur over the track.
- Fatigue: repetitive loads, usually reaching its maximum values, may lead to fatigue problems.
- Braking and start-up forces.
- Dynamic effects that can be manifested in two ways. The first one is that the static effects are increased due to the effect of impact. This is particularly noticeable in railways-tracks in poor maintenance. On the other hand, this kind of bridges can be affected by the resonant phenomena (section 2.1.1).

2.1.1 Resonance and cancellation

Resonance can occur when the frequency of the external excitation coincides with the main natural frequencies of a structure. For railway bridges, the resonant phenomenon has been observed when the train is travelling at speeds exceeding 200 km/h, such as the case of HSR lines.

It is possible to quantify these effects using the wavelength of excitation $\lambda$, shown in eq. (2.1). It relates the train velocity $v$ to the fundamental frequency of the bridge $f_0$.

$$\lambda = \frac{v}{f_0}$$  \hspace{1cm} (2.1)

Resonance will appear when such wavelength of excitation $\lambda$, or any of its multiples, coincides with the separation of the axles of the given train $D_k$, as shown in eq. (2.2).
\[ \lambda = \frac{D_k}{f}, j = 1,2k \]  \hfill (2.2)

Experimental results of real bridges as well as mathematical models show that the most noticeable dynamic effects can be reached while the train is travelling and not necessarily when it has left the bridge [12].

The completely opposite phenomenon, known as cancellation, may also occur under particular circumstances in bridges with longer span than the train length and multi-span bridges with continuous deck. It results in a suppression of the dynamics effects. The reason is that several axles of the train will be passing over the bridge at different phases. An illustrative example can be found in Figure 2.1, that explains how certain train configurations can affect the dynamic response of a bridge, in terms of amplification or cancellation effects [13]. The plots show the maximum accelerations for 10 HSLMs and their normalised amplitude of free vibrations.

![Figure 2.1: a) Maximum vertical accelerations for the train load-model, b) normalized amplitude of free vibrations [13]](image)

As a result, it is possible to find the most optimal bridge span length-to-vehicle ratio. However, in real practice this is quite complicated due to the needs for interoperability, where more than one train shall be assessed. For curious readers, cancellation effects in the conception stage of bridge design have been addressed in [14], where the author proposes taking them into account in order to enhance the dynamic response. Moreover, in [15] an optimization process of cross-section for simply supported and single-track bridges was performed based on vertical accelerations.

### 2.2 Parameters that influence the dynamic response

The recent development of high-speed railway lines had led to the necessity of upgrade the existing infrastructures as well as the design of new ones. For this reason,
ascertaining the most influential parameters on the dynamic response has been the topic of numerous researchers during the last decades.

The European Rail Research Institute (ERRI) started the research carried out by Frýba [5], where the effects of some of the most relevant parameters affecting the dynamic response of a bridge were studied. The conclusions pointed out the train speed as the most significant parameter for the design, affecting deflections, bending moment and accelerations in a similar manner. It must be highlighted that as the span length $l$ (Figure 2.2.a), damping $\theta$ (Figure 2.2.b) or bridge weight $G$ increase, the vertical accelerations diminish. Finally, Frýba also stated that the dynamic response of concrete bridges is lower than for steel bridges due to their higher values of damping and mass.

\[ \text{a) Influence of span length } l \]

\[ \begin{array}{c}
\text{a/q (1)} \\
\mbox{ICE 2} - Eurostar - Talgo AV 2
\end{array} \]

\[ l (\text{m}) \]

\[ \text{b) Influence of damping } \theta. \text{ Case } a \text{ out of resonance and case } b \text{ at resonance} \]

\[ \begin{array}{c}
\theta (1) \\
\hline
\text{a} - \text{b}
\end{array} \]

\[ \text{Figure 2.2: Qualitative comparison of the effect of damping } \theta \text{ and span length } l \text{ in vertical accelerations under trains at 350 km/h [5]} \]

It is not always possible to know beforehand some important parameters such stiffness, damping or the fundamental frequency of a bridge, supposing their possible over- or underestimation. The safety of the structures underlies on the accuracy of the models,
and therefore, Spanish design codes require a static and dynamic testing. The results of such test, conducted in more than 119 bridges with different superstructures (from precast to posttensioned girders), allowed to establish a relationship between span length and fundamental frequency. Figure 2.3 clearly shows that the natural frequency tends to decrease as the span length increases. This tendency is apparently unrelated to the superstructure type [3].

![Figure 2.3 Measured first natural frequency for 119 bridges [3]](image)

The influence of number of spans is another popular subject of research. In that sense, vertical accelerations were found lower for multi-span bridges in comparison to single-span ones, as long as its natural frequencies were within the Eurocode limits [16]. On the contrary, short continuous bridges with lower natural frequencies were observed to yield higher accelerations [6].

Yau [7] demonstrated that continuous bridges lead to lower dynamic response than simply supported ones, and consequently, an enhanced dynamic behaviour. However, his observations also showed that more resonance peaks were produced (Figure 2.4). Continuous multi-span bridges are almost always preferable therewith. With this configuration, greater stiffness is achieved and more “noise” in the dynamic response, and thus, less resonance problems [4].

![Figure 2.4: Dimensionless speed versus impact factor $I = (R_{dy}-R_{sta})/R_{sta}$. Where $R_{dy}$ and $R_{sta}$ are the dynamic and the static responses respectively [7]](image)
2.2. Parameters that influence the dynamic response

2.2.1 Effect of torsional rigidity

Goicolea [17, 18] considered of especial importance the need of taking into account not only the bending but also the torsional component of bridges’ deck. This last component is particularly significant in those bridges with low torsional rigidity.

As a practical example, the case study of Las Piedras Viaduct, the first steel-concrete composite continuous deck bridge in the Spanish HSR line is presented in [18]. The authors studied its dynamic response and compared the results to those obtained when altering its torsional rigidity, by means of considering open cross-section. Finally, they concluded that when considering open cross-section the subsequent accelerations were inadmissible (Figure 2.5).

![Figure 2.5: Maximum vertical acceleration when considering two different configurations of the Las Piedras Viaduct cross-section [18]](image)

The importance of considering coupled bending-torsional effects in dynamic analysis of bridges using FE models was also investigated. The Río Cabra Viaduct, a continuous deck bridge composed of hollow slab deck cross-section was proposed as a case study [9]. The author studied the applicability of some simplified methods for considering the torsion by means of comparing the subsequent results to those obtained with a 3D complete FE model of coupled bending-torsion. It was also compared such results to those obtained when considering a box slab cross-section instead. This comparison revealed that in bridges with high torsional stiffness the advantage gained by considering coupled bending-torsion was insignificant, which is very interesting from a practical point of view.

Lastly, the work developed in [19] is also perceived as essential for these thesis purposes. The authors discussed the difference between several bridge typologies belonging to the Spanish HSR line. The explanation of why it is considered indispensable for the present study is that one of the bridges analysed is the Ulla River Viaduct, subject of our case study. Two different calculations were performed in parallel, whether considering or not the torsional modes. A significant increment in the dynamic response of such bridge was
observed when both mode types were considered. That is, reaching values five times as those obtained when computing only vertical modes in terms of displacements, and twice as much in terms of accelerations.

### 2.3 Longitudinal schemes

This section aims to summarize the most relevant aspects in terms of longitudinal schemes and deck morphologies/characteristics. To this end, the structures built up to this moment in China and Spain were accounted. Both countries have been already referred as those ones counting with the largest extension of high-speed network.

As commented previously in this work, the vast majority of bridges are composed of double railway-ballasted track. The deck’s width generally adopted is 14 m. The selection of their typology depends essentially on the span length, as it can be understood from Table 2.1 [20].

<table>
<thead>
<tr>
<th>Typology</th>
<th>Span length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab section</td>
<td>5</td>
</tr>
<tr>
<td>solid slab</td>
<td></td>
</tr>
<tr>
<td>hollow slab</td>
<td></td>
</tr>
<tr>
<td>Box section</td>
<td>5</td>
</tr>
<tr>
<td>constant depth</td>
<td></td>
</tr>
<tr>
<td>variable depth</td>
<td></td>
</tr>
<tr>
<td>Prefabricated</td>
<td>5</td>
</tr>
<tr>
<td>simply supported</td>
<td></td>
</tr>
<tr>
<td>hyperstatics</td>
<td></td>
</tr>
<tr>
<td>Composite</td>
<td></td>
</tr>
</tbody>
</table>

It is noteworthy the work done in [1], where a cost efficiency criterion is discussed for choosing the most appropriate structural forms in China. In general, for medium spans (100-200 m) tied steel arch, rigid frames or hybrid steel arch bridge with concrete girder are recommended. On the other hand, when it comes to long spans (>200 m) steel truss arch and cable-stayed bridge with truss girder are the preferred ones.

### 2.4 Envelopes and verifications

Bridge response under dynamic loads can vary mainly because of the three factors below:

- Different vertical loads in each point of the structure (vehicle speed and vibration of structure).
- Successive loads entering to the bridge at uniform and evenly spaced intervals.
- Track and vehicles irregularities.
Standing design codes [21, 16, 22, 23] usually consider the dynamic response of a railway bridge through the magnification of the dynamic response for a single moving load with respect to the static one, by the so-called dynamic factor or impact coefficient.

Nevertheless, due to the increment of speed and vehicle lengths, the resonant phenomena became a crucial issue to be taken into account, and thus, the only use of this factor were unacceptable [24]. For this reason, dynamic loads for high-speed traffic need to be assessed by a specific and complete dynamic analysis.

ERRI carried out a research in order to update the design codes. They defined a set of actions due to traffic loads that needs to be considered in the design of HSR bridges. Those include from horizontal and vertical loads to static and dynamic ones. Their combination together with all the other loads has to fulfil the ultimate state requirements.

