

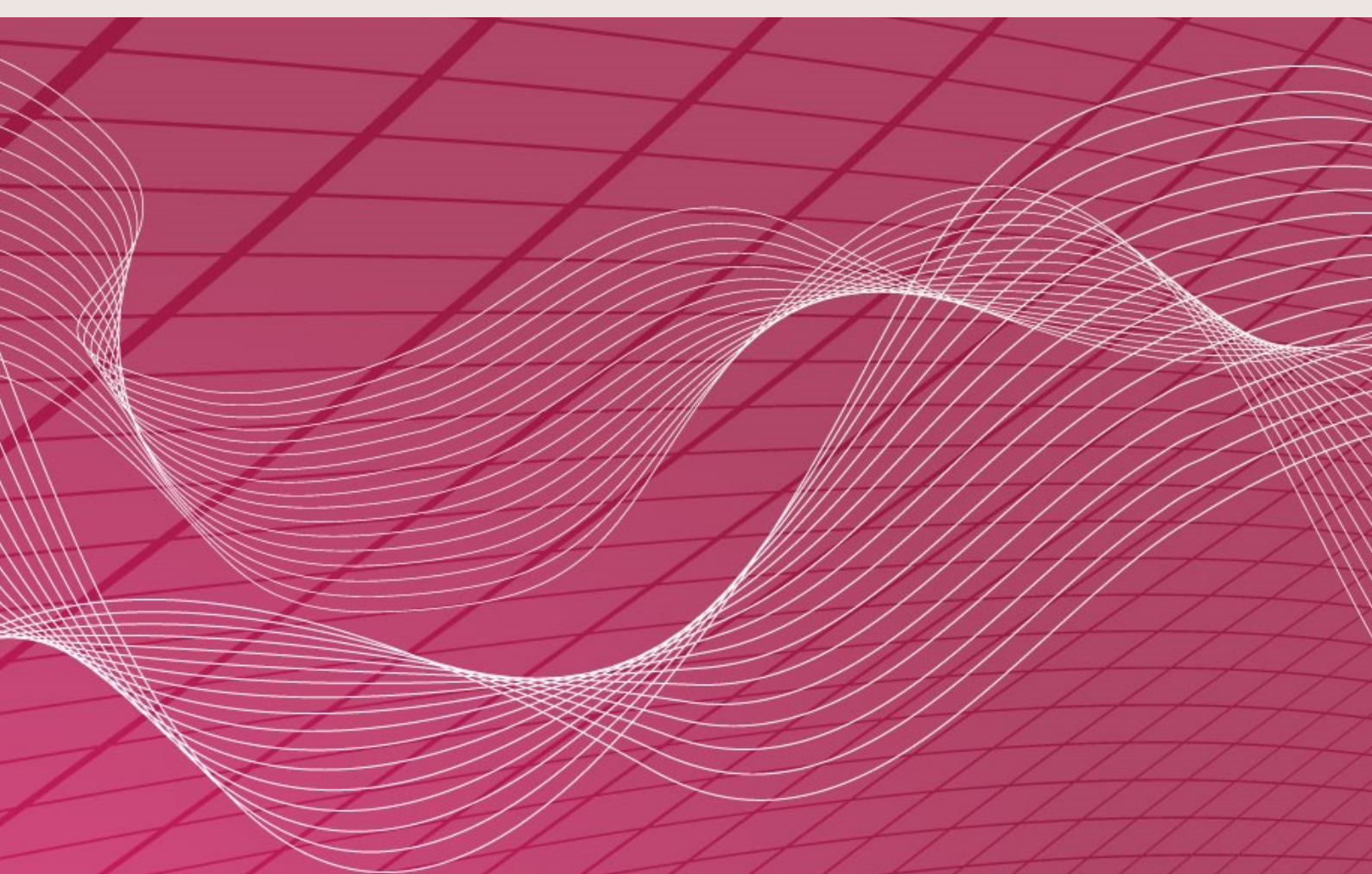
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M. Papadrakakis, M. Fragiadakis, C. Papadimitriou (Eds.)



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VERTICAL COUPLING EFFECT OF THE BALLASTED TRACK ON THE DYNAMIC BEHAVIOR OF MULTITRACK RAILWAY BRIDGES COMPOSED BY ADJACENT DECKS

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Keywords: railway bridge dynamics, ballasted track, vertical acceleration, moving-loads

Abstract. *In the present contribution the effect of the continuity of the ballasted track on the dynamic response of simply-supported railway bridges composed by structurally independent adjacent slabs at each span is addressed. Previous research works and experimental campaigns performed on simply-supported structures have shown that the presence of the ballast layer and other track components (rails, sleepers) may induce a dynamic coupling effect between the bridge spans or adjacent decks. In this study the influence of this effect on the train-induced vibrations is assessed. With this purpose a preliminary three-dimensional finite element (FE) model that includes the track components as a set of discrete mass, stiffness and damping elements has been implemented to numerically evaluate the influence of the continuity of the track components on the prediction of the bridge acceleration response under the passage of railway vehicles. The numerical model is calibrated with the load test results performed on a simply-supported railway bridge composed by several adjacent slabs. Finally the measured structural response of the bridge under train induced vibrations is compared with the numerical predictions. Preliminary conclusions regarding the importance of considering the continuity of the track components for the prediction and the assessment of the Serviceability Limit State of vertical acceleration in ballasted simply-supported railway bridges are presented.*

1 INTRODUCTION

The evaluation of the dynamic performance of railway bridges under the passage of the modern railway transportation systems is an issue of main concern to guarantee structural integrity and traveling comfort. In particular, for short-to-medium simply-supported (SS) bridges the Serviceability Limit State of vertical acceleration, limited to 3.5 m/s^2 for ballasted tracks according to Eurocode (EC) [1], is one of the most demanding requirements for their design or upgrading to new traffic requirements, since they are especially susceptible of experiencing considerably high vibration levels at circulating speeds above 200 km/h due to their low associated structural damping and mass [2, 3]. Therefore, the development of reasonably accurate numerical models becomes crucial in practical applications.

