

D2.2 - Worst-case combination of critical parameters of existing bridges

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1 Introduction

The development of a Single European Railway Area is a key objective of Europe's Rail Joint Undertaking (EU-Rail), aiming to harmonize standards and improve interoperability across Europe. A critical challenge in this initiative is to enhance the structural analysis of railway bridges to optimize capacity, service quality and cost efficiency. Railway bridges play a fundamental role in rail transport networks, yet their dynamic response under operating conditions remains a complex issue requiring advanced methodologies for accurate assessment.

Launched on September 1, 2023, the InBridge4EU project (<https://inbridge4eu.eu/>) aims to develop a dynamic interface between railway bridges and rolling stock, proposing new methods compatible with existing regulations, namely INF TSI [1], LOC&PAS TSI [2], EN 15528 [3], EN 1990-Annex A2 [4] and EN 1991-2 [5], and approaching the analysis of existing infrastructures, which role is critical for the sustainability of the European rail system. The project is funded by EU-Rail under Horizon Europe research and innovation program. The project is structured into seven work packages, the second of which is devoted to identifying the critical parameters that govern the vertical acceleration of the deck, ultimately related to traffic safety.

During the first year of the project, with the support of five railway Infrastructure Managers, an extensive and representative set of European railway bridges from 11 conventional and High-Speed (HS) lines are selected. Main structural features, including technical drawings and project documents, are retrieved and stored in a database: <https://computeruse.us.es/>. Starting from the beginning of the second year, time-step calculation (TSC) transient dynamic analyses are performed over the complete database to obtain the bridges dynamic response under the circulation of 26 train models, selected based on their dynamic signature to be an envelope of more than 300 existing High-Speed trains across Europe. From the statistical analysis of these results, realistic worst-case combinations of critical parameters leading to the maximum vibrational response of the bridges are identified.

The database is a tool for the management of data generated within the project. It includes information on bridges, data from experimental tests and results from numerical analyses. The application was developed using **Python**. Data visualization is effectively achieved by means of both GET and POST methods, taking advantage of the capabilities of **Matlab**, **Python** and **Julia** for comprehensive analyses and visualisation. The implementation of a virtual private cloud ensures enhanced security measures. Users can securely access the system using user login credentials and API tokens, with group permissions facilitating control over access levels and functionalities. The Deliverable 2.1 [6] includes a description of the structure and contents of the bridge parameters database and access to it.

The integration of large-scale databases and advanced computational techniques is currently redefining structural assessment and maintenance strategies for bridges [7]. In this context, Jesus et al. [8] presented a statistical analysis of the dynamic response of a railway viaduct concluding that key parameters cannot be analysed individually because in some cases interaction effects can be more important than single effects. Monti et al. [9] proposed a fully automated, model-free decision support system to assist railway operators in bridge safety management via real-time monitoring data. Regarding lifecycle management, Collings [10] evaluated data from both in-service and decommissioned structures to identify critical parameters for structural assessment. Other

research has focused on specific structural components; for instance, Shen et al. [11] utilized a quasi-static test database and physics-enhanced convolutional neural networks to estimate the bearing capacity of railway bridge piers. Similarly, Elnahla et al. [12] leveraged non-destructive evaluation data to establish a quantitative framework for predicting bridge deck deterioration. Complementarily, Bayane et al. [13] introduced a real-time approach for damage detection in operational bridges relying on five unsupervised anomaly detection algorithms. Furthermore, the use of extensive synthetic datasets has proven essential for complex dynamic studies, as demonstrated by Le Nguyen et al. [14], who employed a numerical database of over 10000 bridges for the generative design of High-Speed ballasted structures.

The objective of Deliverable 2.2 (D2.2) is to systematically identify realistic combinations of bridge parameters within the compiled database and determine the most critical parameters that induce maximum vertical deck accelerations during train passages. By characterizing the statistical distribution of these critical parameters, this report aims to establish a robust predictive model based on the project's data. Ultimately, this model will provide clear insights into the key factors influencing different bridge typologies, facilitating the application of these findings to railway bridge design while supporting interoperability across Europe.

This deliverable is a component of Work Package 2 (WP2), which is titled "Identification of critical bridge parameters for the assessment of the economic impact of the new DTCs". WP2 specifically focuses on obtaining the critical structural and operational parameters that govern the vertical acceleration of bridge decks. Because excessive vertical deck acceleration can lead to railway traffic safety and track stability problems, understanding these parameters is essential for evaluating both structural performance and subsequent economic implications.