### 2.4.1 Train load models

An aspect worth emphasising is the convenience of interoperability of different families of trains. Thus, the infrastructure needs to behave as desired when either of the existing trains is passing, in terms of dynamics effects. Since the worst effects are not necessarily reached for maximum train velocities, the referred effects shall be evaluated for all possible trains speeds as well. The current vehicles have a broad variation between axles, coach lengths, etc. and can be split in three categories (Figure 2.6):

- Articulated trains: THALYS, AVE and EUROSTAR
- Conventional trains: ICE2, ETR-Y and VIRGIN
- Regular trains: TALGO AV

![Figure 2.6: Different typologies of high speed trains according to [19]](image)

The consideration of all the trains might be quite time consuming and laborious. For this reason, together with the possibility of appearance of new train configurations, ERRI D214 committee established a High-Speed Load Model (HSLM) [25]. Being HSLM-B for bridges with span lower than 7 m and HSLM-A for all the rest.
consist of 10 different train load models that represent fictitious universal trains which signature (Figure 2.7) is the envelope of all the real high-speed trains.

![Figure 2.7: Envelope of dynamic signatures for all European high-speed trains and HSLM-A model [17]](image)

The characteristics of these universal fictitious trains are summarised in Table 2.2. The complete dynamic analysis should be performed for all 10 HSLM and considering series of speed up to 1.2xMaximum Permitted Vehicle Speed, every 10 km/h.

Table 2.2: Characteristics of HSLM-A

<table>
<thead>
<tr>
<th>Universal train</th>
<th>Number of intermediate coaches $C$</th>
<th>Coach length $D$ [m]</th>
<th>Bogie axle spacing $d$ [m]</th>
<th>Point force $p$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>18</td>
<td>18</td>
<td>2</td>
<td>170</td>
</tr>
<tr>
<td>A2</td>
<td>17</td>
<td>19</td>
<td>3.5</td>
<td>200</td>
</tr>
<tr>
<td>A3</td>
<td>16</td>
<td>20</td>
<td>2</td>
<td>180</td>
</tr>
<tr>
<td>A4</td>
<td>15</td>
<td>21</td>
<td>3</td>
<td>190</td>
</tr>
<tr>
<td>A5</td>
<td>14</td>
<td>22</td>
<td>2</td>
<td>170</td>
</tr>
<tr>
<td>A6</td>
<td>13</td>
<td>23</td>
<td>2</td>
<td>180</td>
</tr>
<tr>
<td>A7</td>
<td>13</td>
<td>24</td>
<td>2</td>
<td>190</td>
</tr>
<tr>
<td>A8</td>
<td>12</td>
<td>25</td>
<td>2.5</td>
<td>190</td>
</tr>
<tr>
<td>A9</td>
<td>11</td>
<td>26</td>
<td>2</td>
<td>210</td>
</tr>
<tr>
<td>A10</td>
<td>11</td>
<td>27</td>
<td>2</td>
<td>210</td>
</tr>
</tbody>
</table>

2.4.2 Envelope dynamic factor

2.4.2.1 Vertical train load model

Figure 2.8 shows the train load model LM -71 [23]. In order to calculate the dynamic factor, such model has to be considered by means of a static calculation. It aims to
represent the maximum effects that the railway traffic may cause on an infrastructure. For one railway-track loaded it is defined as follows:

a) Four axles \( Q_{vk} \) of 250 kN each, separated a distance of 1.6 m. Located in the worst position along the axis of the railway lane.

b) A uniformly distributed load \( q_{vk} \) of 80 kN/m. This load shall be applied in the worst situation (location and extension) for reaching the maximum absolute values of displacements.

\[ \text{Figure 2.8: Load Model LM-71 [23]} \]

### 2.4.2.2 Definition

From the static point of view, the Ultimate Limit State is verified through the dynamic factor \( \Phi_2 \) [16], also known as impact coefficient in the Spanish code [21]. Its definition is based on statistical analysis of existing bridges and it covers all the dynamic effects arising from railway traffic, including track imperfections. However, such effects cannot be split because they are derived from measurements. It may be calculated according eq. (2.3) for railway tracks well maintained.

\[
\Phi_2 = \frac{1,44}{\sqrt{L_\Phi}} + 0,82 \quad 1,00 \leq \Phi_2 \leq 1,67
\]

(2.3)

Where \( L_\Phi \) relates the determinant length defined in Table 6.2 in [16]. Accordingly, its application is limited to conventional structures, in other words, to structures included in this table.

Note that this dynamic factor \( \Phi_2 \) is only applicable when the design velocity is lower than 220 km/h and for fundamental frequencies within the limits specified in the Eurocode. This is because it omits the possibility of resonant phenomenon occurring and assumes that the maximum accelerations do not overpass the limiting values set in the referred code under any circumstances.

In the event that the aforementioned conditions are not satisfied, it results necessary to carry out a specific dynamic analysis. Nevertheless, in these cases, the same concept may be applicable by means of the envelope dynamic factor defined in eq. (2.4).

\[
\Phi = \max_{\frac{\sigma_{\text{real}}}{\sigma_{\text{nominal}}}} \frac{\sigma_{\text{dyn}}}{\sigma_{\text{static}}}
\]

(2.4)
Where,

- $\delta_{\text{static}}^{\text{nominal}}$ is the static maximum deflection caused by the load model LM-71, for two railway-track loaded.
- $\delta_{\text{dyn}}^{\text{real}}$ is the maximum dynamic deflection caused by real trains. To consider two trains travelling simultaneously, the values calculated for one railway-track loaded shall be combined using the square root of the sum of squares procedure (SRSS). In other words, the results are multiplied by $\sqrt{2}$.

The determination of $\delta_{\text{dyn}}^{\text{real}}$ not only includes the dynamic effects produced by the train loads themselves but also by the track irregularities. Its decomposition can be expressed as shown in eq. $(2.5)$.

$$\delta_{\text{dyn}}^{\text{real}} = (1 + \varphi' + \varphi'') \delta_{\text{sta}}^{\text{real}}$$

Where,

- $\delta_{\text{sta}}^{\text{real}}$ is the quasi static deflection caused by real high-speed trains. It may be calculated by considering a train speed slow enough.
- $\varphi'$ is a coefficient that represents the enhancement of the dynamic effect with respect to the static one. Its analytical expression is shown as follows:

$$\varphi' = \frac{K}{1 - K + K^4}, \text{ being } K = \frac{v}{2 L_\varphi f_0}$$

- $\varphi''$ is a coefficient that takes into account track irregularities. Values of $\varphi''$ are $\approx 0$ for long span bridges and low natural frequencies. It is defined in expression B.12 in [21]:

$$\varphi'' = \left[0.56 e^{-\left(\frac{L_\varphi}{10}\right)^2} + 0.5 \left(\frac{f_0 L_\varphi}{80} - 1\right) e^{-\left(\frac{L_\varphi}{28}\right)^2}\right]$$

Where again, $L_\varphi$ represents the determinant length and $f_0$ the fundamental frequency.

### 2.4.2.3 Treatment in different design codes

The requirements of the Eurocode [16] with respect to this factor $\Phi$ are in accordance to the Spanish code [21]. In this context, the main differences between them are that, for double railway-track bridges, the first one considers only one track loaded whereas the second one considers both tracks loaded. Another distinction rests in the values of the classification coefficient $\alpha$. In the Spanish code it represents an increment of 21% in the response caused by the train LM-71. This percentage was set aiming to keep the validity of the infrastructures if the vehicles increase weight in 30 tn. Finally, the Eurocode requires the application of extra loading models for continuous bridges or freight transportation in opposition to the only model LM-71 required in the Spanish code. Further definition and utilization of this coefficient according others European codes is addressed in the literature [12, 24].
2.4.3 Verifications

Traffic safety is ensured when the limitations related to the parameters listed below are satisfied [16]. The most relevant ones for this thesis are addressed in the following subsections.

- Vertical acceleration of the deck.
- Torsion of the deck.
- Vertical deformation of the deck.
- Horizontal deformation of the deck.

2.4.3.1 Vertical accelerations

The vertical accelerations are limited to $3.5 \text{ m/s}^2$ for ballasted tracks and $5 \text{ m/s}^2$ for un-ballasted ones, in order to ensure ballast stability. For its calculation, it is necessary to consider frequencies up to the greatest value of either 30 Hz or 1.5 times the fundamental frequency but including at least the three first oscillation modes. These frequencies should be filtered according to the acceleration criterion, and can be done through any of the following methods [26]:

- Suppressing all modes with higher frequencies, but keeping enough mass.
- Filtering through the time step integration election.
- Filtering the whole complete calculation.

2.4.3.2 Vertical displacements $\delta$

The serviceability limit state criteria establish an upper bound of the maximum vertical displacements $\delta$ shown in Figure 2.9. However, this plotting is used for simply supported bridges with three spans or more, and in case of continuous beams they should be multiplied by a coefficient of 0.9.

As a final remark, according to the relevant Spanish codes, if the span length of the examined bridge is greater than 10 m, a dynamic and load testing should be performed to verify the accuracy of the calculation models.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure29.png}
\caption{Maximum permissible vertical deflections for bridges for railway traffic corresponding to a permissible vertical acceleration of 1 m/s$^2$, figure A.2.3 of the code [16]}
\end{figure}
2.4.4 Amplification factor

The amplification factor, also known as the deformation response factor, represents the amplification in the displacements caused by dynamic effects. It is defined as the ratio of the dynamic displacements $\delta_{\text{dyn}}$ to the static one $\delta_{\text{sta}}$. Figure 2.10 shows this factor for an undamped system subjected to a harmonic force plotted against the frequency ratio $\omega/\omega_n$. Where $\omega$ corresponds to the forcing frequency of excitation in rad/s and $\omega_n$ the natural frequency of the system. Note that for resonant frequency this factor becomes maximum under resonant circumstances [27].

