Advanced Evaluation of Heavy Vehicle Impact on flexible road viaduct expansion joints

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Abstract

One of the most common challenges in maintaining viaduct decks in optimal condition is ensuring the durability and performance of road expansion joints (REJ). These elements are particularly susceptible to deterioration due to the continuous impact and dynamic loads generated by heavy vehicle traffic. This study focuses on the structural response of flexible REJ under such conditions and proposes an optimized method for simplified analysis of their behavior. The proposed methodology is validated through both theoretical and empirical approaches. The theoretical analysis compares the structural response of flexible REJ subjected to heavy vehicles at varying speeds, contrasting the results obtained using the proposed method with those derived from traditional Finite Element Method (FEM) simulations. Meanwhile, the empirical analysis is conducted by monitoring the in-service behavior of a flexible REJ installed on a pergola over a railway line within the Spanish motorway network. The findings demonstrate that the proposed approach provides reliable results while significantly reducing computational time, energy consumption, and CO_2 emissions compared to conventional methods. Furthermore, its straightforward integration into digital asset management systems enhances its applicability for infrastructure maintenance, offering a cost-effective and sustainable solution for optimizing the service life of REJ.

Keywords

Road expansion joints on viaducts, Impact on structures, Modal analysis of structures, Structural health monitoring, Pergola viaduct decks.

1. Introduction

When designing the layout of linear works, the intersection of two alignments with a deviation as close as possible to 90° is always desirable. This deviation assumes a minimum calculation span and intersection area. On the other hand, there are cases where the intersection of two alignments with a very small deviation is necessary. In these cases, the design span of the upper alignment is much larger than the strict span in the case of a perpendicular intersection. The pre-dominant structure in the case of intersections with small deflection angles are pergolas. These structures are designed with a beam arrangement perpendicular to the axis of the lower alignment, so that the bending span of the load-bearing structural elements is reduced to a minimum [1-3]. In the design of pergola viaducts, the use of expansion joints coinciding with the intersection of the upper roadway and the centre span of one of the supporting beams creates a fexible roadway joint. These joints are susceptible to dynamic problems due to the dynamic amplification generated by the impact of passing heavy vehicles. The potential level of damage that vehicle impacts can cause to these structural elements makes it necessary to improve knowledge on the behaviour of structures under impact phenomena [4-6]. The main design codes treat these impacts as an equivalent static force [7,8]. The impact process is a complex event but, on many occasions, an accurate simplified method can provide sufficiently accurate results [9]; the key point is to know when this simplified method can be ap-plied [10,11].

This paper presents an analytical formulation to analyse the impact of heavy vehicles on flexible roadway joints using a generalised spring-mass system (1DoF) for each vibration mode of the pergola viaduct considered in the calculation. The results obtained by applying the proposed optimised method are compared with the results of a detailed analysis using a Finite Element Model (FEM), modelled using the structural calculation program Midas Civil [12]. Additionally, the solution is experimentally validated by means of a dynamic load test on a pergola over a railway line on a motorway on the Spanish road network.

2. Conventional methods

In the design and dimensioning of pergola viaducts, it is common to use spatial grid structural models [13] [14]. In these models, the bearing capacity of the structure is provided by the main girders spanning the span between the pergola abutments (Fig.1). The total longitudinal bending stiffness of the viaduct deck is given by the stiffness of the main girders and the effective width of the top slab collaborating with each main girder [15,16]. The load distribution is realised by the transverse stiffness provided by the top slab in collaboration with the torsional stiffness of the main girders. This type of structural models allows the static and dynamic characterisation of pergola viaducts, adequately reflecting the phenomena of transverse deformation and longitudinal bending of the viaduct deck. In the analysis of spatial grid models, it is necessary to use specific struc-tural calculation programmes to obtain the structural response of the pergola viaducts to the moving loads caused by heavy vehicles. This process is usually complex and time-consuming, so it is interesting to idealise simplified methods to optimise the process of calculating the dynamic amplification of the structural response generated by the passage of heavy vehicles.

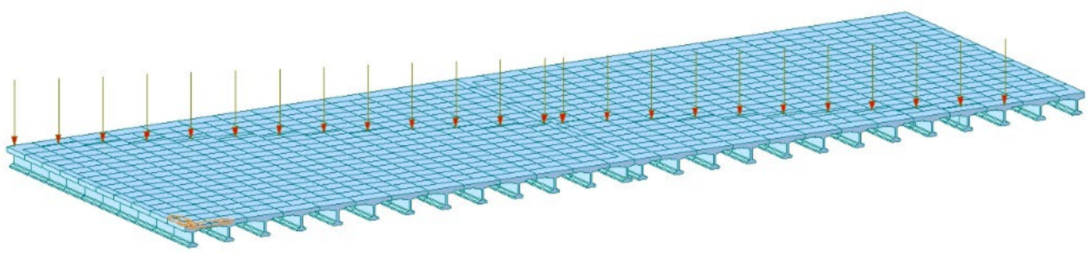


Figure 1. Structural model of a pergola viaduct with 'n' main girders.

3. Proposed method

A matrix method is proposed based on the use of a virtual model reflecting the transverse distribution of the pergola viaduct by means of a slab supported on a series of springs reflecting the bending stiffness eq. (1) and torsional stiffness eq. (2) of the main girders spanning the pergola abutments (Fig.2). Two degrees of freedom (DOF) are considered for each main girder: (1) the deflection and (2) the rotation of the top slab in the central span of the main girder. The matrix resolution of the transverse distribution of the moving loads is materialised by means of the matrix eqs. (3, 4 and 5) [17,18].

