UPDATING AND VALIDATION OF THE FINITE ELEMENT MODEL OF A RAILWAY VIADUCT BASED ON DYNAMIC TESTS

Joel Malveiro, Diogo Ribeiro, Rui Calçada, Raimundo Delgado

ABSTRACT

This paper describes the calibration and experimental validation of the numerical model of a railway viaduct with precast deck located in the Portuguese railway network. The three-dimensional numerical model used to evaluate the dynamic behavior of the structure for the passage of Alfa Pendular trains is presented. The calibration of the numerical model is performed by an optimization process based on the application of a genetic algorithm. The dynamic test under railway traffic allows obtaining the dynamic response in terms of displacements and accelerations at different deck locations for the passage of Alfa Pendular train. This dynamic test shows the existence of a non-linear behaviour of the viaduct’s supports. The calibrated model was validated based on a comparison between numerical and experimental responses and a very good correlation was obtained.

Keywords: railway viaduct, dynamic test, railway traffic, model updating, validation

1. INTRODUCTION

Railway bridges and viaducts are structures subjected to high intensity moving loads, where the dynamic effects can reach significant values. In the last decades, these effects have been of great importance due to the increase of the circulation speed, not only in conventional lines but also in high speed lines. In high speed railway lines the dynamic effects tend to increase considerably as a result of the resonance phenomena originated by the periodic loading associated with the passage of regularly spaced axles groups of the trains [1].

The dynamic analyses are usually based on finite element numerical models of the structure that involves assumptions and simplifications that may cause errors. These errors are basically related to the inaccuracy in the FE model discretisation, uncertainties in geometry and boundary conditions and variation in the material properties. Due to these aspects, the importance given to the automatic calibration process of the numerical models has been increased, especially taking into account the

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modal parameters of the structure [2, 3]. However, the accuracy of the finite element model in reproduce the dynamic behaviour of the structure strongly depends on the experimental validation of the numerical results that is usually performed by means of dynamic tests.

This paper describes the calibration and experimental validation of the numerical model of a railway viaduct with precast deck. The calibration of the three-dimensional numerical model is performed by an optimization process based on the application of a genetic algorithm. A dynamic test under railway traffic enables to obtain records of displacements and accelerations of the lower slab of the deck during the passage of Alfa Pendular train at different speeds. Finally, the experimental validation of the calibrated numerical model based on modal parameters is performed by the comparison between the numerical and the experimental responses.

2. ALVERCA RAILWAY VIADUCT

Alverca railway viaduct is a flyover structure on the Portuguese railways’ Northern Line that establishes the rail connection between Lisbon and Porto. Its construction allowed to separate the rail traffic flowing in the downstream and upstream directions and also to maintain the maximum speed of trains at 200 km/h. Figure 1 shows a perspective view of the current zone of the viaduct (Figure 1 a)) and a cross section of the deck (Figure 1 b)).

![Figure 1 Alverca railway viaduct: a) perspective view; b) cross section](image)

The viaduct, that supports one single railway track, has a total length of 1091 m divided into 47 simply supported spans: 9×16.5 m + 9×17.5 + 29×21.0 m. Each span is composed by a prefabricated and prestressed U shape beam connected by an upper slab cast in situ, forming a single-cell box-girder deck. The deck is directly supported in the piers by elastomeric reinforced bearings. These bearings are fixed in one extremity and longitudinally guided in the other extremity. The track is continuous between successive spans and is composed by 30 cm of ballast, monoblock sleepers and UIC60 rails.

3. NUMERICAL MODELLING

The dynamic analysis of the Alverca railway viaduct was carried out using a three-dimensional numerical model developed in ANSYS software. The analysis focused on the three spans adjacent to the North abutment: one 16.5 m span and two 21 m spans. An extra extension of the track, with a length of 6 m, apart from the abutment, was modelled in order to simulate the effect of the track over the adjacent embankment. Figure 2 shows an overview of the numerical model with a detail of the track components.
The prefabricated beam, the upper slab and the ballast retaining walls were modeled by shell finite elements. The sleepers, the rail pads and the ballast layer were modeled by volume finite elements. Each support was regarded as a single point and modeled by a spring element. The stiffness of the supports was calculated taking into account the confinement effect of the neoprene layers provided by various metallic plates. The stiffness of the support in the vertical direction \( k_v \), which can be calculated considering a system of series-connected springs, each one simulating an individual neoprene layer, was initially taken equal to 5200 MN/m. The longitudinal stiffness of the support \( k_l \), which considers the shear modulus of the neoprene, was also taken into account and assumed equal 3.6 MN/m.