![Figure 2.10: Amplification factor (Deformation response factor) for an undamped system subjected to a harmonic force [27]](image)
Chapter 3

Dynamic analysis of bridges under high speed trains

3.1 Introduction

There are available several dynamic analysis methods for assessing railway bridges:

1. Static calculation through the dynamic factor or impact coefficient.
2. Time integration of the dynamic equations for the structure. In general, by considering one of the following models: dynamic calculation by moving loads or by vehicle-bridge interaction.
3. Simplified models using the dynamic train signature.

Such calculations are usually easy to apply to simple and isostatic structures. In such structures, the first eigenmodes predominate, permitting to characterise their dynamic response. On the contrary, when it comes to hyperstatic structures, more complex analysis methods are needed. That is because many vibration modes contribute to its dynamic response [9].

The first method is based on calculating a dynamic factor \( \Phi \) (section 2.4.2) that shall be applied to the envelope of the internal reactions obtained from a static analysis. The bridge then can be designed with these new values of internal reactions. Due to its limited applicability to train speeds \( v \leq 220 \text{ km/h} \), a specific dynamic analysis is needed for HSR.

The dynamic train signature can be understood as a function that describes the dynamic effect caused by a train in a bridge. Such function is characterized as a combination of damped harmonics. This fact limits its applicability to simply supported bridges that can be defined, in dynamic terms, by their fundamental oscillation mode [17].

For all these reasons, this chapter focuses in the calculation method: time integration of the dynamic equations for the structure. To facilitate the explanation, the following paragraphs treat the vehicle-bridge interaction problem as part of the moving load problem. In other words, it expounds the three different moving load problems that are generally discussed in the literature: moving mass problem, moving load problem itself
and moving oscillator (vehicle-bridge interaction). The general procedure to solve the moving load problem is by means of FE method.

### 3.2 FE models for torsional considerations

The FE models can be 2D or 3D. Eccentric loads, as it is the case of only one train travelling over a double railway-track bridge, produce deformations not only in terms of flexion but also of torsion. For this reason, an adequate calculation FE model that allows capturing the deformations for all the relevant modes involved is crucial. In other words, the coupling between bending and torsion must be properly captured. This section contains the type of models and elements more recommendable, taking into account these torsional considerations.

#### 3.2.1 Two-dimensional models

It omits the transversal direction; a plane (typically \(X-Z\)) embeds the structure that only varies along its longitude \(X\). It is the simplest way of modelling and it can contain a combination of shell and wire elements.

There exists a broad range of elements to be used in such models: two-dimensional continuum solid elements, truss elements or beam elements, depending on the structure geometry and characteristics. The very simplest way of modelling bridges in 2D is using a beam models.

Beam elements are also separated in sub-families depending on whether the cinematic hypotheses adopted is Euler-Bernoulli or Timoshenko. The selection of one of these sub-families rests in the slenderness span length \(l\) to thickness \(t\) ratio. The recommendations according to [28] are presented as follows:

- \(l/t \geq 10\) Bernoulli beam elements
- \(2 \leq l/t < 10\) Timoshenko beam elements

Railway bridges usually present slenderness higher than 10/1 and the shear deformation for span lengths superior than 15 m is not significant. This supposes that the most recommended elements are Bernoulli beam elements, being the base for many dynamic simplified models. It is possible to assess the torsional effects in a simplified manner by using a bridge beam model which associated DOFs shall include the torsional twist of the transverse section (Figure 3.1). Due to the definition of its nature, such models are still treated as 2D. The torsional response of the structure shall be included specifically in the dynamic calculations, generally introducing the effects produced by the eccentricity \(e\) of a load \(p\) [12].

20
3.2.2 Three-dimensional models

The model is embedded in the coordinate system $X$, $Y$, $Z$ and can be a combination of 3D solid, shell and wire elements. These models are necessary for assessing the torsional effects in non-conventional bridges, in other words, bridges that cannot be reduced as a 2D beam model. The torsional effects are included directly by defining the loads in the space.

For dynamic calculations, the torsional modes of vibration shall be captured properly. If the transversal and torsional stiffness are overestimated, the model will yield higher torsional modes and therefore its contribution to the dynamic response could be undervalued. Figure 3.2 shows the deformed mesh of a detailed 3D FE model of the Río Milanillos viaduct after having applied a torsional moment to the deck. This bridge belongs to the Spanish Segovia-Garcillán HSR line and it is composed of a two box beams separated a distance of 7 m and joined together through a flexible deck. The resulting deformation revealed that the transversal section cannot be defined by the hypothesis of rigid body motion, which is the same as saying that the transversal stiffness is not infinity [9].

3.3 Dynamic analysis models for structures and vehicles

A train crossing a bridge is treated as a moving load problem, where the load location varies along time. The so-called vehicle-bridge interaction problem is so far one of the most studied type of moving load problem. It is handled as a moving elastic subsystem over a primary elastic system. While this kind of problems is well defined in terms of
numerical and mathematical models, their application in commercial calculation software is usually limited [29].

Various typical moving load problems are presented as an easy following tutorial in [30]. The aforementioned work also describes a brief but complete state-of-the-art review, starting from Fryba’s monograph [31] and including a great deal of literature that covers several types of bridges, making its reading very enjoyable and highly recommended.

Even though it is not a popular subject of investigation, some researchers have also studied the effects of separation and reattachment of the moving subsystem from the supporting structure. That may occur when the travelling speed of the first one is high enough, causing an impact during the reattachment. An overview of this problem is exposed in the literature [32, 29] and it is considered of interest for the case of high-speed trains.

The three approaches of moving load problem (moving mass problem, moving load problem itself and moving oscillator) are explained in detail in the following paragraphs.

3.3.1 Moving mass problem

In this case the loads are simulated by point masses moving along the bridge (Figure 3.3). The moving mass problem is the most intuitive and straightforward way of modelling moving loads using ABAQUS [10]. The following steps must be taken [29]:

1. Model the primary structure (i.e. deck: shell elements).
2. Model the secondary moving structure (i.e. point mass) and apply a non-structural inertia and its corresponding weight as a load.
3. Select an appropriate contact between both of them. In ABAQUS it is possible to specify Node-to-surface contact with frictionless tangential behaviour and hard contact in the normal direction. Such contact allows capturing the separation and reattachment of the moving structure.
4. Apply the speed of the moving structure as a constraint, by either prescribing velocity or time history displacements.

![Figure 3.3: Moving mass problem [33]](image)

On the other hand, the drawbacks of this modelling approach, among others, are:

- The coupling stiffness between the primary and secondary systems are assumed infinity.
- In order to apply more than one moving mass, each one has to be modelled separately with its corresponding contact properties, resulting in an awkward and time-consuming way of modelling moving loads.

3.3.2 Moving load problem

This approach assumes that the axles of the train cause a constant concentrated load that moves along with it. Thus, the vibrations produced in the vehicle are neglected as well as its inertia effect. Nevertheless, in the literature it is considered accurate enough for practical applications [9].

In this respect, the vehicle is treated as a set of concentrated loads $p_j$ separated a determined distance depending on the geometry of the given train (Figure 3.4) which travels at a constant speed $v$ as can be schematized in Figure 3.5.

![Figure 3.4: Moving loads pattern of a real train. Each axle force $p_j$ is separated a distance $d_k$](image)

There are basically two available ways of applying moving loads in ABAQUS and its implementation is based on defining load histories that are applied to each node. The first one is using the already implemented subroutines (i.e. DLOAD) using a Fortran compiler.

![Figure 3.5: Moving load problem. Concentrated loads $p_j$ travelling at a speed $v$](image)

The second option is applying the time history loads as tabular amplitudes. First of all, it is necessary to define such tabular amplitudes following the steps below:

1. Numerate the nodes that belong to the load trajectory.
2. Define a reference time $t_0$ that represents the initial time where the first axle of the train enters the bridge.
3. Determine the instant $t_i$ that corresponds to the arrival time of the first axle of the train to each node $j$. The axle force $p_j$ causes triangular amplitude to this node as shown in Figure 3.6.

4. Determine the arrival time steps $t_i$ of the posterior axles of the train to each node $j$ and its corresponding triangular amplitudes.

![Figure 3.6: Load history definition of an axle force $p$ travelling at a speed $v$ and causing a triangular amplitude to each node $j$][9]

### 3.3.3 Moving oscillator (sprung mass)

The moving oscillator is the most realistic approach; the coupling stiffness is finite and considers inertial effects of the moving structure. The train is represented by point masses, bodies and springs as shown in Figure 3.7.

![Figure 3.7: Moving oscillator. The axle loads $p_j$ combine masses and springs][33]

These kinds of models can be complete or simplified. In both cases they are represented by sprung and unsprung masses as well as by the primary suspension of each axle. The completed ones also consider the length, mass and inertia of the bogies, vehicle body geometry and secondary suspension of bogies.

The interaction between vehicle and bridge is accounted, therefore the complexity of the model increases considerably and so the computational time does. That is why they result highly interesting for research aims but not useful for many practical applications [9].
Furthermore, according to Gabaldón [24], for long span bridges or continuous deck bridges, the accuracy gained by using interaction models are usually very small. For these reason, their application are only when the passenger comfort inside the vehicles is evaluated, but is not considered necessary for design purposes.