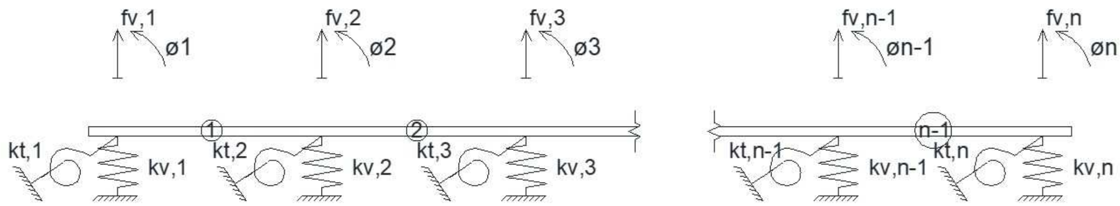


Figure 2. Structural model of a pergola viaduct with 'n' main girders.

The proposed method considers 2 degrees of freedom for each longitudinal rib of the bridge deck: (1) the deflection and (2) the rotation of the bridge deck slab at the centre of the span of the longitudinal rib. Figure 2 represents the schematic of the structural model of a bridge deck with a single cell box girder. The matrix approach that solves the structural problem of the transverse distribution of the live loads between the different longitudinal ribs composing the bridge deck is presented in equations (1), (2) and (3).

$$K_{v,i} = \frac{48 \cdot EI_i}{{L_i}^3} \tag{1}$$

$$K_{t,i} = \frac{2 \cdot GJ_i}{L_i} \tag{2}$$

$$\overline{K_e} = \frac{EI_e}{L_e^3}\begin{pmatrix} 12 & 6L_e & -12 & 6L_e \\ 6L & 4{L_e}^2 & -6L_e & 2{L_e}^2 \\ -12 & -6L_e & 12 & -6L_e \\ 6L & 2{L_e}^2 & -6L_e & 4{L_e}^2 \end{pmatrix} = \begin{pmatrix} \overline{K_{11,e}} & \overline{K_{12,e}} \\ \overline{K_{21,e}} & \overline{K_{22,e}} \end{pmatrix} \tag{3}$$

$$\overline{K_{B\iota}} = \begin{pmatrix} k_{v,i} & 0 \\ 0 & k_{t,i} \end{pmatrix} \tag{4}$$

$$\begin{pmatrix} P_1 \\ M_{t,1} \\ P_2 \\ M_{t,2} \\ P_3 \\ M_{t,3} \\ \vdots \\ P_{n-1} \\ M_{t,n-1} \\ P_n \\ M_{t,n} \end{pmatrix} = \begin{pmatrix} \overline{K_{11,1}} + \overline{K_{B1}} & K_{12,1} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \cdots & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} \\ \overline{K_{21,1}} & \overline{K_{22,1}} + \overline{K_{11,2}} + \overline{K_{B2}} & \overline{K_{12,2}} & & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} \\ \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & K_{21,2} & \overline{K_{22,2}} + \overline{K_{11,3}} + \overline{K_{B3}} & & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} \\ \vdots & & & \ddots & & \vdots \\ \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & & \overline{K_{22,n-2}} + \overline{K_{11,n-1}} + \overline{K_{Bn-1}} & \overline{K_{12,n-1}} \\ \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \begin{smallmatrix}0&0\\0&0\end{smallmatrix} & \cdots & K_{21,n-1} & \overline{K_{22,n-1}} + \overline{K_{Bn}} \end{pmatrix} \cdot \begin{pmatrix} f_{v1} \\ \theta_1 \\ f_{v2} \\ \theta_2 \\ f_{v3} \\ \theta_3 \\ \vdots \\ f_{vn-1} \\ \theta_{n-1} \\ f_{vn} \\ \theta_n \end{pmatrix} \tag{5}$$

Where EI_i = longitudinal bending stiffness of main girder "i"; GJ_i = longitudinal torsional stiffness of main girder "i"; L_i = span of main girder i; EI_e = transverse bending stiffness of the top deck slab element "e"; L_e = length of the top deck slab element "e".

3.1. Modal analysis

A concentrated mass dynamic structural model is proposed in which a single DOF is considered for each main girder of the pergola viaduct, the deflection at the centre span of the girder (Fig. 3). The DOFs associated with the torsion of the central span of the main girders and the bending of the top slab are condensed into the DOFs associated with the deflection of the main girders by means of the static condensation eq.(7) [19-21]. Previously, it will be necessary to rear-range the global stiffness matrix, so that the DOFs associated with the vertical movement appear first, and finally the DOFs associated with the rotation eq.(6).The concentrated masses that are activated by the vertical movement experienced by the centre span section of the main girders of the pergola viaduct "mi" are the result of the sum of the following values: (a) half of the mass associated with each main girder; (b) half of the mass associated with the part of the upper slab that gravitates on each main girder eq.(8). By means of the dynamic equilibrium in free vibrations eq.(9) at the initial instant of motion, the matrix equation eq.(10) is obtained, which allows obtaining the frequencies and eigenmodes functions of vibration of the pergola viaduct [22-24]. The columns of the eigenmode function matrix "Øn" correspond to the deformation of the pergola viaduct cross-section coincident with the centre of span of the main girders and associated to each vibration mode analysed. The eigenmode function associated with each main girder "Øn,Gi" is approximated by a sinusoidal function eq.(11). The proposed simplified method analyzes the modes of vibration whose number "n" is greater than the number of main girders of the deck (n≥NB) by means of the matrix equations eqs. (8) and (9) posed for for a pergola viaduct deck with equivalent span "Leq=L/N". The numbering of the modes of vibration is defined by their structural stiffness and the mobilized mass. The fundamental modes of vibration of the pergola viaduct deck correspond to the most flexible and mass mobilizing modes, associated with a lower vibration frequency. Figures 4 and 5 show the structural model and shape functions of the eigenmodes of a pergola viaduct deck with twenty-six main girders with a support span of 20.40 m, a main girder spacing of 2.39 m and a depth of 1.25 m.