However, the dynamic response of these structures can be difficult to predict during the design or upgrading stages, since there is a main source of uncertainty in what concerns the modeling of the super-structure components, formed by ballast, sleepers and rails in ballasted tracks. In many publications and practical applications the effect of the track is disregarded, and the coupling effect exerted by its continuity along SS spans of the same bridge, and also by the shared ballast layer between structurally independent decks that form one span in certain bridges, can be significant. Also the track distributes the axle loads and may exert a restraining effect on the boundary conditions at the end supports of the bridge. Previous studies have already suggested the importance of the coupling and restraining effect induced by the ballasted track [4, 5, 6, 7], but this effect is not yet well known and requires further research.

In this regard a preliminary study of the influence of the track components and its continuity on the bridge acceleration response under the passage of railway convoys, was performed by the authors in a previous work [8]. In this study, the bridge spans were represented by Bernoulli-Euler (B-E) beams resting on elastic supports, and a three layer discrete track model consisting of a set of springs and dampers representing the track components (sleepers, railpads, etc.) as the one proposed by Zhai was used [9]. With this preliminar model a sensitivity analysis over the stiffness and damping coefficients of the track parameters was performed, showing that the variation of track damping constants has a negligible effect on the maximum vertical acceleration response induced by train passage. This preliminar observation is also in accordance with previous works which consider that the dynamic response of the track structure occurs only for frequencies considerably higher than those of the vehicle and the bridge deck, of the order of 100 Hz or above [10]. For this reason, the use of Mode Superposition analysis for solving the dynamic equilibrium equations of the bridge is a reasonable strategy to reduce computational cost in the prediction of train-induced vibrations. Previous studies apply this technique, therefore neglecting the damping introduced by the track elements at discrete positions but including global damping ratios instead. [5, 11, 12]

In this contribution, the dynamic behavior of multi-span SS bridges formed by structurally independent single-track decks at each span is investigated, in order to evaluate the effect of the continuity of the ballasted track on the vertical acceleration response under train-induced vibrations. As the object of study, a bridge belonging to the Spanish railway network is used, which is described in Section 2. The results of an experimental campaign performed by the authors in this bridge are used to calibrate the three-dimensional finite element model described in Section 3. In Section 4 the results of the comparison between numerical predictions and experimental measurements are shown, in terms of both natural frequencies and mode shapes and railway-induced vibrations. Finally, some conclusions are extracted regarding the effect of the track superstructure on the dynamic response of the bridge.

2 CASE STUDY: OLD GUADIANA RIVER BRIDGE

2.1 Bridge description

In a view to evaluate the vertical coupling effect of the ballasted track on the bridge dynamic response, an existing double-track bridge composed by two simply-supported (SS) identical bays (span length $L=11.9$ m) with separated single-track decks is used as the object of study. This bridge belongs to the conventional railway line Madrid-Alcázar de San Juan-Jaén, in Spain (Fig.1).



Figure 1: Old Guadiana river bridge.

As can be seen in Fig.1 the two structurally independent decks that form each span are composed by a reinforced concrete slab resting on five pre-stressed concrete rectangular girders. Each deck accommodates a ballasted track with Iberian gauge UIC60 rails and mono-block concrete sleepers at regular distances of 0.60 m. The longitudinal girders of the decks rest on the two abutments and on a central support through laminated rubber bearings. A cross-section of the bridge is shown in Fig.2.

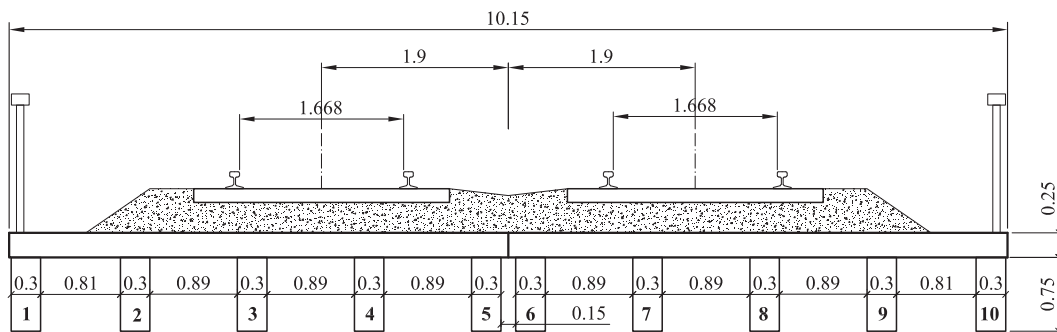


Figure 2: Cross section of the bridge.