### 3.4 Dynamic analysis approaches

FE method can generally be applied to any arbitrary structure, even if non-linear effects are considered. It is based in the discretization of the structure into elements in space and nodal coordinates in time, yielding to a \(N\)-degrees of freedom (DOFs) system of equations shown in eq. (3.1).

\[
M\ddot{u} + C\dot{u} + Ku = f(t)
\]  

(3.1)

Where,

- \(M\) is the mass matrix
- \(C\) is the damping matrix
- \(K\) represents the stiffness matrix
- \(f(t)\) is the external loads vector
- \(u\) is the nodal displacements vector, unknown

These equations are usually coupled and therefore need to be solved simultaneously. The solution is given by multiplying the nodal coordinates by shape functions.

The dynamic analysis of the structure may be performed either through the integration along time of the \(N\)-DOFs system or via modal superposition (MS) method, separating space from time in the coupled equations [27].

### 3.4.1 Time domain

This procedure solves the whole set of \(N\) differential equations for each time step by means of direct integration. Since these equations are coupled, they need to be solved simultaneously. The stability of the solution of this problem depends only on the transient response because no steady state solution exists.

#### 3.4.1.1 Explicit schemes

Use the equation of motion at a time \(t = t_{i+1}\) plus assumptions to find the solution at a time \(t = t_i + \Delta t_i\). In other words, at the end of each time step the matrices are updated and the system of equations is solved. Those methods are conditionally stable with respect to the time step election, if the increments are small enough the solution will be accurate, otherwise it will diverge because the equilibrium is not enforced.
3.4.1.2 Implicit schemes

In this case, uses the equation of motion at a time \( t = t_i - \Delta t_i \) plus assumptions to find the solution at \( t = t_i \). After each increment, the equilibrium is enforced by iterating with the Newton-Raphson algorithm. Consequently, it allows using larger time steps and the accuracy is generally higher than the explicit case.

Nevertheless, the mesh selected has to be coarse enough to capture the deformation of the structure and the time step small enough to capture the higher frequency sought.

3.4.1.3 Time step selection

Recommendations for selecting the time step can be found in the literature [12]. Some of them are summarized below in this subsection, and are differentiated according to the aspect considered for the dynamic analysis. In this context, the time step \( \Delta t \) can be taken according to:

- Higher natural frequency \( f_{\text{max}} \) of the bridge to be accounted:
  \[
  \Delta t \leq \frac{1}{8 f_{\text{max}}}
  \]  
  (3.2)

- Minimum number of time intervals that an axle needs to travel across the shortest span \( l_{\text{min}} \) of the bridge at a speed \( v \):
  \[
  \Delta t \leq \frac{l_{\text{min}}}{200 v}
  \]  
  (3.3)

- Number of natural frequencies \( n \) considered, length of the shorter span of the bridge \( l_{\text{min}} \) and train speed \( v \):
  \[
  \Delta t \leq \frac{l_{\text{min}}}{4 n v}
  \]  
  (3.4)

3.4.2 Frequency domain

For systems with a large number of DOFs the computational effort for solving simultaneously may be very extensive. Performing a modal analysis using the modal-superposition method allows solving a reduced system of uncoupled equations, considering only \( n \) (\( n \ll N \)) oscillation modes. The procedure consists of transforming these equations to its modal coordinates expressing the subsequent solution in terms of modal contributions [27].

The accuracy of the solution, hence, underlies on the number of modes considered. Calculating moment and shear forces requires a larger number of modes than displacements and accelerations, due to the major contribution of higher oscillation modes.
Note that since this approach expresses the response as a superposition, it is only applicable for linear elastic structures, as it is generally the case of structural dynamics of bridges.

### 3.4.2.1 Fast Fourier transform (FFT)

Any arbitrary periodic signal can be expressed in either time or frequency domain, being the last one the so-called spectrum of a signal. Such spectrum may be constructed as a linear combination of different periodic functions associated to a different frequency (Figure 3.8). This transform is already implemented in MATLAB [11].

![Figure 3.8: Fast Fourier Transform](http://mri-q.com/fourier-transform-ft.html)

Nyquist frequency, expressed in eq. (3.5), establishes the limit frequency necessary to reconstruct a signal, the sampled frequency should be greater.

\[ Nyquist\ frequency = \frac{1}{2T} S_n \]  

(3.5)

Where $S_n$ relates the number of samples recorded in a total time $T$.

---

1 Figure retrieved from http://mri-q.com/fourier-transform-ft.html
Chapter 4

Case study: Ulla River Viaduct

4.1 Introduction

In the present chapter, a case study is presented, where the methodology explained up to this point will be applied. The structure selected for this study is the Ulla River Viaduct (Figure 4.1), which constitutes the most remarkable intervention for the High-Speed Atlantic Railway Line [34]. It is worldwide recognised and resulted finalist for the 2016 Outstanding Structure Award given by the International Association for Bridge and Structural Engineering (IABSE). It is located in Galicia, Spain, as shown in Figure 4.2. IDEAM was responsible of the design project and TYPSA oversaw the technical assistance at construction, but many other companies were involved in the entire project.
The reasons for choosing this structure were:

- Its complex typology made this structure attractive for a fully study of its dynamic behaviour.
- The torsional circuit of the deck is closed by an alternative solution different to the typically employed in bridges with similar typologies.
- Detailed information of the bridge, including design drawings and testing results of the real built bridge, has been provided.

\[\text{Figure 4.2: Location of Ulla River Viaduct}\]

\[\text{Figure 4.3: Elevation view of Ulla River Viaduct}\]

4.2 Description of the structure

The structure has a total length of 1620 m, distributed in 12 spans of 225+240+225 m the main ones, several approaching spans of 120 m and side spans of 50 m and 80 m (Figure 4.3).

Given the project constraints, seeking to minimize the number of piers and avoid the environmental impact to certain extent, the resultant solution is composed of a steel lattice composite bridge, with double steel-concrete composite action near the supports.

The cross-section is variable in depth, from 17.5 m near the supports and 8.75 m at midspans. The total width is 14 m, and it is equipped with two ballasted railway-tracks. The concrete top slab has its maximum depth of 0.77 m over the upper chords axis, the minimum one of 0.39 m in the edges and a central part of 0.46 m. The two steel lattices are rotated approximately 45° with respect to the horizontal, the upper chords are separated a distance of 6 m to each other. The characteristic cross-section is shown in Figure 4.4. More information about materials and cross-sections can be found in Appendix A.
4.2. **Description of the Structure**

In the main central spans, the piers are rigidly connected to the deck resulting in frames. This configuration gives the sufficient stiffness required for the passing trains. The two central piers (P-6 and P-7) are hollow boxes cross-section, whereas the two resting ones (P-5 and P-8) are non-connected shafts, allowing the movements due to imposed displacements.

The double composite action near the supports is achieved thanks to the cast-in-place bottom slab (Figure 4.5). Where this action is not required, non-connected prefabricated plates of 2 m long are resting on the bottom chords (Figure 4.6). This configuration results in an increment of the torsional rigidity of the deck due to the closed section.

**Figure 4.4**: Characteristic cross-section of Ulla River Viaduct

**Figure 4.5**: Views of the bottom slab taken from the real bridge
4.3 FE models

Due to the complexity of the structure together with the aim of the study, the 3D FE base model (hereon named as MODEL 1) reproduced in detail the design drawings of the bridge. Several parts of it were altered for study purposes. Accordingly, six different FE models and three different train load positions have been examined.

ABAQUS [10] is composed of separated modules dedicated to the different aspects necessary in the modelling process. In that sense, for developing the FE model was required to define the geometry, material, constraints, loads and generate the mesh using the modules Part, Property, Assembly, Step, Interaction, Load and Mesh in that order. While moving along the modules an input file was created and modified to generate the moving loads. It was thereupon submitted to calculation.

Dynamic analyses using this commercial package can be quite time and memory consuming. In this context, MODEL 1 resulted in 104237 elements, 143137 nodes and a total number of 674292 variables in the model, including all degrees of freedom plus maximum number of any Lagrange multiplier variables. Nevertheless, one of its benefits that it is possible to select only a few nodes to store the results and use multiple processors in parallel leading to a noticeably computing time and computational memory reduction.

This section organization starts with a detailed description of the MODEL 1, including the geometry, mass and cross-section and finally some aspects related to the mesh. It proceeds later by summarizing the total six models and train load locations.

4.3.1 Assumptions and limitations

All models only considered the main central spans and two approaching spans. In this respect, the models comprised from pier P-4 to pier P-9 (Figure 4.3). The greater dynamic effects were expected in these central main spans, but on the other hand, the excitation caused by the train when entering and leaving the bridge is always higher in the extreme spans, thus, some approaching spans had to be also modelled.
For simplification, the bracings of the upper truss have been neglected, among other reasons, because their placement is mainly related to construction purposes and not so much to its dynamic response. Soil-structure interaction contribution has been also neglected and the stiffness of the piers P-4 and P-9 (see Figure 4.3) was assumed to be infinite in the vertical direction.

The railway track was assumed to be one beam, which is the same as saying that both wheels of the train were concentrated in one axle.

### 4.3.2 Geometry and materials

This subsection describes the geometry and materials employed in MODEL 1. The materials remained the same for all the other models, whereas the geometry could vary. Both geometry and materials of the model were based on the design drawings, see Appendix A for further detail.

The following members of the bridge were modelled in separated parts: truss, top slab, bottom slab, head, base and body of the piers. Figure 4.7 shows the geometry of the assembled MODEL 1.