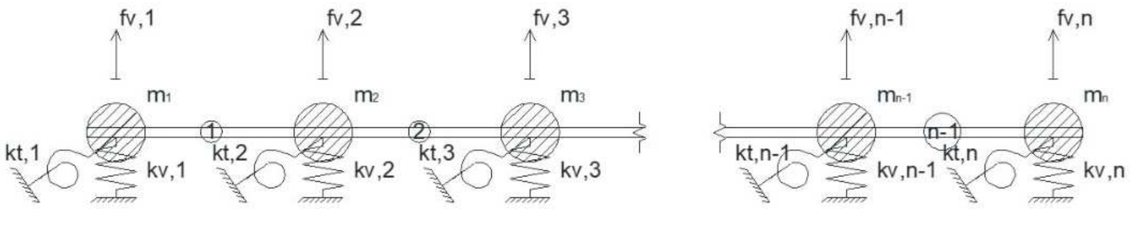


Figure 3. Structural model of concentrated masses.

$$\begin{pmatrix} P_1 \\ P_2 \\ P_3 \\ \vdots \\ P_{n-1} \\ P_n \\ M_{t,1} \\ M_{t,2} \\ M_{t,3} \\ \vdots \\ M_{t,n-1} \\ M_{t,n} \end{pmatrix} = \begin{pmatrix} \overline{K_{VV}} & \overline{K_{V\theta}} \\ \overline{K_{\theta V}} & \overline{K_{\theta\theta}} \end{pmatrix} = \begin{pmatrix} f_{v,1} \\ f_{v,2} \\ f_{v,3} \\ \vdots \\ f_{v,n-1} \\ f_{v,n} \\ \theta_1 \\ \theta_2 \\ \theta_3 \\ \vdots \\ \theta_{n-1} \\ \theta_n \end{pmatrix} \tag{6}$$

$$\overline{K_{DD}} = \left(\overline{K_{VV}} - \overline{K_{V\theta}} \cdot \overline{K_{\theta\theta}}^{-1} \cdot \overline{K_{\theta V}}\right) \tag{7}$$

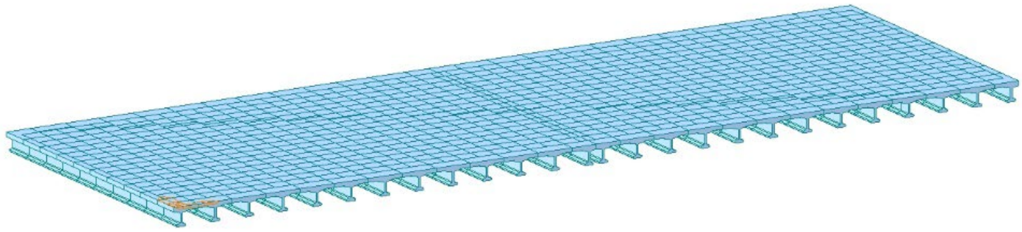
$$m_i = \int_0^L m \cdot \left[\sin\left(\frac{\pi \cdot x}{L}\right)\right]^2 \cdot dx = m \cdot \frac{L}{2} \tag{8}$$

$$\overline{M} \cdot \vec{\ddot{u}} + \bar{C} \cdot \vec{\dot{u}} + \overline{K_{DD}} \cdot \vec{u} = \vec{0} \tag{9}$$

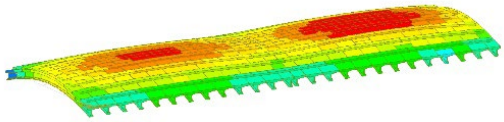
$$({w_n}^2 \cdot \bar{I} + \overline{M}^{-1} \cdot \overline{K_{DD}}) \cdot \overline{\emptyset} = \vec{0} \tag{10}$$

$$\emptyset_{n,Gi} = \sin\left(\frac{\pi \cdot x}{L/_N}\right) \cdot \emptyset_n(i) \quad / \quad N = ceil\left(\frac{n}{NB}\right) \tag{11}$$

Where: $\overline{K_{DD}}$ = condensed stiffness matrix in the degrees of freedom associated with the vertical movement of the central span section of the longitudinal girders; m_i = Point mass associated with degree of freedom “i”; m = Mass per linear meter gravitating on main girder “i”; $\overline{M}$= Concentrated mass matrix; $\bar{C}$= Structural damping matrix; $\vec{u}$ = Motion vector; wn= Angular pulsation associated with mode “n”; $\overline{\emptyset}$ = Eigen mode function matrix; Øn= Eigen mode function associated with mode “n”; Øn,Gi = Eigen mode function associated with main girder “i” and mode “n”; NB = Number of main girders.