The analysis presented in D2.2 builds directly upon the foundational achievements of preceding tasks, most notably the creation of the comprehensive bridge parameter database in Deliverable 2.1 (D2.1) [6]. Collaborating with five European railway infrastructure managers during the first year of the project, technical data, plans, and design documents were gathered for an extensive set of bridges spanning eleven conventional and high-speed lines. At the beginning of the second year, simplified dynamic time-history analyses were conducted using beam models to simulate the passage of twenty-six representative train models across all 509 bridges in the database. These simulations, carried out at speeds ranging from 144 to 450 km/h, mapped the envelope of dynamic signatures for over 300 real European high-speed trains to assess serviceability limit states.

To refine these initial findings, the project progressed to detailed three-dimensional finite element modelling for the subset of bridges that did not satisfy vertical acceleration limits during the simplified beam analysis. Using safety thresholds of 5 m/s^2 for ballasted tracks and 7.5 m/s^2 for slab tracks, ninety-four bridges were identified as requiring more sophisticated evaluation. The resulting more comprehensive models allowed for advanced modal and transient time-history analyses, capturing complex three-dimensional deck deformations that simplified beam models cannot represent, thereby providing the high-fidelity data analysed in this deliverable.

The numerical analysis established in D2.2 is used for subsequent project activities. Specifically, these results will enable a direct comparison between simplified beam models and detailed models to clearly define the limitations of simplified modelling approaches. Furthermore, for the bridges that continue to exceed the established ac-

celeration thresholds, these findings will guide the evaluation of economic impacts and the design of targeted retrofitting or structural strengthening measures, ensuring the long-term safety, efficiency, and interoperability of the European railway network.

2 Data base

A preliminary evaluation of the bridge population reveals that 55 % of the structures present simply-supported (SS) spans, 15 % are continuous bridges, and 30 % are portal frames (Figure 1). Two types of culvert-like structures are analysed: closed portal frames, characterized by a continuous base slab (box culvert), and open portal frames, which consist of a top deck and lateral walls supported by independent footings. The frame configuration is characterized by a large-span deck rigidly connected to its supports. As per the main structural material, 28 % are prestressed concrete bridges, 25 % steel-concrete composite structures, and 4 % are steel bridges, with the remainder consisting of reinforced concrete. The most prevalent deck configuration is the full slab (40 %), followed by filler beam (22 %), beam (9 %), and U-girder decks (7.5 %). Furthermore, the sample is approximately evenly distributed between single-track and double-track configurations. Finally, 89 % of the analysed cases feature ballasted tracks.

2.1 Numerical analysis

All the bridges within the database are analyzed with either planar analytical or three-dimensional (3D) finite element (FE) numerical models. In the first case only longitudinal bending is accounted for and in the second, the 3D deformation of the bridge decks is considered. The computational procedure involves determining the structural dynamic response under several train passages in a wide speed interval ranging from 144 to 450 km/h in 1.8 km/h intervals, specifically assessing maximum vertical accelerations and displacements at the platform.

Damping values are assigned according to EN 1991-2 [15], based on the bridge material and span length.

The selection of the trains for the dynamic analyses is based on the concept of dynamic train signature, introduced by European Rail Research Institute (ERRI) [16]. This is a simplified method that enables the dynamic effects of different trains at resonance and away from resonance to be compared independently of the characteristics of the bridge. For simply-supported bridges and neglecting structural damping the following expression is derived:

$$S_0(\lambda) = \max_{i=1,\dots,M} \sqrt{\left[\sum_{k=1}^i P_k \cos\left(\frac{2\pi x_k}{\lambda}\right) \right]^2 + \left[\sum_{k=1}^i P_k \sin\left(\frac{2\pi x_k}{\lambda}\right) \right]^2} \quad (1)$$

where i is the axle number and M the number of axles of the train, P_k is the axle load of the k -th axle, x_k the distance between the k -th axle and the first one, and λ the wavelength of excitation defined as v/n_0 , where v is the train speed and n_0 the fundamental frequency of the bridge.