![Figure 4.7: Elevation view of the assembled parts of MODEL 1 in ABAQUS](image)

The modal damping ratio $\xi$ applied to all the modes during the simulation is 0.005, according to Table 4.1. Moreover, the mass- and stiffness-proportional Rayleigh damping were introduced as $\mu$ and $\beta$ respectively, calculated as shown in eq. (4.1).

$$
\mu = \frac{2\omega_i\omega_j(\xi_j - \xi_i)}{\omega_i^2 - \omega_j^2}, \quad \beta = \frac{2(\omega_i\xi_j - \omega_j\xi_i)}{\omega_i^2 - \omega_j^2}
$$  \hfill (4.1)

Where,

- $\omega_i$ and $\omega_j$ are the cyclic natural frequencies of the $i$ and $j$ modes, in rad/s
- $\xi_i$ and $\xi_j$ are the critical damping ratio for the $i$ and $j$ modes, in this case it has been considered 0.5% for all modes (Table 4.1).
### Table 4.1: Lower bound values of the structural damping to be used in design of railway bridges [16]

<table>
<thead>
<tr>
<th>Bridge Type</th>
<th>Span $l &lt; 20 \text{ m}$</th>
<th>Span $l \geq 20 \text{ m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel and composite</td>
<td>$\xi = 0.5 + 0.125 \ (20 - l)$</td>
<td>$\xi = 0.5$</td>
</tr>
<tr>
<td>Prestressed concrete</td>
<td>$\xi = 1.0 + 0.07 \ (20 - l)$</td>
<td>$\xi = 1.0$</td>
</tr>
<tr>
<td>Filler beam and reinforced concrete</td>
<td>$\xi = 1.5 + 0.07 \ (20 - l)$</td>
<td>$\xi = 1.5$</td>
</tr>
</tbody>
</table>

The altered density of the concrete top slab accounted the dead loads of non-structural elements such as ballast, edge beams, railway-track and safety equipment. More precisely, such dead loads have been accounted according to [4], considering that the weight of two ballasted tracks and all bridge finishes has typical values of $120 \text{ kN/m}$. This weight was assumed to be concentrated in the central part of the deck, to be more realistic, where the ballast was resting (Figure 4.8). Accordingly, the total density of the concrete top slab was $31750 \text{ kg/m}^3$ distributed in an area of $3 \text{ m}^2$ with density $\rho_1 = 2500 \text{ kg/m}^3$ and $4.36 \text{ m}^2$ with density $\rho_2 = 5552 \text{ kg/m}^3$.

![Figure 4.8: Equivalent density distribution over concrete top slab](image)

### 4.3.3 Boundary conditions

Despite the only modelling of the central spans of the bridge, the continuity of the real one should be simulated by applying boundary conditions.

According to the design drawings, the supports in piers P-4 and P-9 allow displacements in direction $u_1$ (longitudinally) and $u_1$ and $u_2$ (transversally) each, as shown in Figure 4.9. A realistic model would have taken into account the stiffness of the POT supports and piers, in their respective direction. However, for simplification, the boundary conditions in these extreme nodes were considered as free in all DOFs but $u_3$ (vertical direction). That simplification led to significant lower eigenfrequencies than if a spring support, with the correspondent stiffness, would have been applied instead.

Finally, the restraint of the piers from the foundations was assumed as fixed (all twists and displacements kept).
4.3. FE MODELS

a) Plan view Ulla River Viaduct. Central spans

b) Allowed displacements in supports P-4 and P-9

Figure 4.9: Supports piers P-4 and P-9

4.3.4 Loading

For the static calculations, the concentrated loads corresponding to the train load models were applied as pressure loads to small surfaces (Figure 4.10). This led to a more realistic approach load-structure interaction.

Figure 4.10: Concentrated loads modelled as pressure in surfaces

On the other hand, the moving loads were directly applied as concentrated loads to the nodes of beam elements that simulated the railway track.

4.3.5 Mesh

Different members of the bridge model were associated to a particular mesh element. In order to capture with accuracy the structural response against dynamic loads, the models combined beam, shell and solid elements.
The lattice members were represented by beam elements rigidly connected to each other simulating the welding unions. The element type chosen was B31, which uses linear interpolation, based on Euler-Bernoulli beam theory but allows specifying transverse shear strain (becoming suitable for both thick and thin beams).

Conversely, despite allowing large magnitude axial strains, it is assumed to be small when computing torsional shear strain. For thick beams this could be an important issue in dynamic analysis, but for slender beams is usually insignificant. In any case, the rotatory inertia does not affect to the same extent that shear deformation effects.

Regarding the concrete members, the shaft and head of the piers were shell elements, connected by solid elements capable to transfer the forces in all directions. Likewise, the top slab was composed of shell elements accurately partitioned longitudinally to reproduce the variable transversal depth. Finally, the cast-in-place and precast bottom slabs, when considered, were also shell elements. The shell elements available were: linear triangular (SR3) and quadrilateral (SR4), quadratic triangular (STRI65) and quadrilateral (S8R). SR4 is a general-purpose shell element that is perfectly suitable for thick and thin shell models in terms of robustness and accuracy, and for that reason was employed in all shell elements.

Choosing the most appropriate mesh size required a convergence analysis, but some aspects needed to be studied carefully when meshing. ABAQUS offers a wide range of constraints modelling. In order to prevent the relative movement between two separate edges or faces, the tie constraint is the preferred one for its generality. It is based in a strict algorithm master-slave. Two approaches can be specified for the discretization method: surface-to-surface and node-to-surface; where only the master is treated as a surface. The slave nodes pass through the normal direction of the master surface and can be penetrated by them, for that reason the surface mesh has to be coarser.

The mesh size of the diagonals was taken large enough to be simulated by only one element each. These members were not subjected to significant bending moments or shear deformations. Due to the interaction between the chords and the concrete slabs, the mesh of the first members was thinner: element size of 0.5 m. However, due to the configuration of the bridge, these members were not subjected to significant bending moments or shear deformations either.

The mesh size for the solid elements that simulates the base of the piers was taken as 0.8 m, the size of the elements composing the body of the piers was thinner: 0.5 m. The concrete top slab mesh size was 0.8 m. Finally, the shell elements of the bottom slab had a size of 1 m.

4.3.6 Different models

The object of the present study is to study the dynamic response of the bridge when the following aspects were altered:

- Bottom slab configuration, considering open and closed cross-sections. The original one is shown in Figure 4.11.
- Mass and stiffness of the piers.
As a consequence, six different models were necessary, hereon denominated as MODEL 1, MODEL 2, MODEL 3, MODEL 4, MODEL 5 and MODEL 6 and which characteristics are described throughout this section. See Appendix A/section A.3 for further detailed description of the models.

- **MODEL 1:**
  The most realistic and detailed model reproduced with fidelity de design drawings of the bridge. The original configuration of the bottom slab was respected (Figure 4.11). The cast-in-place bottom slab was variable in depth and rigidly connected to the bottom chord. All shell elements of the prefabricated plates were non-connected to each other and simply supported over the bottom chords, what means no rotational degrees of freedom allowed in the interaction between these parts. Figure 4.6 shows a 3D view of the central span of MODEL 1 and a zoom of its middle part.

- **MODEL 2:**
  This model did neglect the contribution of the non-connected prefabricated plates of the bottom slab. Therefore, only the cast-in-place bottom slab was modelled (see Figure 4.11), keeping the double steel-concrete composite action near the supports. It was performed to study two aspects: 1. if the only length of cast-in-place bottom slab was sufficient to transfer the torsional flow to the piers and 2. to determine up to what extent the placing of the prefabricated plates affected its dynamic response. Figure 4.13.a shows a zoom of the central span of this FE model.
• MODEL 3:
This model aimed to determine if the discontinuity in the prefabricated plates of the bottom slab offered more advantages than if those plates were connected to each other, from a dynamic point of view. Consequently, the bottom slab was continuous along all the span length as it can be discernible from the zoom presented in Figure 4.13.b.

• MODEL 4:
In this model, an open cross-section of the deck was considered, decreasing noticeably its torsional rigidity. For this purpose, neither cast-in-place nor prefabricated plates of the bottom slab were modelled (see Figure 4.13.c). It is worth highlighting that in this configuration the double steel-concrete composite action near the supports was supressed, affecting the flexural longitudinal behaviour as well.

Figure 4.13: Zoom of the central span of: a) MODEL 2, b) MODEL 3 and c) MODEL 4

• MODEL 5:
To determine how much the mass of the piers contributed in the dynamic response of the bridge, this model assumed null density for the concrete of the piers.

• MODEL 6:
This model did not consider the body of the piers. On the contrary, the head of the piers that allows the rigid connection to the truss remained. The boundary conditions applied (as fixed) resulted in a configuration like a continuous beam with fixed intermediate supports.

4.3.7 Railway track locations
As it can be drawn from subsection 4.3.6, the torsional response of the bridge in front of dynamic loads plays a crucial role, and therefore the position of the railway-track must excite it. For this purpose, three different configurations were contemplated (hereon denominated as Position A, Position B and Position C), accounting one or two trains travelling in the same direction depending on the case. According to Figure 4.14:

- Position A: One train travelling directly over one upper chord axis – Train load (1).
- Position B: Two trains travelling in the same direction, each of them directly over the axis of its respective upper chord. This configuration did not excite torsion – Train loads (1) and (3).
- Position C: One train travelling across the centre of the deck; this position does not excite torsion either - Train load (2).
4.4 Model validation

Incorrect inputs lead to poor results, especially when a dynamic analysis is involved. Some relevant aspects such as mass, stiffness or boundary conditions may affect considerably the subsequent results. For that reason, the quality control of MODEL 1 included checks/verifications from a static as well as dynamic point of view. The available information to validate the model was the static loading testing of the real bridge plus its expected natural frequencies and mode shapes.

4.4.1 Mass check

The total weight of the model was 76360 t corresponding to a relative error of 6% from the expected weight calculated according to the drawings (75905 t). This error was considered acceptable on the part of the author. When neglecting the weight of the piers, the total mass of the model was reduced to 59905 t, which equivales to 65 t/m.