(a)



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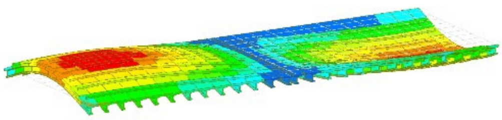

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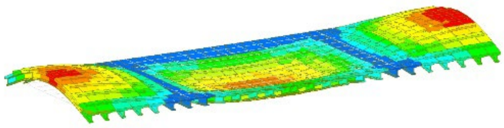

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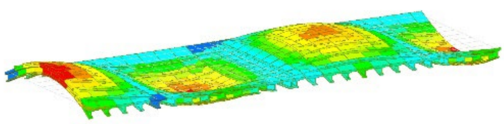

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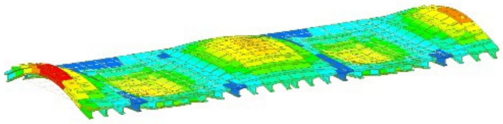

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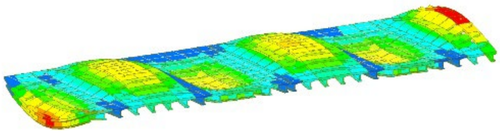

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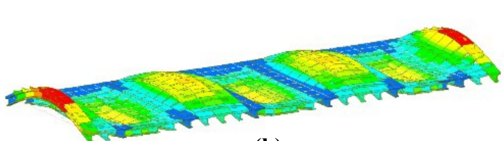

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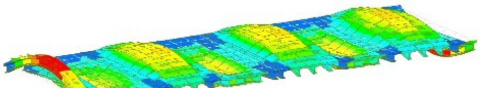
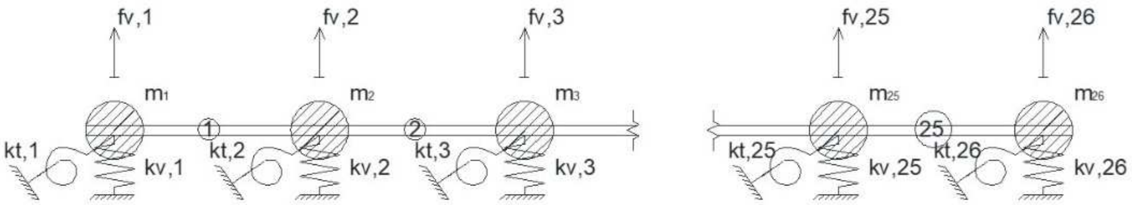
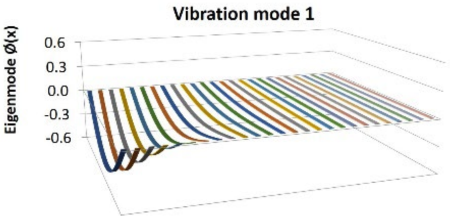


(i)

Figure 4. Modal analysis of a pergola viaduct deck using FEM: a) structural model; b) vibration mode 1; c) vibration mode 2; d) vibration mode 3; e) vibration mode 4; f) vibration mode 5; g) vibration mode 6; h) vibration mode 7; i) vibration mode 8.



(a)



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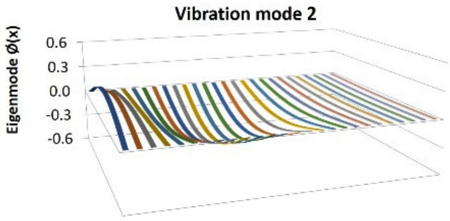

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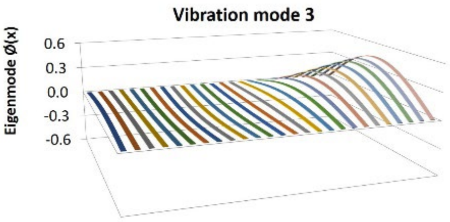

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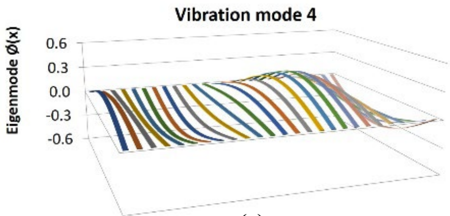

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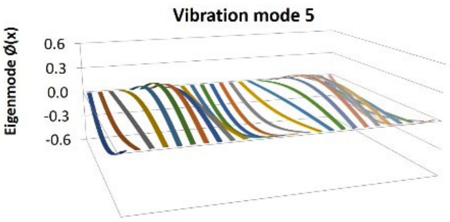

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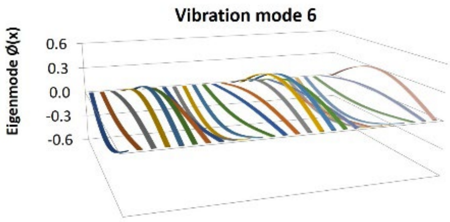

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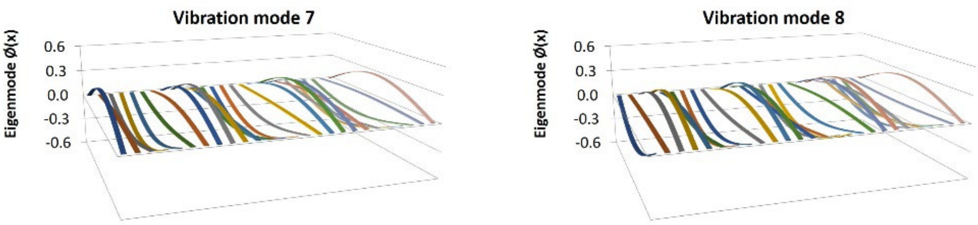


(h) (i)

Figure 5. Modal analysis of a pergola viaduct deck using the model proposed by the authors: a) structural model; b) mode 1 shape function; c) mode 2 shape function; d) mode 3 shape function; e) mode 4 shape function; f) mode 5 shape function; g) mode 6 shape function; h) mode 7 shape function; i) mode 8 shape function.