First, over 350 HS European real trains are selected from a passenger train database also developed within the InBridge4EU project as starting point, and their dynamic signatures are computed. Then, the signatures of the 10 High-Speed load model-A

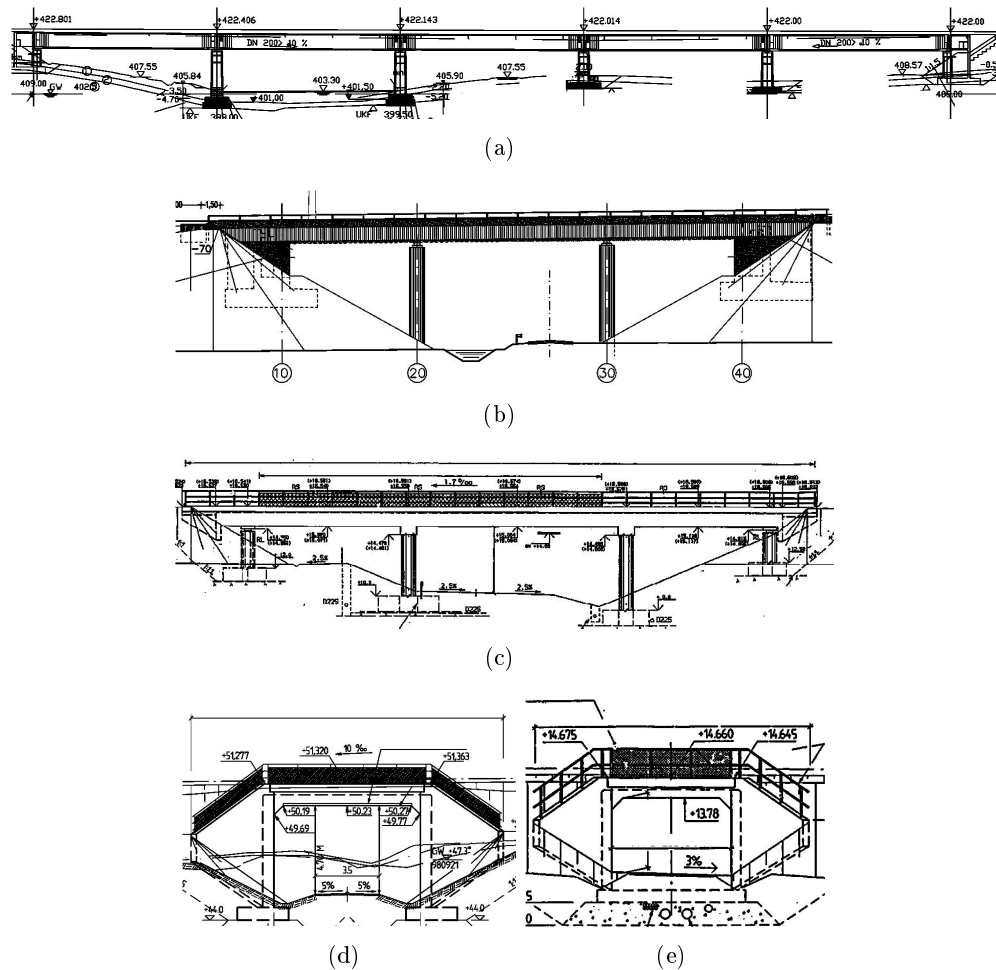


Figure 1: Bridges longitudinal configurations: (a) simply-supported, (b) continuous, (c) frame, (d) open portal frame and (e) closed portal frame.

(HSLM-A) from Eurocode (EC) trains [5] are superimposed. It is a known fact that these artificial trains are no longer an envelope of the dynamic effects of some of the newest trains in Europe. Therefore, for the analyses performed herein a (not too large) subset of these real trains is selected with the aim of covering the HSLM-A gaps in the signature wavelengths.

Figure 3 shows the real HS trains and the HSLM-A signatures. By an optimization process based on minimizing the normalized error relative to the HSLM-A signature 16 real trains are finally selected. These 16 trains and the 10 HSLM-A trains constitute an envelope of the dynamic signatures of the starting 350 trains in the database, as shown in the figure.