4.4.2 Static loading test

MODEL 1 was also verified against experimental measurements regarding static deflections. In [35] the results obtained in the static loading testing of the real viaduct are provided. Several locations of the train were considered in order to achieve the maximum deflection for each span. The measurements were obtained using displacement transducers with an expected minimum accuracy of 5%, located in the following points, referred to the corresponding span and pier numbering shown in Figure 4.15:

- Span 6: Bottom chord, midspan. Approximately at 112.5 m from P-5 and P-6 pier axis.
- Span 7: Upper chord, midspan. Approximately 120 m distance far from P-6 and P-7 pier axis.
- Span 8: Bottom chord, midspan. Approximately at 112.5 m from P-7 and P-8 pier axis.
The railway vehicle used in the loading testing is composed of four sets of one locomotive 333.3 and a maximum of six hoppers RENFE-80T (see Appendix C, Figure C.1). The loading positions are defined in Appendix C, they accounted two tracks loaded in all the cases. Due to the only modelling of the central spans four loading positions were checked, named from loading position 11 to 14.

The measured displacements together with the calculated ones are shown in Table 4.2.

<table>
<thead>
<tr>
<th>Loading position</th>
<th>Span</th>
<th>Measured [mm]</th>
<th>Calculated [mm]</th>
<th>$e_r$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>6</td>
<td>-98.48</td>
<td>-96.50</td>
<td>2.05</td>
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<td>8</td>
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<td>12</td>
<td>7</td>
<td>-72.68</td>
<td>-88.18</td>
<td>17.58</td>
</tr>
<tr>
<td>13</td>
<td>6</td>
<td>-95.58</td>
<td>-83.26</td>
<td>14.80</td>
</tr>
<tr>
<td></td>
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</tr>
<tr>
<td></td>
<td>8</td>
<td>-86.98</td>
<td>-83.48</td>
<td>4.19</td>
</tr>
</tbody>
</table>

### 4.4.3 Modal analysis

A 2D plane truss model was developed by IDEAM [35] to determine the first natural frequencies of the bridge, aiming to compare the results calculated with the experimental ones obtained during the dynamic load testing. In this respect, the dynamic load testing should aim to excite the first bending mode of the main span.

Despite the absence of experimental results, the calculated theoretical first bending mode of the main span was 0.765 Hz. The respective mode obtained in ABAQUS for MODEL 1 was associated to a frequency of 0.813 Hz, resulting in a relative error of 6.27%, which was considered within acceptable limits.
On the other hand, TYPSA provided information about the first 120 natural frequencies and mode shapes that were employed to study the influence of the torsional modes on the dynamic response of the bridge [19]. The FE model (Figure 4.17) was performed using the software SAP 2000 [36]. The comparison of natural frequencies between both models is shown in Appendix B/section B.2.

![Figure 4.17: SAP 2000 model of the bridge developed by TYPSA](image)

### 4.4.4 Boundary conditions

As explained in subsection 4.3.3, the boundary conditions in the side spans allowed the movements in all directions except the vertical one. In order to analyse the effect of this assumption in the overall dynamic response of the bridge, the movements in the transversal direction were restrained in a modified MODEL 1. The obtained results in terms of accelerations and displacements for this modified MODEL 1 and the original MODEL 1 were compared (Figure 4.18, Figure 4.19).

![Figure 4.18: Accelerations for MODEL 1 and MODEL 1 modified boundary conditions](image)

![Figure 4.19: Displacements for MODEL 1 and MODEL 1 modified boundary conditions](image)
One might note that despite the undertaken simplification highly affected the oscillation modes, the dynamic response when a train HSLM-A1 was travelling across the bridge was almost identical. The peak observed for velocities around 280 km/h in MODEL 1 was not produced in the modified MODEL 1. However, the author assumed that this very small difference would not have affect the final conclusions and thus could be neglected for this study.

4.5 Total simulation and time step selection

The implicit integration scheme (see section 3.4) has been employed for the dynamic calculations in ABAQUS. The stability of the solution for a moving subsystem over a primary one is determined by the transient response [29].

ABAQUS/Standard allows to specify transient fidelity categorise in the direct integration step. In applications with a physical time scale, parameters to set the wished level of accuracy can be introduced. This implies that the software would be in charge of choosing the time step increments. Likewise, it is possible to establish a lower and an upper limit of time step for the resolution of the system of equations. In this case, when a large number of iterations are needed, the time step is automatically taken as minimum whereas if only a few iterations are sufficient, this time step will increase up to the maximum defined [37].

All the simulations lasted until the last axle of the train had left the bridge plus 5 s, the total time depended thus on the train speed. Regarding the time step and despite the literature recommendation (see subsection 3.4.1.3) of choosing at least one eighth of the minimum period considered, the accuracy gained was not worth the time consumption. In that sense, an example for a particular train speed of 300 km/h the total simulation time is of 9.21 h, 3.98 h and 2.81 h corresponding to the time step of 0.002 s, 0.005 s and 0.008 s respectively. For that reason, the upper limit of time step for the dynamic calculations was chosen to be 0.008 s. Figure 4.20 and Figure 4.21 show the dynamic responses when different time steps were contemplated, revealing that the acceleration peaks are slightly shifted towards the right, but the maximum values are almost the same.

Figure 4.20: Accelerations for different time steps
As a final verification, the relevance of neglecting the inertial effects when simulating the train as moving loads has been studied. To this effect, a moving point mass and a moving load, both corresponding to 170 kN, were applied to MODEL 1. The results in terms of displacements (Figure 4.22) exposed an insignificant difference, supporting that the selection of modelling the train as moving loads was correct.
Chapter 5

Results and discussion

5.1 Introduction

This chapter presents the results obtained along this work as well as a brief discussion of them. It first starts by the outcome obtained from the modal analysis. It then proceeds to show the static and dynamic calculations.

5.1.1 Dynamic aspects examined

Ulla River Viaduct is an existing bridge and therefore all the design code requirements were already verified in its design stage. For that reason, this work does not contemplate checks/verifications but it does examine some of the most relevant aspects in terms of dynamics. Those are:

- Vertical accelerations. The limitation of the vertical accelerations is a requirement present in all the relevant design codes (see subsection 2.4.3) because it involves from safety (ballast stability) to comfort conditions for the users.
- Vertical deformations. In long span bridges the vertical deformations tend to be the demanding parameter. They are limited for comfort purposes (see subsection 2.4.3).
- Amplification factor. It represents the increment of the dynamic deflection with respect to the static one. The last one may be obtained for train speeds slow enough (see subsection 2.4.4). In this case it was considered 50 km/h, despite not being extremely slow the subsequent results were in the safe side.
- Envelope dynamic factor. It indicates if the calculated load effects due to high-speed are larger than those caused by conventional railway traffic (characterised by a nominal train load model). It was calculated according subsection 2.4.2.

The dynamic calculations were carried out for train speeds from 200 km/h to 420 km/h with a step of 20 km/h. Results for the entire range of train speeds were obtained by interpolation using cubic splines. There is available an already implemented function in MATLAB for splines. All the results are represented as envelopes of the maximum values obtained over time.
Finally, it is also presented the twist of the deck caused by torsion, for a given train speed. It corresponds to the angle, in radians, that the deck rotates around the longitudinal axis.

### 5.2 Modal analysis

Appendix B shows the natural frequencies and mode shapes of MODEL 1. MODELS 1 to 3 had similar values of natural frequencies, whereas the ones of MODEL 4 were lower due to the significantly low-value of torsional inertia of the deck. The first torsional mode for this last referred model corresponded to mode 10 and it was associated to a frequency of 0.77 Hz (Figure 5.1). MODEL 6 had totally different mode shapes that were similar to those expected for a continuous beam with fixed supports.

![Figure 5.1: First torsional mode for MODEL 4 corresponding to mode 10 and 0.77 Hz](image)

According to the design codes, the cut off frequencies needed to calculate using the mode superposition method is 30 Hz. MODEL 1, resulted in more than 370 oscillation modes of the deck, after all non-relevant local modes had been supressed. The influence of the oscillation modes contribution on the dynamic response can be observed in Figure 5.2, where the Fast Fourier Transform (FFT) of MODEL 1 and MODEL 4 are compared for vertical accelerations. These results were extracted in span 7 for a train speed of 200 km/h. The time step and total time considered were 0.008 s and 25 s respectively. Its associated Nyquist frequency was 62.5 Hz, however, the plot shows values up to 30 Hz because of scale reasons. These transforms were normalised with respect to its maximum values.

The maximum peak for both models was produced around 3 Hz. The explanation rests in the fact that HSLM-A1 train’s axles separation is 18m, for this particular velocity, resulted in an impact every 0.324 s (3.08 Hz).

Note that all the peaks found for MODEL 4 were slightly shifted towards the right because of its lower values of natural frequencies. In this sense, the points indicated as 1 and 2 corresponded to the first bending mode of this span, 0.76 Hz and 0.81 Hz respectively. Points named 3 and 4 corresponded to torsional modes only existing in MODEL 4.
5.3 Measurement points

Due to the modelling of the train as a singly-lane, located over the same vertical plane as the upper chord, the overall behaviour of the bridge will not be symmetrical because of the torsional excitation. As a consequence, the results obtained were expected to vary significantly depending on the measured node, not only in the longitudinal direction but also in the transversal one.