3.2. Structural response of the pergola viaduct to vehicle impacts

In order to study the structural response of flexible roadway joints in pergola viaducts to the impact produced by the passage of heavy vehicles, the authors propose an analysis based on the assimilation of the effect of the passage of each axle of the heavy vehicle as a triangular impulse on the roadway joint (Fig. 6). For this purpose, the structural response associated with each vibration mode will be analysed using a generalised system with a single DoF eq.(12) (Fig. 7), whose generalised mass (mn), damping (Cn) and stiffness (Kn) are given by eqs. (13-15) respectively. In the structural response of the flexible roadway joint, 2 phases are distinguished: P1) a first phase in which the tirangular impulse generated by the passage of the axle continues to act on the roadway joint; P2) and a second phase in which the behaviour of the roadway joint in free vibrations is studied, after the passage of the axle of the heavy vehicle under analysis. The modal structural response for study phases 1 and 2 is given by eqs. (16,17) respectively. The overall structural response of the pergola viaduct deck will be the sum of the structural response associated with each mode of vibration eq. (18).

$$\ddot{y_n} + 2 \cdot \eta \cdot w_n \cdot \dot{y_n} + {w_n}^2 \cdot y_n = \frac{{\emptyset_n}^t \cdot P \cdot \left(1 - \frac{t}{t_1}\right)}{m_n} \tag{12}$$

$$m_n = {\emptyset_n}^t \cdot m \cdot \emptyset_n \tag{13}$$

$$C_n = {\emptyset_n}^t \cdot C \cdot \emptyset_n \tag{14}$$

$$K_n = {\emptyset_n}^t \cdot K \cdot \emptyset_n \tag{15}$$

$$y_{P1,n}(t) = \frac{\frac{{\emptyset_n}^t \cdot \frac{P}{K_n}}{\sum_{n=1}^{N} {\emptyset_n}^t \cdot \frac{P}{K_n}}}{f_{st}} \cdot \left[1 - \frac{t}{t_1} + e^{-\eta \cdot w \cdot t} \cdot \left(\frac{1 - \eta \cdot w \cdot t_1}{w_d \cdot t_1} \cdot \sin(w_d \cdot t) - \cos(w_d \cdot t)\right)\right] \tag{16}$$

$$y_{P2,n}(t) = e^{-\eta \cdot w \cdot (t-t_1)} \cdot \left[\frac{\dot{y}(t_1)+\eta \cdot w \cdot y(t_1)}{w_d} \cdot \sin\big(w_d(t-t_1)\big) + y(t_1) \cdot \cos\big(w_d \cdot (t-t_1)\big)\right] \tag{17}$$

$$y(t) = \sum_{n=1}^{N} y_n(t) \tag{18}$$

Where: y_n = deflection at the road way joint associated with mode 'n'; η = structure damping factor; w_n = angular pulsation associated with mode 'n'; P = static load transmitted by the axle of the heavy vehicle; t_1 = time interval during which the static influence of the heavy vehicle axle on the roadway joint; f_{st} = deflection associated with the load transmitted by the axle of the heavy vehicle; w_d = damped angular pulsation.

Figure 6. Assimilation of the impact of the axles of the heavy vehicle to a triangular impulse.

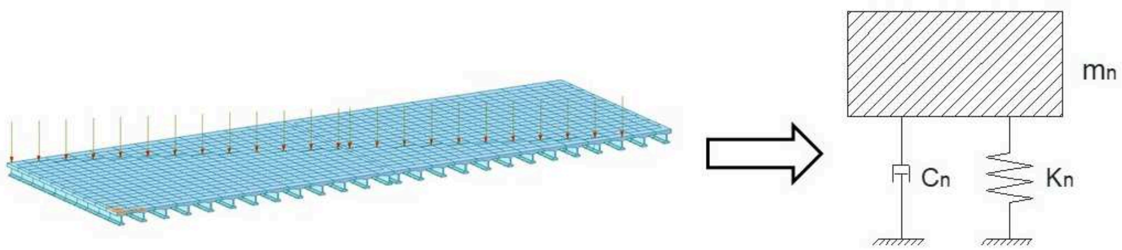


Figure 7. Reduction of the structural model to a single DoF.

4. Validation of the method in a full-scale structure

As an empirical validation of the proposed method, the authors have analysed the structural response of a pergola over a railway line on a motorway on the Spanish road network. The deck consists of 6 sections of 28.42-30.25-29.55-29.51-59.52-29.78 metres. In several of the REJ between road surface sections, the centre of the edge girder of the road surface section coincides with the lane for heavy vehicles on the motorway. This means that the impact of heavy vehicles on the road surface joint causes a dynamic amplification phenomenon that results in the deterioration of the joint. The authors have carried out a threefold analysis of the structural response of the pergola viaduct: 1) empirical analysis using instrumentation; 2) theoretical analysis using FEM; 3) theoretical analysis using the pro-posed method.

4.1. Empirical analysis of the structural response

To characterise empirically the structural response of the pergola viaduct deck, the authors installed a total of four piezoelectric accelerometers, model 'Metra KS48C', which provide the acceleration experienced by the edge beams at the analysed REJ (Fig. 8). The acquisition, recording and supervision of the data provided by the sensors is carried out by a Structural Health Monitoring System (SHMS) consisting of the following elements: (a) a 'NI-CDAQ-9188' model Modular Central Data Acquisition and Processing Unit (MCDA&PU) with the capacity to simultaneously manage the signal from up to eight Data Acquisition Units (DAU); (b) a 'NI-9234' model acceleration DAU that facilitates the processing of the analogue signal from the accelerometers; (c) a workstation responsible for communicating with the MCDA&PU and for recording and displaying the data provided by the sensors through a Data Acquisition and Monitoring Programme designed and programmed by the authors (Fig. 9). The instrumentation arranged in the structure allows us to characterise the frequencies associated with the predominant vibration modes in the structure, as well as the acceleration experienced by the edge beams of the analysed road joint due to the impact produced by the passage of heavy vehicles.