The modulus of elasticity of the precast beam \( E_c \) and the upper slab of the deck (different for the three spans – \( E_{c1} \), \( E_{c2} \) and \( E_{c3} \)) were considered equal to 40.9 GPa and 35.4, respectively, corresponding to the average value of this parameter at 28 days with the necessary adjustment to the age of the concrete at the experimental tests. The density of the concrete \( \rho_c \) was assumed equal to 2469.8 kg/m\(^3\). The modulus of elasticity of the ballast layer \( E_{bal} \) and its density \( \rho_{bal} \) were considered equal to 145 MPa and 2039 kg/m\(^3\), respectively.

Figure 3 presents global vibration modes of the deck \( G \) and local vibration modes of the upper slab of the deck \( L \) obtained from a modal analysis of the initial numerical model of the viaduct.

- **Mode 1G**: \( f = 6.16 \) Hz
- **Mode 1L**: \( f = 27.54 \) Hz
- **Mode 2G**: \( f = 6.07 \) Hz
- **Mode 2L**: \( f = 27.42 \) Hz
- **Mode 3G**: \( f = 9.13 \) Hz
- **Mode 3L**: \( f = 48.22 \) Hz
4. MODEL UPDATING

The initial numerical model of the viaduct was calibrated taking into account the modal parameters experimentally identified through the ambient vibration test, namely its natural frequencies and modal configurations for global and local vibration modes. More details about the ambient vibration test can be found in Malveiro et al. [4]. The calibration process of the numerical model involved the definition of an objective function and the application of an optimization technique based on a genetic algorithm. The iterative calibration process involves the use of three software packages: ANSYS, MATLAB and OptisLang [2]. The objective function includes two terms, one related to the natural frequencies and other related to the MAC values, comprising the global and local vibration modes:

\[
 f = a \sum_{i=1}^{nmodes} \left[ \frac{f_i^{\text{exp}} - f_i^{\text{num}}}{f_i^{\text{exp}}} \right] + b \sum_{i=1}^{nmodes} |MAC(\phi_i^{\text{exp}}, \phi_i^{\text{num}}) - 1| \tag{1}
\]

where \( f_i^{\text{exp}} \) and \( f_i^{\text{num}} \) are the experimental and numerical frequencies for mode \( i \), \( \phi_i^{\text{exp}} \) and \( \phi_i^{\text{num}} \) are the vectors containing the experimental and numerical modal information regarding mode \( i \), \( a \) and \( b \) are weighing factors of the objective function’s terms, considered equal to 1.0 in this case, and \( nmodes \) is the total number of modes, equal to 6 (3 global and 3 local vibration modes).

Table 1 shows the parameter’s values that originated the lowest residual of the objective function, which should be considered in the numerical model in order to minimize the differences between numerical and experimental responses.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Initial value</th>
<th>Optimal value</th>
<th>Unity</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_{c1} )</td>
<td>35.40</td>
<td>30.07</td>
<td>GPa</td>
</tr>
<tr>
<td>( E_{c2} )</td>
<td>35.40</td>
<td>33.35</td>
<td>GPa</td>
</tr>
<tr>
<td>( E_{c3} )</td>
<td>35.40</td>
<td>35.10</td>
<td>GPa</td>
</tr>
<tr>
<td>( E_c )</td>
<td>40.94</td>
<td>48.08</td>
<td>GPa</td>
</tr>
<tr>
<td>( \rho_c )</td>
<td>2469.8</td>
<td>2590.4</td>
<td>Kg/m³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Initial value</th>
<th>Optimal value</th>
<th>Unity</th>
</tr>
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<tbody>
<tr>
<td>( E_{\text{bal}} )</td>
<td>145.00</td>
<td>142.70</td>
<td>MPa</td>
</tr>
<tr>
<td>( \rho_{\text{bal}} )</td>
<td>2039.0</td>
<td>1995.9</td>
<td>Kg/m³</td>
</tr>
<tr>
<td>( K_{h1} )</td>
<td>3.6</td>
<td>188</td>
<td>MN/m</td>
</tr>
<tr>
<td>( K_{h2} )</td>
<td>3.6</td>
<td>238</td>
<td>MN/m</td>
</tr>
<tr>
<td>( K_{h3} )</td>
<td>3.6</td>
<td>252</td>
<td>MN/m</td>
</tr>
</tbody>
</table>
Figure 4 shows the comparison between experimental and numerical frequencies and MAC values, before and after the calibration process.

![Comparison of frequencies and MAC values before and after calibration](image)

Figure 4 Modal parameter’s values before and after calibration: a) frequencies; b) MAC

After the calibration, the average error of frequencies decreased from 11.6 % to 6.5 % for global vibration modes and from 7.3 % to 3.1 % for local vibration modes. The average value of the MAC parameter slightly increased from 0.899 to 0.902 in case of global vibration modes and increased from 0.657 to 0.760 in case of local vibration modes.