In Figure 5.3 is indicated the name used hereon when presenting the results. The numbering of the spans respected those assigned in the design drawings, and were kept the same in the entire present document. Extrapolating to the FE model, the measurement point named as deck relates to the central node of the shell elements that represents the concrete top slab. On the other hand, upper chord refers to the central node of the upper chord belonging to the vertical plane where the train load was applied.
5.4 Static displacements

5.4.1 For high-speed trains (HSLM-A1)

The static deflection $\delta_{HSLM-A1}^{static}$ caused by high-speed trains, in this case HSLM-A1, was needed for calculating the amplification factor as well as the envelope dynamic factor. It was obtained as the maximum values reached in all three mid spans, when the train was travelling at a speed low enough. The speed considered in this work was 50 km/h and despite not being extremely slow, the maximum results obtained were in the safe side. That is because the dynamics effects produced were almost negligible and the maximum displacements obtained were greater than if lower velocities would have been taken.

Figure 5.5 and Figure 5.5: Static displacement caused by train HSLM-A1 travelling at 50km/h. Measurement point: deck

Figure 5.5 and Figure 5.5 present the results for MODEL 1 and MODEL 6 respectively. The effect of the pier’s stiffness was particularly important for the maximum displacements obtained. In this context, in MODEL 1 the stiffness of the piers P-5 and P-8 (Figure 4.3) conferred certain flexibility to the displacements. Besides, when the train passed over one span, it influenced the surrounding ones. All of this led to reaching the maximum displacements in the extreme spans. On the contrary, MODEL 6 yielded to lower deflections in general, since the piers’ DOFs were restrained in all directions. The maximum values were reached in the central span due to its larger length. It can also be drawn from the aforementioned figure that the history output of the static deflections can be understood as an envelope of the critical position of the train.

![Displacement under HSLM-A1 50km/h. MODEL1](image)

**Figure 5.4:** Static displacement caused by train HSLM-A1 travelling at 50km/h. Measurement point: deck
Figure 5.5: Static displacement caused by train HSLM-A1 travelling at 50 km/h. Measurement point: deck

The values obtained for all models and spans, when measuring in both upper chord and deck, are gathered in Table 5.1 and Table 5.2 respectively.

Table 5.1: Static deflection caused by a train travelling at 50 km/h. Measurement point: deck

<table>
<thead>
<tr>
<th>Model</th>
<th>Span 6 [mm]</th>
<th>Span 7 [mm]</th>
<th>Span 8 [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>99.15</td>
<td>90.30</td>
<td>99.15</td>
</tr>
<tr>
<td>Model 2</td>
<td>86.68</td>
<td>91.82</td>
<td>86.68</td>
</tr>
<tr>
<td>Model 3</td>
<td>99.43</td>
<td>91.82</td>
<td>99.43</td>
</tr>
<tr>
<td>Model 4</td>
<td>123.07</td>
<td>113.15</td>
<td>123.07</td>
</tr>
<tr>
<td>Model 5</td>
<td>99.15</td>
<td>90.30</td>
<td>99.15</td>
</tr>
<tr>
<td>Model 6</td>
<td>58.48</td>
<td>53.24</td>
<td>58.48</td>
</tr>
</tbody>
</table>

Table 5.2: Static deflection caused by a train travelling at 50 km/h. Measurement point: upper chord

<table>
<thead>
<tr>
<th>Model</th>
<th>Span 6 [mm]</th>
<th>Span 7 [mm]</th>
<th>Span 8 [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>18.97</td>
<td>15.64</td>
<td>19.04</td>
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<td>Model 2</td>
<td>19.50</td>
<td>16.11</td>
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</tr>
<tr>
<td>Model 3</td>
<td>19.63</td>
<td>16.17</td>
<td>19.45</td>
</tr>
<tr>
<td>Model 4</td>
<td>19.50</td>
<td>16.11</td>
<td>19.59</td>
</tr>
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<td>18.97</td>
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<tr>
<td>Model 6</td>
<td>19.04</td>
<td>18.97</td>
<td>15.64</td>
</tr>
</tbody>
</table>
5.4.2 For nominal train LM-71

The static deflection $\delta_{\text{UIC-71}}^{\text{static}}$ caused by the nominal train load model was needed for calculating the envelope dynamic factor. The loading position depended on the span wished to analyse, so that the maximum deflection was reached. That is, loading the span considered and its alternates ones. The results obtained in the centre of each span, measuring in the deck and in the upper chord are resumed in Table 5.3 and Table 5.4 respectively. All these results are already multiplied by the classification coefficient of $\alpha = 1.21$.

Table 5.3: Static deflection for train model LM-71 [23] (classification coefficient $\alpha = 1.21$). Measurement point centre of deck

<table>
<thead>
<tr>
<th></th>
<th>$\delta_{\text{UIC-71}}^{\text{static}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Span 6</td>
</tr>
<tr>
<td>Model 1</td>
<td>99.15</td>
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<tr>
<td>Model 5</td>
<td>99.15</td>
</tr>
<tr>
<td>Model 6</td>
<td>58.48</td>
</tr>
</tbody>
</table>

Table 5.4: Static deflection for train model LM-71 [23] (classification coefficient $\alpha = 1.21$). Measurement point centre of upper chord

<table>
<thead>
<tr>
<th></th>
<th>$\delta_{\text{UIC-71}}^{\text{static}}$ [mm]</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Span 6</td>
</tr>
<tr>
<td>Model 1</td>
<td>98.22</td>
</tr>
<tr>
<td>Model 2</td>
<td>85.22</td>
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<td>Model 3</td>
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<td>Model 4</td>
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<tr>
<td>Model 5</td>
<td>98.22</td>
</tr>
<tr>
<td>Model 6</td>
<td>52.42</td>
</tr>
</tbody>
</table>

5.5 Accelerations and dynamic displacements

Figure 5.6 and Figure 5.7 show the accelerations calculated for the measurement points upper chord and deck, caused by the load model HSLM-A1. As it can be observed, in all the cases the design requirements for ballasted bridges ($a_{\text{max}} < a_{\text{lim}} = 0.35g$) were fulfilled.

These results show the influence of the effects produced when the train is entering to and leaving the bridge, reaching the maximum values of accelerations in span 6 and span 8. When the train was entering to the structure, it caused forced excitations whereas when it was exiting the structure was allowed to vibrate freely.
5.5. Accelerations and Dynamic Displacements

Figure 5.6 clearly reveals that when the measurement point was the centre of the deck, all the models followed the same pattern. A peak response in accelerations was observed around 360-380 km/h. This peak was shifted towards 380-400 km/h for MODEL 6 because the frequencies and mode shapes for this model were quite differentiated from the others.

MODEL 4 gave rise to larger accelerations within the speed range 320-420 km/h than all the other models, excluding MODEL 6, for midspans 6 and 8. This fact was particularly noticeably in midspan 7.

![Graphs showing accelerations in the deck caused by HSLM-A1](image_url)

Figure 5.6: Accelerations in the deck caused by HSLM-A1

Higher accelerations were reached in the upper chord where the axle of the train was passing. In this member, both torsion and bending were important. In this case, the effects of the train entering the bridge were different for each model. Conversely, same pattern has been observed in midspans 7 and 8 in all the models. Likewise, the
acceleration peaks in midspan 6 were produced for different speeds than in midspan 8, with the exception of MODEL 6. It is important to highlight that the accelerations increased as the train speed did (Figure 5.7).

The displacements are represented in Figure 5.8, for both measurement points: deck and upper chord. Their values were observed to increase together with the train speed, especially for high velocities (380-420 km/h).

An important aspect to highlight is that, considering an open cross-section had more influence on the deflections than the stiffness of the piers. In that sense, MODEL 6 resulted in the lower displacements due to the clamping of the piers whereas MODEL 4 yielded the maximum displacements due to the torsion. One may note as well, that in span 7 the magnitude order of the displacements was similar for MODEL 1, MODEL 2,
MODEL 3 and MODEL 5 in comparison to MODEL 6, implying that the two central piers gave sufficient stiffness.

The results obtained in the deck and upper chord are presented side by side for the sake of clarity. It is straightforward to see the effects due to the torsion of the deck; the displacements reached in both measurement points were similar for all the models excepting MODEL 4. The torsional rotation of the deck is noticeably for this last model. That fact implies that the geometry was distorted, the two steel lattices were working separately and on their own, increasing considerably the dynamic effects.

In overall, these results showed a similar magnitude order in terms of accelerations and displacements for all the models with exception of MODEL 4, where the maximum values were generally reached. This fact is highly meaningful because it demonstrates the importance of closing the torsional circuit of the deck.
It can be also drawn that the mass of the piers did not have a significant influence on the dynamic response. As a final remark, it is important to note that generally, MODEL 1 yielded results comprised between MODEL 3 and MODEL 4 because it corresponded to the intermediate case. In addition, these results were substantially alike to those obtained from MODEL 2.

### 5.6 Amplification factor

Figure 5.9 depicts the amplification factor for the deck and the upper chord, calculated as the ratio of the dynamic displacements $\delta_{\text{dyn}}$ to the static ones $\delta_{\text{sta}}$ induced by HSLM-A1.
5.7 Twist of the deck

As it happened with the displacements, this factor increased together with the train speed, fact particularly evident for high velocities (380-420 km/h). The amplification factor was found near the unity in all the models except MODEL 4. In this model, the results were dramatically different since the amplification factor could reach values near to 2. The explanation rests on the fact that the torsional component was crucial in the amplification of the dynamic deflections with respect to the static ones. For this same reason, when measuring in the upper chord, these values were even higher; the twist of the deck played an essential role.

5.7 Twist of the deck

Figure 5.10 presents the twist of the deck produced for a given train speed of 360 km/h in MODELS 1-4. The results are only shown in midspan 7 because when observing the dynamic displacements (Figure 5.8) the rotation of the deck was expected similar for all the spans. Likewise, the train speed was selected in accordance to the peaks in the dynamic response produced at this velocity (Figure 5.6). As expected, the results reflected an outstanding twist of the deck for MODEL 4 in comparison to MODELS 1-3.