Given the number of bridges and the time-intensive nature of the dynamic simulations, the methodology prioritizes the use for the most computationally efficient models possible. Initially, three fundamental beam configurations are employed across all typologies of the 509 bridges included in the study: simply supported, continuous, and fixed-end beams. These Bernoulli-Euler analytical models are used to evaluate the serviceability limit state in a first stage in order to roughly differentiate the bridges that exhibit inadequate dynamic behaviour. Accordingly, only the longitudinal bending re-

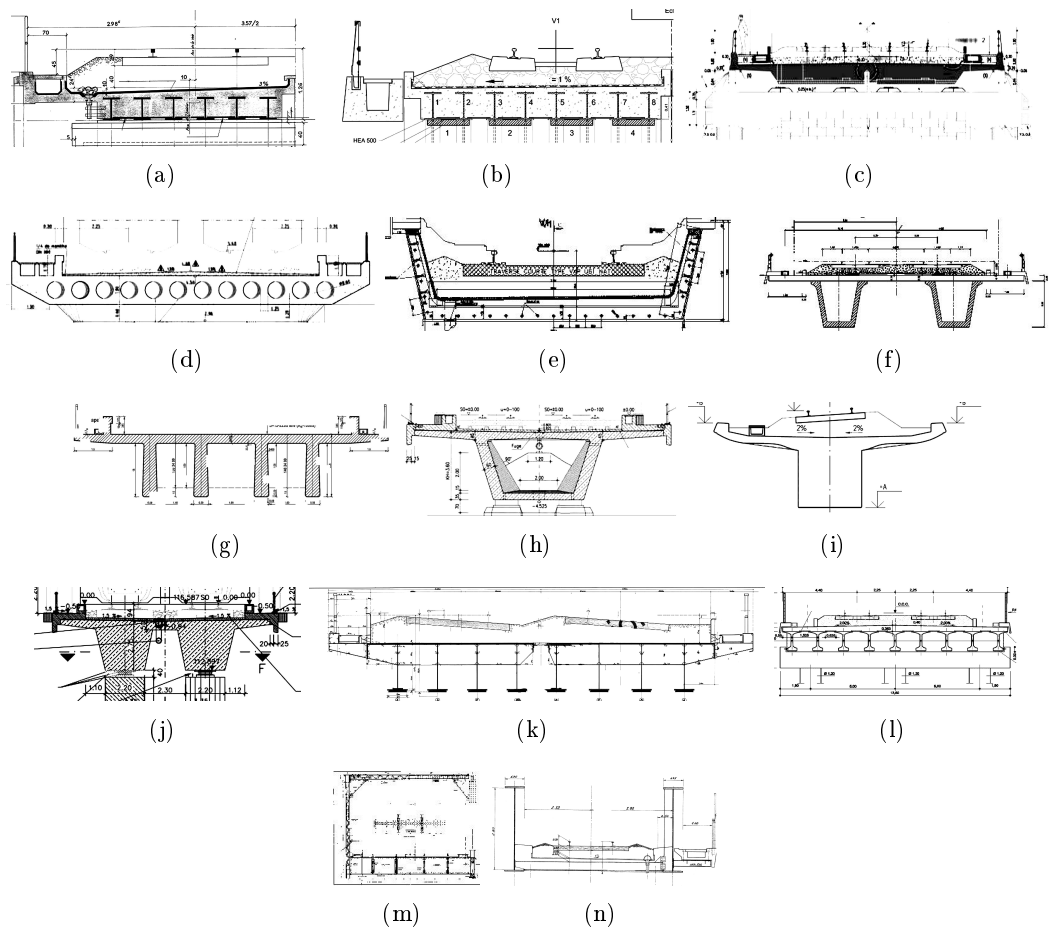


Figure 2: Schematic representation of the most common deck types analyzed: (a) filler beam, (b) reinforced filler beam, (c) full slab, (d) hollow slab, (e) U-type, (f) U-girder, (g) T-beam, (h) box-girder, (i) beam, (j) slab beam, (k) grillage, (l) multi-girder, (m) truss, and (n) half-through bridges.

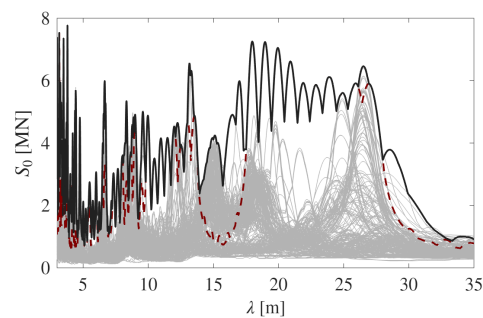


Figure 3: (Grey) Database HS trains, (dashed red) HSLM-A envelope and (black) envelope signatures of 10 HSLM-A + 16 trains final selection.

sponse of the structures is accounted for. Also, factors such as the deck obliqueness, supports flexibility and interaction with either the soil, the vehicle or the track are dis-

regarded. It should be stressed that despite these simplifications the adopted models and analysis procedure are aligned with the standards recommendations.