2.2 Experimental campaign

In May 2019 the authors performed an experimental campaign on the bridge which included a dynamic characterisation of the soil and of the structure. As per the acquisition equipment, a portable acquisition system LAN-XI of Brüel & Kjaer was used. The Analog/Digital conversion (A/D) was carried out at a high sampling frequency that avoided aliasing effects using a low-pass filter with a constant cut-off frequency. The sampling frequency was $f_s = 4096$ Hz. As

regards the soil characterisation, which was carried out by the by Spectral Analysis of Surface Waves test, a rather stiff soil was identified with a shear wave velocity higher than 250 m/s in the upper soil layer.

For the characterisation of the structure the acceleration response was measured under ambient and train-induced vibrations at 18 points of the lower flange of the pre stressed concrete girders (points 1-18 in Fig.3). Endevco model 86 piezoelectric accelerometers with a nominal sensitivity of 10 V/g and a low frequency limit of 0.1 Hz were installed in the aforementioned locations.

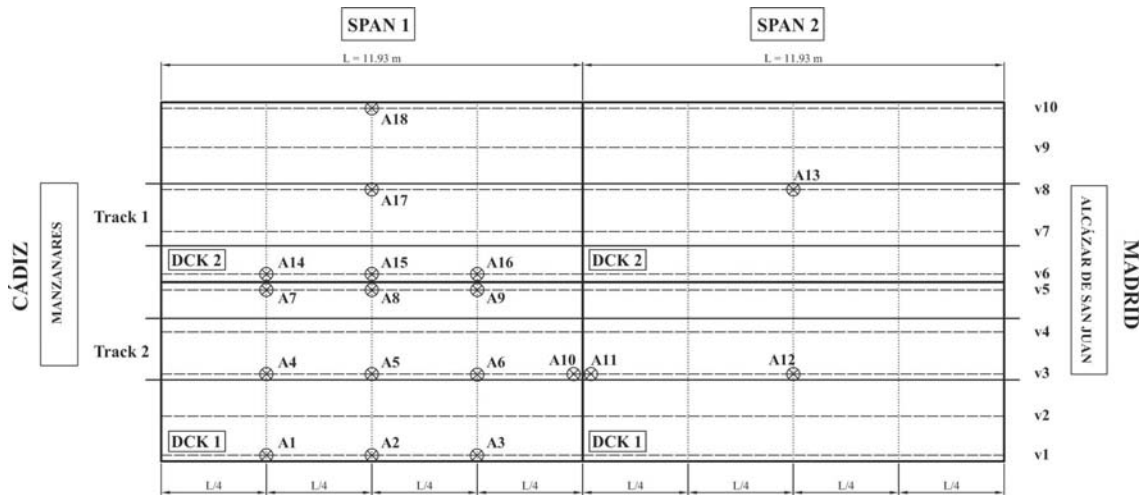


Figure 3: Location of the sensors at Old Guadiana river Bridge.

The ambient vibration data recorded during 3600 s was used for the identification of the modal parameters of the bridge by state-space models, using MACEC software [13]. Table 2 (top) shows the damping ratios and natural frequencies of the first 6 identified modes (f_{exp}) and their mode shapes are shown in Fig.4 in solid black trace. As can be seen the lowest one in frequency order corresponds to the first longitudinal bending of each bay where the two adjacent decks vibrate in phase. The second mode corresponds to the typical first torsion mode of a continuous deck in each span, where the two independent slabs that share the ballast layer deform accordingly. In the third mode, the two adjacent decks in a span deform under independent torsion but out of phase such that they conform a typical first transverse bending mode of a continuous slab. In the aforementioned modes, the deck coupling caused by the ballast layer is very evident. In the higher frequency modes the decks deform under combinations of torsion, longitudinal and transverse bending.

3 NUMERICAL ANALYSIS

3.1 Finite element model

The finite element (FE) model shown in Fig.5 has been implemented in the commercial code ANSYS to obtain preliminary conclusions about the vertical coupling effect of the ballast layer on the dynamic behaviour of Old Guadiana bridge. The main features of the model are:

- The reinforced concrete slabs that form each span are simulated by means of isotropic thin plates discretised with shell elements with 6 degrees of freedom (dof) per node.

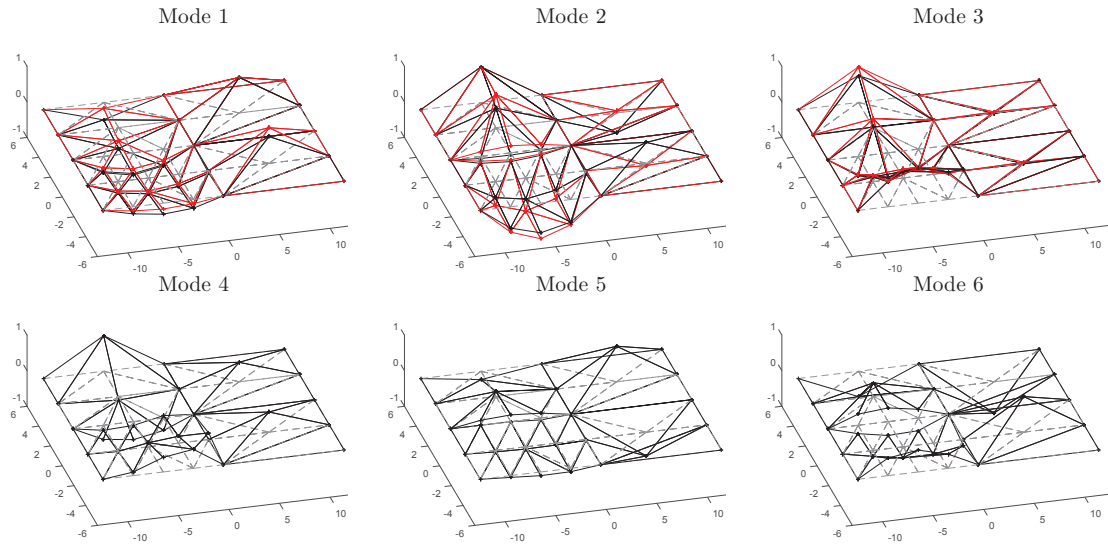


Figure 4: Experimental (black solid line) vs. numerical with coupled spans and decks (red solid line) first six identified mode shapes. Undeformed shape (dashed grey line).