![Figure 5.10: Torsional twist of the deck in rad](image)

5.8 Envelope dynamic factor Φ

The displacements under HSLM-A1 presented in subsection 5.4.2, obtained by a dynamic analysis served as the basis for calculating the envelope dynamic factor Φ according eq. (2.4). The subsequent results were compared to the dynamic factor Φ₂ for train speeds v ≤ 220 km/h, calculated according eq. (2.3). Considering the determinant length Lₚ = 279m resulted in a value of Φ₂ = 0.907, taking consequently Φ₂ = 1.0.

Figure 5.11 shows the values of the envelope dynamic factor Φ for HSLM-A1. For all the models excepting MODEL 4 the magnitude order was the same. As occurred with the amplification factor (section 5.6), when measuring in the upper chord the torsional contribution led to higher values in this model.

Note that for all spans and models the value of impact coefficient for the universal train was lower than the unit, which indicates that the deflections caused by LM-71 [19] were larger than the dynamic deflections caused by HSLM-A1. In other words, the calculated
load effects for HSR traffic were lower than those to conventional railway traffic, characterised by a nominal train.

This result is very useful from a practical point of view, because it implies the suitability of adopting the impact coefficient $\Phi_2$ for evaluating the dynamic effects in the design stage, in terms of deflections.

![Figure 5.11: Impact coefficient for real trains. Deck](image)

5.9 Different locations of passing train

Figure 5.12 presents the accelerations and dynamic displacements produced in MODEL 1 and MODEL 4 for different locations of the train. It allows several observations regarding three different aspects: the train was entering to or leaving the bridge, the
induced effects were symmetrical/non-symmetrical and the consideration of an open/closed cross-section of the deck.

The maximum value of accelerations was reached for Position B. Two clearly differentiated acceleration peaks were produced for train speeds ranges 320-360 km/h and 360-400 km/h in both models for this position. Such peaks values were similar for MODEL 1 and 4 in spans 6 and 7 but moderately larger for MODEL 4 in span 8. On the other hand, Position A resulted in similar values of maximum accelerations but different peaks location in the span 6. The explanation may rest in the fact that, even if the impact was produced in the upper chord in both situations, the bridge response for Position B was symmetrical. On the contrary, a slight peak was produced for Position C and train
speeds between 340-400 km/h, the values were minimum because the impact was not produced in the upper chord.

It is noteworthy that the effect of the train entering to the bridge was more accused for Position A than the train exiting it. Especially when considering open cross-sections. When looking at the results obtained in span 7, the accelerations pattern was almost identical for Position A and B. In this case, the stiffness that the piers conferred to this span played a crucial role.

The values of the displacements were greater for Position B, as was expected. The static displacements caused by two trains should be exactly twice as large as those caused by one train. However, the dynamic displacements were not exactly double.

For all the cases, MODEL 4 resulted in the largest values of displacements and accelerations. This implies that bridges with low torsional rigidity of the deck, a 2D model would neglect significant accelerations and dynamic displacements.
Chapter 6

Conclusions

6.1 Conclusions

HSR traffic requires in general structures with greater stiffness than conventional traffic ones. Currently, the vast majority of these bridges are equipped with a double railway-track, fact that emphasises the importance of considering the torsional flows caused by the eccentric loads of only one train passing.

This research focuses on studying the particular bridge of Ulla River Viaduct, where the bottom steel truss traditionally used in composite steel-concrete lattice structures for closing the cross-section torsional circuit is replaced by a concrete bottom slab. For the study, a model that is capable to combine the dynamics of the truss members and the deck was required.

The most important remarks that can be drawn from this study are presented in the following paragraphs. This section starts commenting the findings concerning different models and train locations and finally it proceeds to explain the conclusions regarding the efficiency of the solution of concrete bottom slab configuration.

6.1.1 Regarding the different models

- The study demonstrated the importance of closing the torsional circuit of the deck. Models with open cross-sections yield the highest values, in terms of acceleration and especially deflections, despite remaining far from the limiting values proposed in the relevant codes. This fact became even more evident when the amplification factor of deformations was examined, observing values twice as large as those corresponding closed cross-sections. Finally, it is remarkable that while this bridge did not present any problem associated to dynamic effects, its peculiar design could be of great interest for other case studies where such dynamic effects pose a critical issue.
- The mass of the piers mobilised did not have a significant influence on the dynamic behaviour of this particular bridge. The calculated results were almost identical whether it was contemplated or not. Conversely, totally different dynamic effects were observed when the supports were assumed as fixed. This fact highlighted the importance of modelling adequately their real stiffness to capture the real dynamic
response of the bridge. It is noteworthy that the central piers conferred sufficient stiffness to the main span; similar displacements were observed when the supports were assumed as fixed.

6.1.2 Regarding different positions of the trains

- Considerable influence of the train location on the accelerations and dynamic displacements due to the excitation of different mode shapes. This evidenced the need to perform 3D models that allow capturing the torsional modes properly. This is not easy when simplified 2D or 3D beam models are used.

6.1.3 Regarding the efficiency of bottom slab configuration

- No significant difference was found in the results when including in the model the prefabricated plates of the bottom slab, regardless of being considered continuous or not. That fact allowed concluding that the length of cast-in-place concrete was sufficient for transferring torsional flows to the piers. Nevertheless, its original deck configuration also results in more advantages non-related to its dynamic behaviour. Among others, the extension of the bottom slab along the entire span facilitates maintenance and inspection tasks as well as allows closing the formal section from the bottom view, being more aesthetical. With all of that in mind, it may be remarked that the solution of this bridge not only resulted very clever, but also more economical than the traditional solution of placing a bottom steel truss.

6.2 Further research

The proposed possible future lines of work are summarized as follows:

- Provide more validity of the results by modelling the entire bridge and contemplating all 10 HSLM
- Research if there would be a pronounced variation of the results when a complex bridge-vehicle interaction model is performed.
- Quantify the minimum torsional rigidity (with respect to the longitudinal one), for which the torsional effects can be neglected and a 2D model could be considered.
- Examine a greater number of HSR bridges with similar characteristics in order to contrast and generalize the conclusions.
- Broaden the study to viaducts with completely different structural forms. Determine if the conclusions obtained in this thesis are applicable.
- Investigate possible simplified models capable to capture the coupled torsional-bending response of the studied bridge taking into account the stiffness that the piers confer.
Chapter 7

Bibliography


Appendix A

FE Models

A.1 Materials

A.1.1 Concrete

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A.2 Cross sections

**Figure A.1:** Box cross-section

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### A.2. Cross Sections

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A.3 FE Models

MODEL 1

- Discontinuity in the prefabricated plates of the bottom slab (middle of spans)
- Cast-in-place bottom slab near the supports

MODEL 2

- Cast-in-place bottom slab near the supports

MODEL 3

- Continuity in the prefabricated plates of the bottom slab (middle of spans)
- Cast-in-place bottom slab near the supports
A.3. FE Models

MODEL 4

- Open cross-section of the deck. No bottom slab.

MODEL 5

- Original bottom slab configuration (MODEL 1).
- Null density in the concrete material of the piers.

MODEL 6

- Original bottom slab configuration (MODEL 1).
- Fixed supports.
Appendix B

Mode shapes

B.1 Oscillation modes MODEL 1

Figure B. 1: Mode 1, $f = 0.21$ Hz

Figure B. 2: Mode 2, $f = 0.22$ Hz

Figure B. 3: Mode 3, $f = 0.38$ Hz

Figure B. 4: Mode 4, $f = 0.42$ Hz
Figure B. 5: Mode 5. $f = 0.45$ Hz

Figure B. 6: Mode 6. $f = 0.5$ Hz

Figure B. 7: Mode 7. $f = 0.67$ Hz

Figure B. 8: Mode 8. $f = 0.81$ Hz

Figure B. 9: Mode 9. $f = 0.86$ Hz

Figure B. 10: Mode 10. $f = 0.87$ Hz

Figure B. 11: Mode 11. $f = 0.90$ Hz

Figure B. 12: Mode 12. $f = 0.96$ Hz
Figure B. 13: Mode 13. $f = 1.18$ Hz

Figure B. 14: Mode 14. $f = 1.28$ Hz

Figure B. 15: Mode 15. $f = 1.41$ Hz

Figure B. 16: Mode 16. $f = 1.47$ Hz

Figure B. 17: Mode 17. $f = 1.58$ Hz

Figure B. 18: Mode 18. $f = 1.60$ Hz

Figure B. 19: Mode 19. $f = 1.62$ Hz

Figure B. 20: Mode 20. $f = 1.76$ Hz
Figure B. 21: Mode 21, $f = 1.77$ Hz

Figure B. 22: Mode 22, $f = 1.89$ Hz

Figure B. 23: Mode 23, $f = 1.93$ Hz

Figure B. 24: Mode 24, $f = 1.99$ Hz

Figure B. 25: Mode 25, $f = 1.99$ Hz

Figure B. 26: Mode 26, $f = 2.18$ Hz

Figure B. 27: Mode 27, $f = 2.53$ Hz

Figure B. 28: Mode 28, $f = 2.62$ Hz
### B2 Comparison eigenfrequencies

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Appendix C

Static loading test

C.1 Vehicle definition

Figure C.1: Locomotive 333.3 composed of 6 axles of 20 t each and Hopper RENFE-80T composed of 4 axles of 20 t each

C.2 Loading positions

Figure C.2: Loading position 11
Figure C.3: Loading position 12

Figure C.4: Loading position 13

Figure C.4: Loading position 14