Figure 8. System of sensors arranged in the flexible road surface joint.

Figure 9. Data acquisition and monitoring routine developed by the authors.

4.2. Theoretical analysis of the structural response

Each section of road surface between REJ is made up of 13 main girders with a span of 20.40 m, a separation of 2.39 m and a depth of 1.25 m. A double theoretical analysis of the structural response is carried out, using the proposed method and using an FEM with the Midas Civil structural analysis programme [12]. Both calculation models allow us to obtain the frequencies and shape functions (Fig.10) associated with the main modes of vibration of the structure, as well as the record of accelerations experienced by the flexible joint of the road surface when heavy vehicles pass over it. A heavy 6-axle vehicle has been considered, with a load distribution of 65+75+75+50+50+50 KN and a wheelbase of 3.2-1.13-5.96-1.13-1.13 m. The following five speeds for heavy vehicles have been analysed: 1) 75 km/h; 2) 80 km/h; 3) 85 km/h; 4) 90 km/h; 5) 95 km/h.

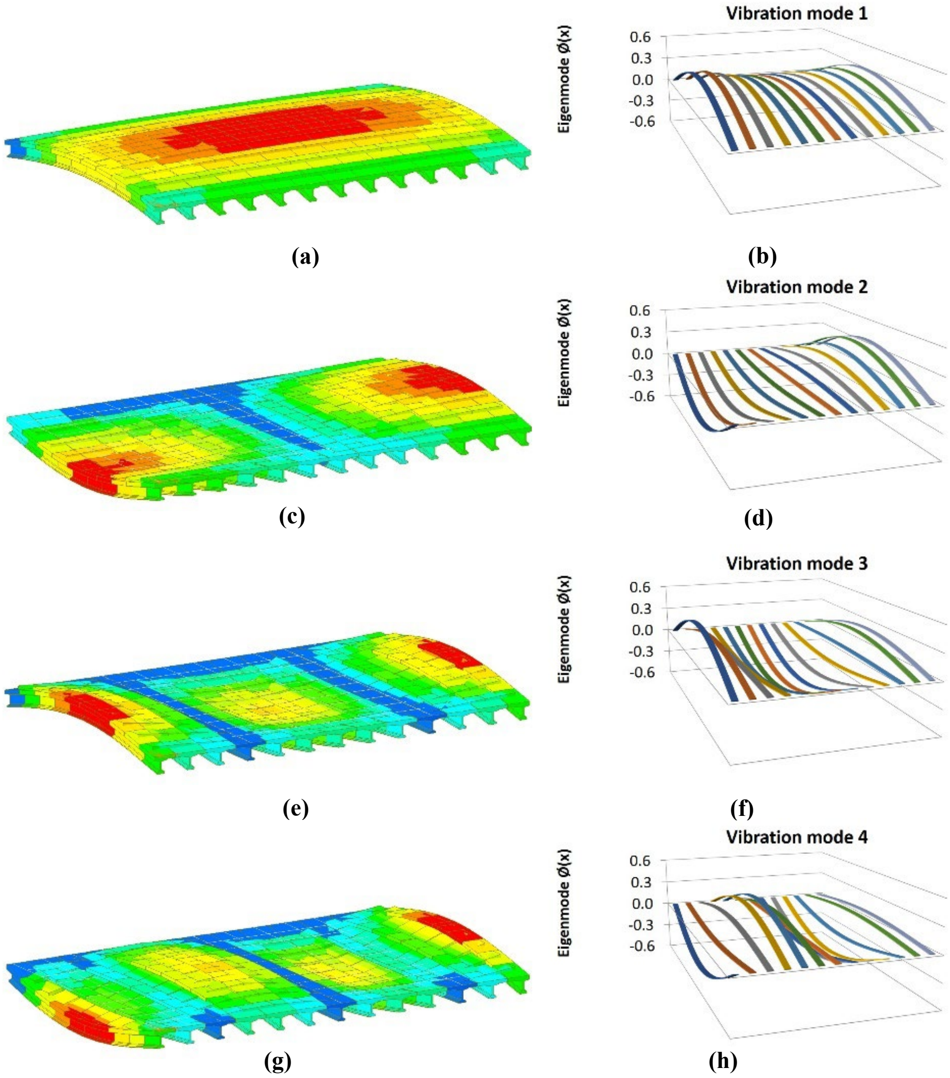
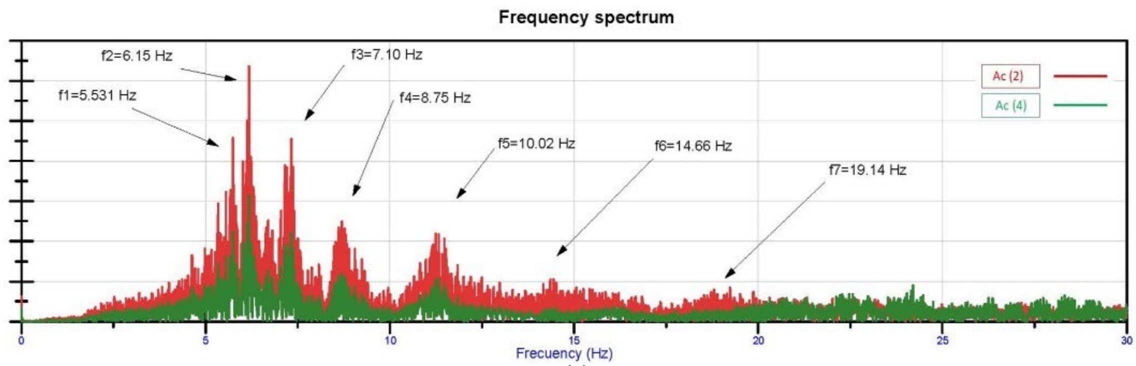