The element size is chosen to adequately reproduce the wavelengths of the modes with frequencies up to 30 Hz as per EC [1].

- For each slab, different mass density elements are defined in order to concentrate the weight of the handrails, sidewalks and concrete slab selfweight in its corresponding position over the platform area.
- The vertical interaction effect induced by the ballast layer on the two separated decks at each span is simulated in a first approach by discrete longitudinal springs with vertical constant stiffness K_{wL} , that are distributed along the free longitudinal border of the adjacent slabs.
- A three layer two-dimensional (2D) discrete track model as the one proposed by Zhai [9] is implemented to include the distributive effect of the train axle loads exerted by each rail, as well as the vertical coupling effect among the bridge spans. In this model, the rails are simulated as Bernoulli-Euler beam elements with 6 dof per node, and the damping and stiffness of rail pads, ballast and subgrade are included at the sleepers positions as seen in Fig.5.
- The longitudinal girders are included in the model as beam elements with 6 dof per node. These nodes are connected to those of the upper plate right above them by means of rigid kinematic constraints. The distance between the plate and the beams nodes equals the real separation between the slab neutral plane and the center of mass of the girders.
- The laminated rubber bearings of the bridge are introduced in discrete positions by means of longitudinal springs with vertical constant stiffness $K_{v,dyn}$.
- A point load model is adopted for the railway excitation, therefore neglecting vehicle-structure interaction effects according to Eurocode 1 [14]. In this regard, some previous works also reveal that the incorporation of interaction effects in the design of new high-speed simply supported bridges or in the assessment of existing ones is not well justified,

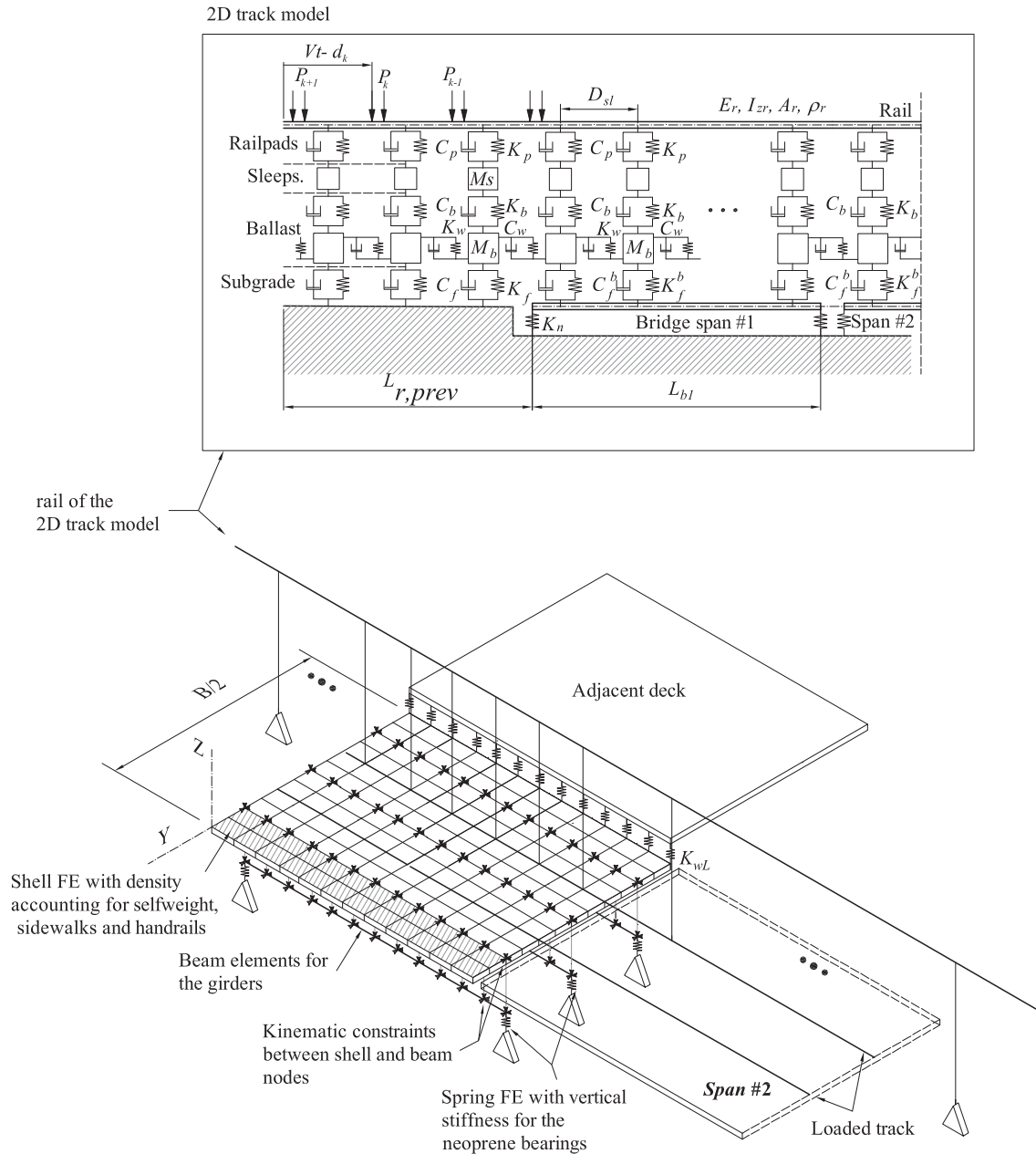


Figure 5: FE model.