5. DYNAMIC TEST UNDER RAILWAY TRAFFIC

The validation of the numerical model was based on the dynamic test under railway traffic. This experimental test allowed the measurement of displacements and accelerations on the lower slab of the deck, in the mid-span section of the span 2 and also the longitudinal displacements of the mobile supports of the span 2. Figure 5 shows some details related to the location of these sensors in the deck.

![Dynamic test under railway traffic](image)

Figure 5 Dynamic test under railway traffic: a) accelerometer; b) LVDT on the lower slab of the deck; c) LVDT on the structural joint between spans 1 and 2, in the longitudinal direction

The dynamic response of the deck was measured for the passage of Alfa Pendular train at 157 km/h and 185 km/h. This conventional train has a total length of approximately 150 m and the axle loads vary between 128.8 kN and 138.4 kN. Figure 6 shows the load scheme of the Alfa Pendular train.
Figure 6 Load scheme of the Alfa Pendular train

Figure 7 shows the records of accelerations, in the time and frequency domain, in the mid-span section of the span 2, experimentally obtained for the passage of Alfa Pendular train at speeds \( v \) of 157 km/h and 185 km/h. The dynamic response is dominated by frequencies related to the train action, as can be seen by the peak of 6.71 Hz, corresponding to the passage of bogies of different carriages \( d = 6.5 \text{ m} \) for a speed of 157 km/h \( f = v/d = 6.71 \text{ Hz} \). This frequency is very close to the frequency of the experimentally identified mode 1G, equal to 6.76 Hz, amplifying the dynamic response and assuming a preponderant contribution towards the remaining frequencies. The same effect is not visible at the speed of 185 km/h due to the difference between the frequencies’ values.

Figure 7 Accelerations records for the passage of Alfa Pendular train: a) time and b) frequency domain
6. VALIDATION OF THE NUMERICAL MODEL

The validation of the numerical model was performed according to the results of the dynamic test under railway traffic. The numerical analyses were performed using the modal superposition method, considering the Alfa Pendular train as a set of moving loads, vibration modes with frequencies up to 30 Hz and an integration time increment equal to 0.002 s. The modal damping coefficients were considered equal to those obtained in the ambient vibration test [4].

As described before, the initial numerical model was calibrated based on modal parameters identified in the ambient vibration test, known as Model 1. However, a new model, known as Model 2 was considered, which was obtained by reducing the longitudinal stiffness of the supports from Model 1. This model was considered in order to obtain a mobility effect of the supports closer to the one shown in the dynamic test under railway traffic. Figure 8 shows the comparison between the experimental and numerical responses based on numerical models 1 and 2, in terms of displacements and accelerations in the mid-span section of the lower slab for the passage of Alfa Pendular train at 185 km/h and also the longitudinal displacements of the mobile support for a passage at 157 km/h.

![Figure 8](image)

**Figure 8** Experimental and numerical responses from models 1 and 2: a) displacement of the deck; b) acceleration of the deck; c) longitudinal displacement of the mobile support of the span 2
In order to minimize the differences between numerical and experimental responses, a value of 4.40 MN/m was considered for the support’s longitudinal stiffness. This value is slightly higher than the one adopted in the initial numerical model, which was equal to 3.6 MN/m. Taking this aspect into account, it is possible to note a better agreement between experimental and numerical results with the numerical model 2. This result seems to show a non-linear behaviour of the supports that act as fixed supports under ambient actions and act as mobile supports under railway traffic.

7. CONCLUSIONS

This paper describes the calibration and validation of the numerical model of a railway viaduct with precast deck, composed by several simply supported spans.

In the calibration process, based on the application of a genetic algorithm, the optimal parameters of the numerical model are obtained. The average error between numerical and experimental frequencies decreased considerably after the calibration, while the average value of the MAC parameter increased, especially considering the local vibration modes.

Dynamic responses regarding to vertical displacements and accelerations of the deck and longitudinal displacements of the supports for two train speeds (157 km/h and 185 km/h) of Alfa Pendular train are obtained, based on a dynamic test under railway traffic. It is possible to note that the response is clearly dominated by frequencies associated to the action. For a speed of 157 km/h it is clear a high amplification on the frequency of mode 1G due to the proximity of one of the train action frequencies.

The experimental validation of the calibrated model based on modal parameters is performed by the comparison between the numerical and the experimental responses measured in a dynamic test under railway traffic. The modified numerical model, with the reduction of the longitudinal stiffness of the supports showed a better agreement between numerical and experimental results, which can be associated to its non-linear behaviour that operate like rigid supports under environmental actions, while operate as mobile supports under the action of railway traffic.

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REFERENCES