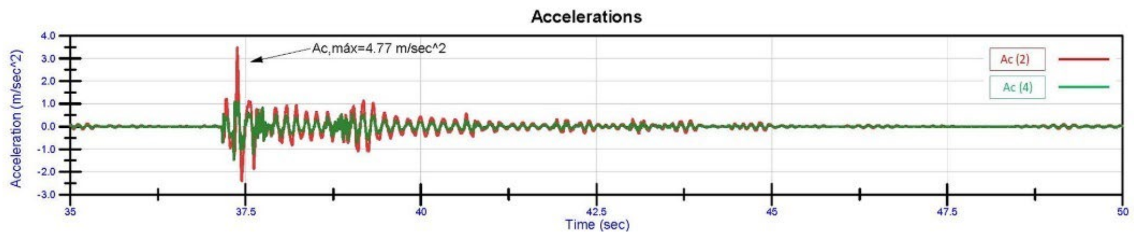
Figure 10. Theoretical modal analysis of the pergola over a railway line on a motorway on the Spanish road network: a) mode 1 shape function (FEM); b) mode 1 shape function (proposed method); c) mode 2 shape function (FEM); d) mode 2 shape function (proposed method); e) mode 3 shape function (FEM); f) mode 3 shape function (proposed method); g) mode 4 shape function (FEM); h) mode 4 shape function (proposed method).

4.3. Results obtained

Analysis of the data provided by the instrumentation installed in the structure (Fig. 11) allows the experimental response of the structure to be compared with the theoretical response obtained using the Midas Civil FEM [12] and the method proposed by the authors. The results of the modal analysis of the structure show an adequate convergence between the values provided by the proposed method and the experimental response of the structure, with a maximum deviation in the predominant vibration modes (modes associated with a vibration frequency of less than 10 Hz) of less than 12% (Table 1). In the theoretical analysis of the maximum accelerations experienced at the road joint using the method proposed by the authors (Fig. 12), a maximum acceleration 20% higher than that obtained by the experimental analysis is obtained, a result that leaves us on the safe side. However, the theoretical analysis using the Midas Civil FEM [12] (Fig. 12) shows a maximum acceleration 14% lower than that obtained by the experimental analysis, a result that would leave us on the side of insecurity (Table 2).

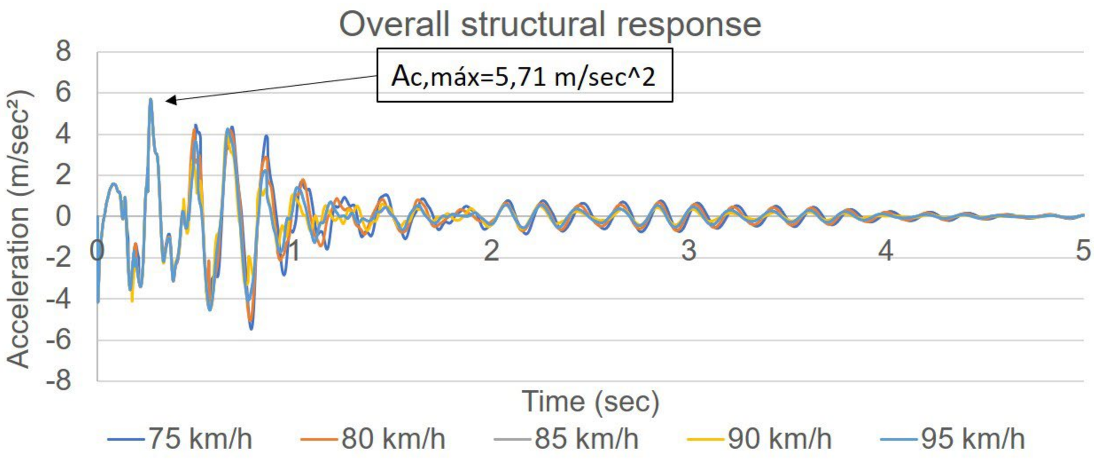


(a)