since they can be very low due to the marked variability of the vehicle suspension characteristics, the mass and frequency ratios between the bridge and the vehicles suspension systems parameters and coaches masses [15, 16, 17, 18, 19]. Therefore, the use of a point load model in this study seems a reasonable approach to provide essential information concerning the track coupling effect.

- The dynamic equations of motion are transformed into modal space and numerically integrated by the Newmark-Linear Acceleration algorithm. Therefore, in a first approach the additional damping introduced by the track components is neglected for the calculation of the deck acceleration response under train induced vibrations and the modal damping ratios identified in the experimental campaign are used instead. This is in accordance

with the conclusions derived from a sensitivity analysis performed by the authors in a previous work, showing a negligible influence of this parameter on the vertical acceleration response of the deck [8]. The time-step is defined as 1/25 times the smallest period used in the analysis (mode contributions up to 30 Hz as per European Standards [1]). This value avoids period elongation errors and enables to capture properly the oscillations of the modal loading functions and the peak responses obtained by the summation of all modal contributions.

3.2 Description of the analysis procedure

The effect of the continuity of the ballasted track on the dynamic behaviour of railway bridges formed by SS decks is evaluated in this work by comparison with the test results of Old Gadiana bridge.

In a first step, the numerical model described in section 3.1 is calibrated in order to reproduce the static tests performed on the structure right before its opening and also the dynamic results obtained in the experimental campaign performed by the authors in May 2019. The calibration of the model is made considering three different approaches, which are:

- (i) Single-deck single-span (SDSS): the 2D discrete track model is not included. Therefore, the four structurally independent decks that form Gadiana Old bridge are not coupled, and the axle railway loads move along the discretised slab following the loaded track position. In order to consider the weight of the track components (rails, ballast, sleepers), the density of the FE located at the position of the track platform is modified accordingly. Since the track is not included in the model, when a load enters or exits the bridge a transient phenomenon takes place which leads to unrealistic high-frequency modal contributions of the plate. This numerical problem is solved in the model including the distributive effect of rails, sleepers and ballast during the application process of the wheel loads when they are close to the abutments. To this end, the value of each axle load is modulated throughout a load-print distributive function based on the Zimmerman-Timoshenko solution for an infinite beam on Winkler foundation, as described in [20].
- (ii) Single-deck double-span (SDDS): in this approach the 2D track model is included but the effect of the transverse continuity of the ballast is neglected by setting the vertical constant stiffness between adjacent decks, K_{wL} , to zero.
- (iii) Double-deck double-span (DDDS): the model shown in Fig.5 is fully implemented to simulate the dynamic behaviour of old Gadiana bridge.

For the calibration of the FE model assuming the previously mentioned approaches, a set of nominal or reference values for the track parameters K_p , C_p , K_b , C_b , K_f , C_f , K_{bf} , C_{bf} and M_s , is defined on the basis of the literature review. These parameters remain constant while the main track parameters affecting deck coupling, which are K_w and K_{wL} , and other bridge parameters, such as the deck and girders elastic modulus, vertical stiffness of the rubber bearings $K_{v,dyn}$, thickness of the ballast layer (which affects directly to the ballast mass) are varied in a realistic range until a satisfactory correspondence between numerical and experimental results in terms of natural frequencies and mode shapes is achieved. Table 1 shows the calibrated properties of the three different approaches considered in the numerical model of Gadiana bridge. As regards the ballast shear stiffness between adjacent decks, K_{wL} , it is provided per unit of span length. As per the subgrade stiffness K_f^b and damping coefficients inside the bridge C_f^b , the

values $100K_f$ and 0 Ns/m have been assigned as it is assumed that the ballast layer rests directly on the concrete slab inside the bridge. All these values will be the ones used in Section 4.

		SDSS	SDDS	DDDS
Slabs	ρ [kg/m ³]		2500	
	E_{slab} [MPa]		25200	
Girders	I_h [m ⁴]		0.011	
	J [m ⁴]		0.00505	
	ρ [kg/m ³]		2500	
	E_{girder} [MPa]		28800	
	K_p [N/m]	–	1E8	1E8
Track parameters	C_p [Ns/m]	–	7.5E4	7.5E4
	M_s [kg]		300	
	K_b [N/m]	–	1.933E8	1.933E8
	C_b [Ns/m]	–	5.88E4	5.88E4
	M_b [kg]		672.2	
	K_f [N/m]	–	7.9387E7	7.9387E7
	C_f [Ns/m]	–	7.5E4	7.5E4
	K_w [N/m]	–	7.84E5	7.84E5
	C_w [N/m]	–	8E4	8E4
	k_{wL} [N/m ²]	–	–	4.07E6
Supports	$K_{v,dyn}$ [N/m]		1.95E8	

Table 1: Calibrated properties of the FE model.