Although simple models are used, computational time can be high, mainly when long bridges are studied. For this reason, a novel analytical method based on time-discrete analysis using sinc interpolation has been developed by the authors and applied in this work to compute the dynamic response of multi-span, non-uniform railway beams with general boundary conditions under moving loads. This method allows for modal superposition by transforming the convolution integrals of normal coordinates into time integrals of trigonometric functions. The methodology is implemented in the SINC-Dyn function package [17]. Alternatively, CALDINTAV [18], a user-friendly software application designed for the dynamic analysis of railway bridges subjected to high-speed train loadings, is used.

In a second stage, 3D FE models are developed [19, 20] for the 94 bridges that failed to meet the acceleration compliance criteria during the initial beam-based analysis (maximum acceleration of 5 m/s^2 and 7.5 m/s^2 in ballasted and non-ballasted tracks, respectively). The approach has been to implement detailed models for all non-compliant bridges to perform modal analyses and extract the lowest natural frequencies and mode shapes along the load paths (rails) and at the response post-processing points. These modal parameters are subsequently utilized in time-step calculation (TSC) dynamic analyses to obtain the bridges dynamic response under the train models previously mentioned employing the SINC-Dyn toolbox [17]. In all instances, only the superstructure is modelled; the stiffness and deformation of piers, abutments, and foundations are disregarded. The particular shell/solid modelling approach varies according to the deck typology. The linear mass of each deck, as specified in the database, is strictly maintained. Furthermore, non-structural masses (comprising ballast, sleepers, rails, sidewalks, etc.) are uniformly distributed over the platform area. These masses are adjusted to ensure that the 3D FE models and their corresponding analytical beam counterparts maintain identical total mass properties.

2.2 Limitations of Beam-Model Analysis

The availability of dynamic response results for a large number of railway bridges subjected to the passage of the 26 trains of study, obtained using both beam models —where only the longitudinal bending behaviour of the bridge is represented— and detailed three-dimensional finite-element models, has enabled a comprehensive comparative assessment of the limitations associated with the classical beam-model analysis. The study was conducted on a subset of 84 railway bridges from WP2 original database, including simply supported bridges and continuous bridges with two, three and four spans, covering the following deck typologies: U-type, U-girder, T-beam, full slab, hollow slab, half-through, box, beam and filler beam. For the two-dimensional analyses, the response was evaluated at three sections per span, whereas 25 post-processing points per span were considered for the three-dimensional models. The responses were compared as the upper limit of the analysed speed range was progressively increased from 150 to 450 km/h (see Figure 4 and Figure 5).

The comparison performed for the 84 bridges, all of which exhibited significant dynamic behaviour and were predicted by the simplified models to experience accelerations above the admissible limit, allowed four distinct situations to be identified (see Figure 6): (a) bridges for which the 3D prediction matches the 2D prediction in both amplitude and speed; (b) bridges for which the 3D models predict the same acceleration amplitude

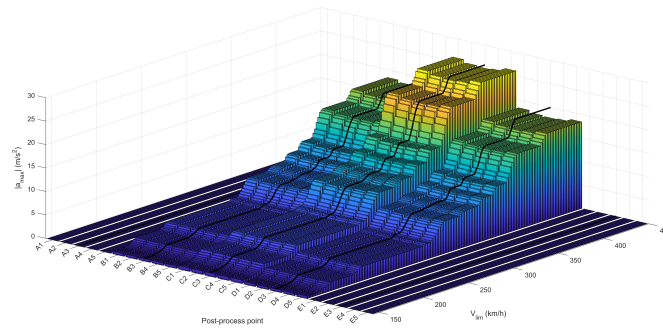


Figure 4: Envelope of bridge deck accelerations for the 26 trains of study as a function of the upper speed limit. Black lines: predictions obtained every quarter-span using the beam model. Surface plot: predictions from the detailed 3D model at each post-processing point.