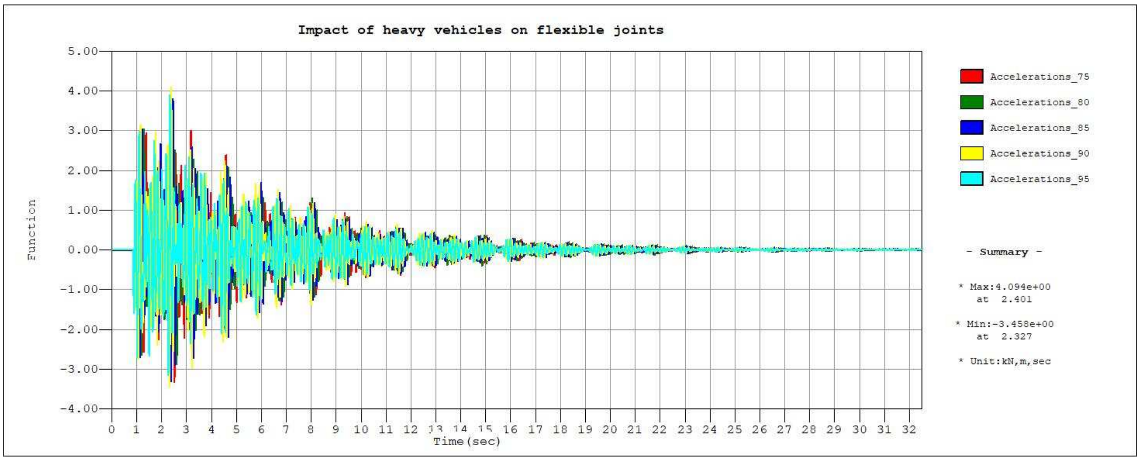


(b)

Figure 11. Analysis of the experimental response of the road joint to the impact of heavy vehicles: a) modal analysis; b) maximum accelerations.



(a)



(b)

Figure 12. Maximum accelerations experienced by the flexible road joint due to the passage of heavy vehicles.: a) proposed method; b) Midas Civil FEM.

Table 1. Theoretical/empirical modal analysis of the pergola over a railway line on a motorway on the Spanish road network.

	Frequency (Hz)		
Mode of vibration	**Proposed method**	**FEM**	**Experimental Analysis**
1	5.06	4.89	5.53
2	5.07	5.04	
3	5.39	5.16	6.15
4	6.50	5.63	7.10
5	8.90	6.49	8.75
6	12.70	7.90	10.02
7	17.76	10.00	14.66
8	19.60	12.76	19.14

Table 2. Theoretical/empirical analysis of the maximum accelerations experienced in the REJ.

Experimental analysis	Proposed method	FEM

Acceleration (m/sec^2)	Acceleration (m/sec^2)	Divergence	Acceleration (m/sec^2)	Divergence
4.77	5.71	20%	4.09	-14%

5. Summary and conclusions

The proposed method has been successfully presented and validated through both theoretical and empirical analyses. The theoretical verification was conducted using the Finite Element Method (FEM) via the Midas Civil structural calculation program [12], while the empirical validation was based on experimental monitoring of a flexible road expansion joint (REJ) installed on a pergola over a railway line within the Spanish motorway network. The modal analysis of the viaduct deck demonstrated a strong correlation between the structural response obtained by FEM and the proposed method, with a maximum deviation of less than 4% in the predominant vibration modes. Additionally, the theoretical-empirical comparison further confirmed the reliability of the proposed approach, with a maximum deviation of 12%. The method allows for the accurate estimation of maximum accelerations experienced by flexible REJs under heavy traffic at different speeds. The comparative analysis of these accelerations indicated a maximum divergence of 20%, with the proposed method yielding slightly higher acceleration values, ensuring a conservative and safer assessment. The theoretical study considered a heavy vehicle carrying the maximum legally permitted load in Spain, ensuring the results are applicable to real-world conditions. The study highlights the importance of integrating different data sources both sensor-based and synthetic into a comprehensive framework to maximize their potential. The proposed method could be integrated into a Digital Twin (DT) model of viaducts, ensuring continuous updates and real-time monitoring. Such an approach would facilitate the implementation of a Decision Support System (DSS) for infrastructure management, improving decision-making processes and enabling safer manual navigation from road control centers. The integration of Vehicle-to-Infrastructure (V2I) communication, such as truck platooning and traffic management applications, would further enhance the accuracy and efficiency of monitoring systems by leveraging Artificial Intelligence (AI) for data analysis. The simplicity and efficiency of the proposed method make it highly adaptable to modern viaduct design strategies [25], including heuristic approaches [26-28] that aim to optimize structural performance in an increasingly demanding transportation network. The growing complexity of traffic demands, coupled with exposure to variable environmental conditions, underscores the necessity of continuous structural assessment. The combination of Structural Health Monitoring Systems (SHMS) with AI-driven analysis offers a promising solution to minimize system failures, reduce unexpected downtimes, and optimize maintenance costs [29]. In recent years, the application of DT technology in civil engineering has expanded significantly, particularly in the domains of operation and maintenance (O&M). Traditional SHMSs have generated vast amounts of monitoring data, providing valuable insights into structural behavior over time [30]. With the rise of AI, these datasets can now be processed using advanced algorithms to predict structural health more accurately [31-33]. However, ensuring a balance between data-driven AI approaches and well-established physics-based models remains crucial. The fusion of both paradigms opens new opportunities for advanced condition assessment and indirect monitoring strategies [34], paving the way for a more efficient and resilient approach to infrastructure management in the digital era.

Author Contributions

Conceptualization, A.G-A. and D.G-S.; methodology A.G-A. and D.G-S.; validation A.G-A. and D.G-S; formal analysis, A.G-A., D.G-S., O.R.R-G.. and D.Z-S.; investigation, A.G-A. and D.G-S.; resources, D.G-S.; writing—original draft preparation, A.G-A. and D.G-S.; writing—review and editing, O.R.R-G.. and D.Z-S.; supervision, A.G-A., D.G-S., O.R.R-G.. and D.Z-S.; project administration, A.G-A. and D.G-S.; funding acquisition, D.G-S. All authors have read and agreed to the published version of the manuscript.

Data Availability Statement

The data that support the findings of this study are available from the corresponding author, Gaute A. (alvaro.gaute@unican.es), upon reasonable request.

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Conflict of Interest

The authors declare no conflict of interest.

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