Secondly, a sensitivity analysis is performed to evaluate the influence of the ballast shear stiffness parameters for longitudinal and transverse coupling between the bridge decks (K_w and K_{wL}), on the natural frequencies and mode shapes of the structure. And finally, the vertical acceleration response of the deck predicted with the calibrated numerical model assuming the three different approaches is compared with the experimental measurements at several sensor locations under the passage of real trains at resonant and non resonant speeds.

4 RESULTS

4.1 Ballasted track coupling effect on the natural frequencies and mode shapes

Table 2 (bottom) shows the numerical natural frequencies and the Modal Assurance Criterion (MAC) values [21] of the paired mode shapes obtained with the calibrated FE models. In this table, only the numerical modes that exhibit MAC values higher than 0.7 with a frequency difference with respect to the experimental value below 10% are shown. For a better comparison between the numerical models, the MAC values provided in the table have been calculated with the measurements of the same number of sensors (sensors A1 to A10 of Fig.3).

As can be seen the total number of paired modes that meet this criterion is scarce. However, it should be mentioned that in the remaining identified modes above the third one not all of them could be fully described due to the limited number and spatial distribution of sensors installed. Therefore, a satisfactory correspondence of these mode shapes with the experimental measurements was not expected.

Among the three different modeling approaches used in this work, the better correspondence with the experimental values is achieved with the model that considers the coupling effect be-

Mode		1	2	3	4	5	6
f_{exp} [Hz]		9.82	11.05	12.86	16.53	17.93	21.05
ζ_{exp} [Hz]		2.3	0.9	1.0	0.3	0.1	1.0
Numerical approach							
SDSS	f_{num} [Hz]	9.98	–	12.21	–	–	–
	MAC [-]	0.98	–	0.99	–	–	–
SDDS	f_{num} [Hz]	9.97	–	12.75	–	–	–
	MAC [-]	0.98	–	0.99	–	–	–
DDDS	f_{num} [Hz]	9.97	11.05	12.75	–	–	–
	MAC [-]	0.98	0.97	0.99	–	–	–

Table 2: (top) First six experimental frequencies, their corresponding modal damping ratios and (bottom) frequencies and AutoMAC of the paired numerical modes.

tween both the bridge spans and the adjacent decks induced by the continuity of the ballasted track. The prediction of the fundamental mode is less affected by the coupling and the restraining effect on the bridges' boundary conditions exerted by the ballasted track, since the three approaches reach a satisfactory correspondence with the measurements. In the third mode, the consideration of the span coupling with the introduction of the track in the numerical model improves the prediction of the natural frequency, while the MAC value remains unaffected when compared to the other numerical approaches. And finally, the second mode corresponding to a first torsion mode of the double track deck in each span, is only predicted when the coupling caused by the ballast layer that is shared between the two adjacent decks is considered.

Fig. 4 shows, in solid red trace, the paired numerical modes predicted with the DDDS FE model. When the MAC values are calculated considering all the measurement points (A1 to A18) the values change to 0.95, 0.74 and 0.99 for the first three modes, respectively. The MAC of the second mode worsens significantly, and it is caused by the measurements of the sensors A12 and A13 located in the other span.

For a better understanding of the effect of the ballasted track coupling parameters, K_w and K_{wL} , on the natural frequencies and MAC values of the first three experimental modes, a sensitivity analysis has been performed considering variations of each of the cited parameters while the others remain constant according to the values provided in table 1. The proposed variations are: $[1/100 \ 1/50 \ 1/10 \ 1/5 \ 1/2 \ 1 \ 10 \ 50 \ 100 \ 10000] \cdot K_w$ and $[1/100 \ 1/50 \ 1/10 \ 1/5 \ 1/2 \ 1 \ 10 \ 50 \ 100] \cdot K_{wL}$.

The results in terms of frequency difference, calculated as $(f_{num} - f_{exp})/f_{exp} \times 100$ and MAC values are shown in Fig.6. The results corresponding to variations of the coupling between spans K_w are shown in black trace, while the coupling between adjacent spans K_{wL} is shown in red trace. As can be seen, there exists an optimum value that ensures the best fitting with the experimental measurements .

4.2 Bridge response under railway traffic

During the experimental campaign performed by the authors in In May 2019 a number of trains crossed the bridge at different speeds. The forced bridge vibrations were recorded showing that the maximum acceleration levels did not exceed the Serviceability Limit State for traffic safety in ballasted tracks, limited to 3.5 m/s^2 as per European Standards [1]. The circulation of one of these trains, the medium distance Renfe Altaría Talgo VI train, is included in this section. It is a regular train with a characteristic distance between axles of the passenger cars of 13.14 m.