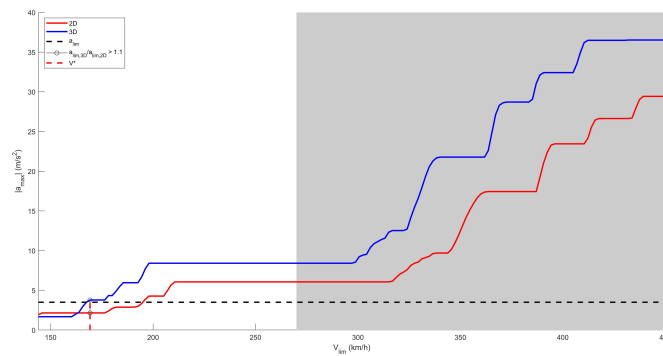


Figure 5: Comparison envelope between the 2D and 3D analyses considering all post-processing points as a function of the upper speed limit. Threshold speed beyond which the beam-model analysis becomes non-conservative.

as the beam model but with a shift in the threshold speed owing to differences in the predicted fundamental frequency; (c) bridges for which the beam model is overly conservative; and (d) bridges for which the beam model becomes non-conservative depending on the speed range considered.

The bridge deck typologies associated with these four situations were analysed, leading to the following conclusions:

- The suitability of beam models for accurately predicting the vertical acceleration of railway bridge decks depends primarily on the deck typology and the skew angle. In contrast, span length and deck width, which is determined by the number of tracks, have a significantly lower influence.
- For both simply supported and continuous bridges (up to four spans) with slab or pseudo-slab deck typologies, beam models are not suitable when the skew angle exceeds 30° , as they significantly overestimate the structural response. The skew angle should be understood as the angle between the horizontal axis perpendicular to the track and the deck support line. For skew angles between 15° and 30° , the increase in the fundamental frequency associated with this geometric parameter

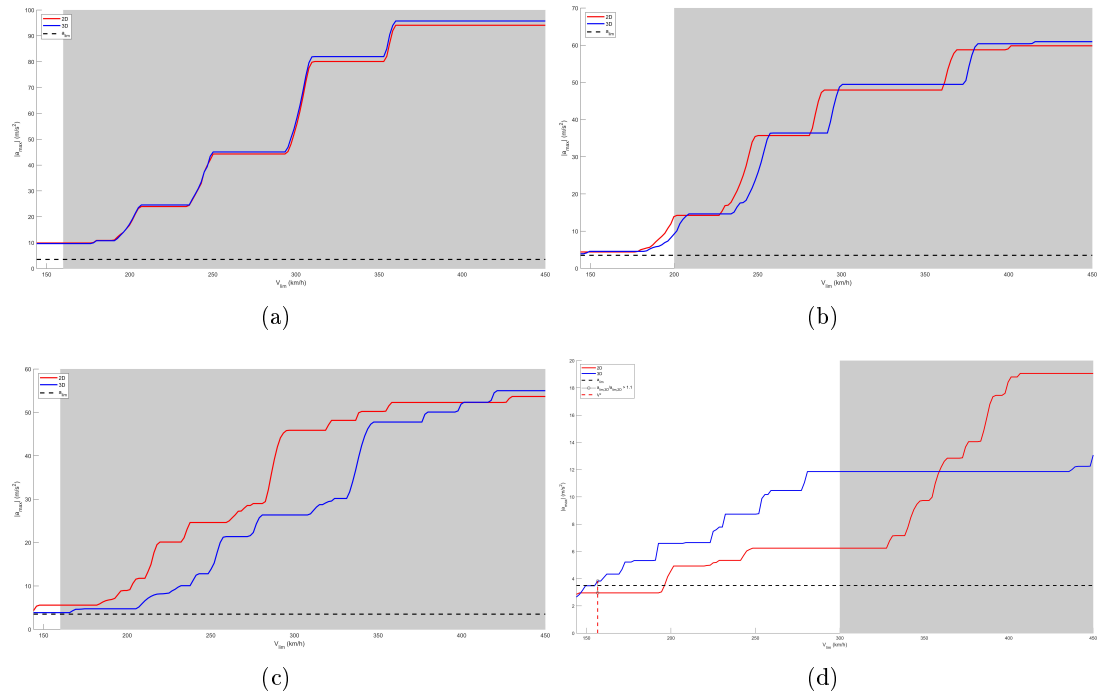


Figure 6: Classification of bridge behaviour in the comparison between beam- model analysis and detailed 3D modelling.

must be taken into account. Once this effect is considered, beam models can be regarded as suitable. For skew angles below 15° , beam models are considered suitable without further correction.