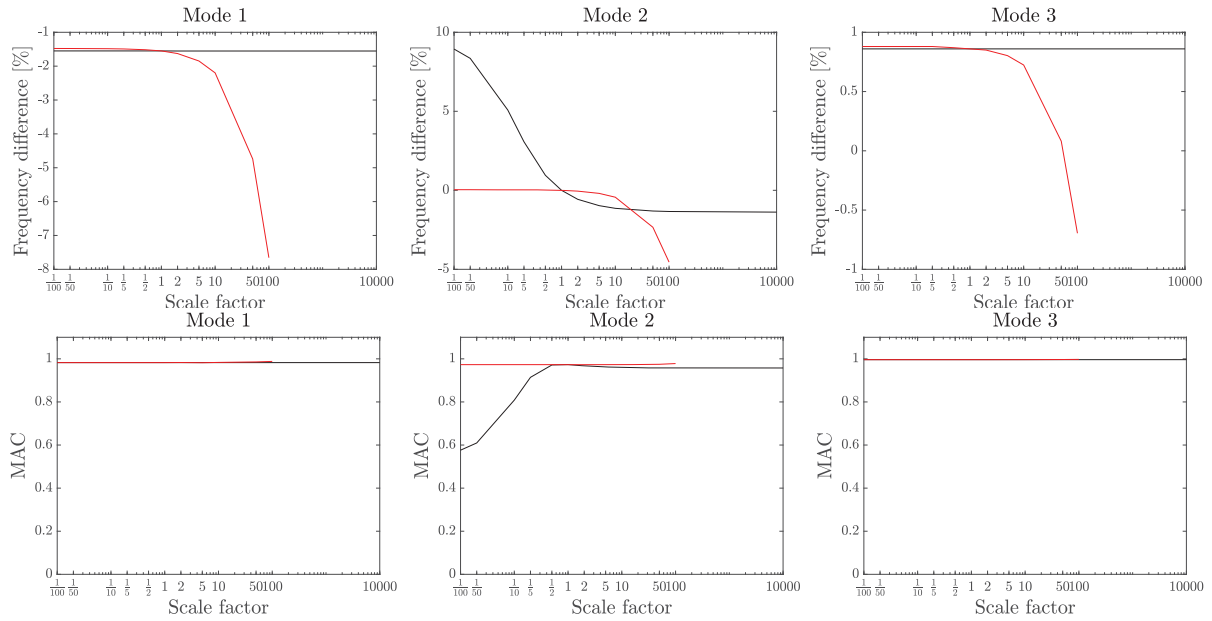


Figure 6: Sensitivity analysis of the influence of K_w (black) and K_{wL} (red trace) on the natural frequencies and MAC values

Figure 7 and Table 3 show the axle scheme and loads. More information about the train can be found in [22]. It travels along track 2 in the direction South-North (Manzanares-Alcázar de San Juan). Its circulation speed was identified from the frequency associated to the bogie distance leading to approximately 155 km/h, which is very close to the theoretical speed associated to a third resonance of the fundamental mode (159 km/h, approximately).

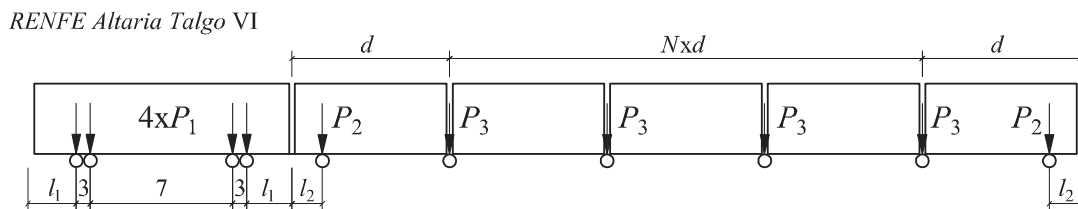


Figure 7: RENFE Altaría Talgo VI axle scheme

Train	N	d [m]	l_1 [m]	l_1 [m]	P_1 [kN]	P_2 [kN]	P_3 [kN]
Altaría	7	13.14	3.44	3.3	225	70	140

Table 3: RENFE Altaría Talgo VI features.

The response of the bridge was calculated using the numerical model described in section 3.1 with the properties included in table 1 by Mode Superposition, including modal contributions up to 30 Hz as per European Standards [1]. A track length of 20 m is included before and after the two-span bridge, a sensitivity analysis of this length was previously performed to guarantee the convergence of the dynamic results.

In a first approach the additional damping introduced by the track elements is therefore neglected, which is also in accordance with previous works [5, 11, 12]. The modal damping

obtained in the experimental campaign was assigned to the paired numerical modes, for the other modes of frequencies up to 30 Hz a value of 1.56% is assumed (Eurocode value [1]).

Fig.8 shows the vertical acceleration response under the passage of the train at different points of the deck, in particular points 2, 5 and 18. The sensors 2 and 5 are located under the loaded decks and the other one under the unloaded deck. The response is plotted in the time domain (first row of the figure) and frequency domain (last row of the figure). In all the plots the experimental signal, plotted in solid black trace, is filtered applying a two third-order Chebyshev filter with high-pass and low-pass frequencies of 1 Hz and 30 Hz. The numerical predictions are plotted with different colours: red line is used for the SDSS model, blue for the SDDS model and green for the DDDS model. Concerning numerical predictions, results at point 18 are only available in the DDDS model, since decks 1 are 2 are uncoupled in the remaining ones and therefore, the acceleration results are zero in the unloaded deck.

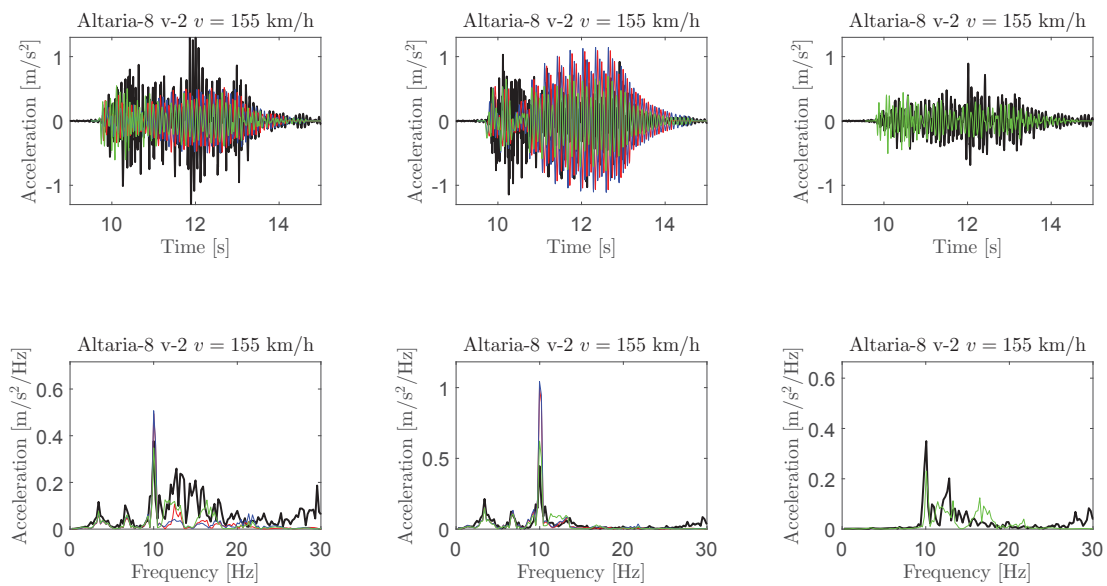


Figure 8: Time history (top) and frequency content (bottom) of the acceleration at point 2 (left), point 5 (center) and point 18 (right) induced by Renfe Altaria: experimental (black line), SDSS model (red line), SDDS model (blue line) and DDDS model (green line)

As can be seen from the frequency domain plots the vertical acceleration response of the bridge is caused by several mode contributions apart from the longitudinal bending one. However, the peak amplitude associated to the fundamental mode predominates when compared to the other frequency contributions, as expected in a resonance situation. This is especially clear at point 5, and is in accordance with the sensor location (at mid-span under the loaded track). The response also presents peaks at low frequencies (in the vicinity of 3 Hz and 6 Hz) associated to the excitation and corresponding to the axle passing frequency (i.e. ratio of train speed v to axle distances v/d_{axle} , respectively, and corresponding multiples). For sensor 18, located in the unloaded deck, the response is still relevant, showing an important coupling between adjacent decks of the same span through the ballast. This effect was also detected under ambient vibration in the mode shapes and seems to be also important under forced vibrations, despite the higher level of vertical vibrations in this case. This coupling effect could be also associated to the shared foundation between both decks.

Regarding the comparison between the response predicted by the three numerical approaches, it is noticeable that the model that considers both the longitudinal and transverse coupling between decks (DDDS model) predicts the frequency contributions in the range [10-15] Hz with higher accuracy, but the three of them overestimate the contribution of the fundamental mode in the response. The authors consider that the additional damping induced by the track elements, that is neglected in this preliminary study and also the effect of the train-bridge interaction, could be responsible of the differences and needs to be investigated in a future work. At frequencies above 15 Hz the predictions worsen, which is in accordance with the model updating, since only the first three experimental modes were successfully identified in the numerical models. In the time domain response, the models tend to underestimate the amplitude of the vibration at points 2 and 18, where the contribution of modes above 15 Hz play a more significant role than at point 5 due to the sensor location. The damping associated to the high frequency modes in the numerical calculations and also the unsuccessful mode pairing with the ones predicted by the numerical models, can be an issue and require further research.

5 CONCLUSIONS

In this work a preliminary evaluation of the effect of the ballasted track continuity on the dynamic response of railway bridges formed by simply-supported spans with structurally independent adjacent decks at each span, is addressed. First, a preliminary FE model that considers the track and therefore, the coupling between adjacent decks and spans has been implemented. The numerical results are compared with the experimental measurements performed on a real bridge formed by several independent decks, and the following conclusions can be extracted:

- The coupling effect exerted by the ballasted track between consecutive spans and adjacent decks that form the same span in railway bridges is clear. In the particular bridge of study, the coupling effect between adjacent decks of the same span predominates over the coupling between different spans. This issue can be detected under both ambient and train-induced vibrations despite the different level of vibration amplitude induced by these excitations. In this regard, the shared foundations of the two decks may also have an important role on this coupling.
- The transmission of vibrations from the loaded deck to the unloaded deck is relevant. The numerical predictions reveal that a more accurate model updating can be achieved if both coupling effects are considered. In terms of natural frequencies and mode shapes, the sole introduction of the coupling between SS spans in the numerical model does not improve the predictions when compared to the ones obtained when the track is neglected for this particular bridge.
- The three implemented numerical approaches overestimate the contribution of the fundamental mode in the response, thus the prediction is more accurate when both couplings effects are considered. The additional damping induced by the track elements, that is neglected in this preliminary study and also the effect of the train-bridge interaction require further research. Also the shared foundation between decks belonging to the same span can have an influence on the deck coupling that is not well known up to date.
- The three implemented numerical approaches overestimate the contribution of the fundamental mode in the response, thus the prediction is more accurate when both couplings

effects are considered. The additional damping induced by the track elements, that is neglected in this preliminary study and also the effect of the train-bridge interaction, could be responsible of the differences and require further research. Also the shared foundation between decks belonging to the same span can have an influence on the deck coupling that is not well known up to date.

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