- The use of detailed models capable of reproducing the three-dimensional deformation of the deck is recommended for bridges with U-girder, half-through, and steel beam decks with transverse stiffeners. In these cases, the agreement between the predictions obtained from beam models and those from three-dimensional models is limited, even for non-skewed bridges.
- No firm conclusions could be drawn regarding U-type and T-beam deck typologies due to the limited number of specimens available within the selected bridge population.

It should be noted that the bridge catalogue considered in this analysis consists of relatively short- and light-span bridges exhibiting significant dynamic behaviour, concentrated within the shaded region shown in Figure 7.

3 Statistical analysis

This section presents a statistical analysis based on the computed results. For bridges where vibration levels remained below the established threshold using simplified models, those initial results were retained (415 bridges). Conversely, in cases requiring more complex structural modelling, the computations derived from these advanced models were utilized for the subsequent analysis (94 bridges).

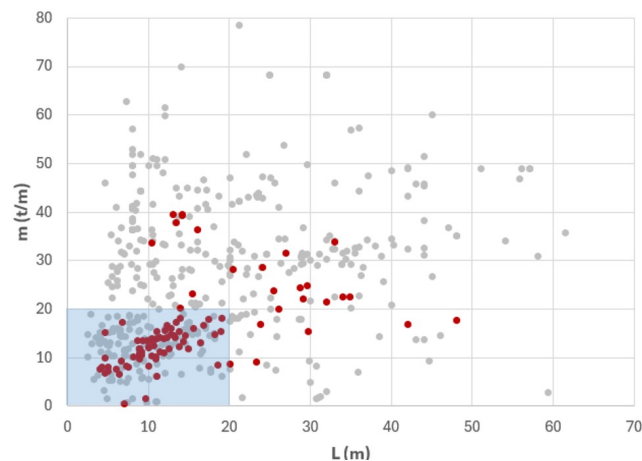


Figure 7: Linear mass versus maximum span length for the bridge catalogue (grey). Subset of bridges used for the beam-model limitations study (red).

3.1 Data analysis, structural parameters and parameter interdependence

In general, portal frame bridges are predominant for short spans ($L < 10$ m). Owing to the short span and the rigid connection between the deck (typically a full slab) and the abutments, these structures exhibit significantly higher natural frequencies than simply supported or continuous bridges. In addition, they present higher bending stiffness than other typologies. In the long-span range ($L > 30$ m), both simply supported and continuous bridges are commonly found. Although the natural frequencies are relatively low due to the large spans, the linear mass is generally high (over 20000 kg/m). This is associated with the structural typologies typical of this span range, such as U-girder or box-girder bridges. The stiffness requirements for these spans result in very massive structures, which are unlikely to experience significant accelerations, even under resonance conditions. The intermediate span range ($10 \text{ m} < L < 30 \text{ m}$) concentrates a large number of bridges with relatively low linear masses and natural frequencies. Many of these bridges are simply supported, with slab bridges and filler beam decks being predominant. Within this span range, continuous bridges generally present higher linear masses. It is within this span range that greater dynamic problems can be expected.

Figure 9 illustrates the variation in the mass per unit length, fundamental frequency, bending stiffness index, damping ratio[5] and maximum predicted acceleration across different bridge configurations. In these plots, the horizontal blue line represents the median (50th percentile). The blue box is defined by the 25th and 75th percentiles, while the black whiskers indicate the dispersion outside this central 50% range. Circles denote outliers. While most configurations maintain a similar median mass, a significant outlier is observed in the portal frame open configuration; this specific case corresponds to an exceptional bridge carrying seven tracks. The fundamental frequencies for the portal frame type configurations are the highest and exhibit the greatest dispersion. Certain atypical values exceeding 100 Hz result from the numerical modelling assumptions, specifically the consideration of fully fixed boundary conditions. Conversely, simply-supported and continuous bridges show lower, more concentrated fundamental frequency ranges. The bending stiffness index shown in Figure 9(c) is

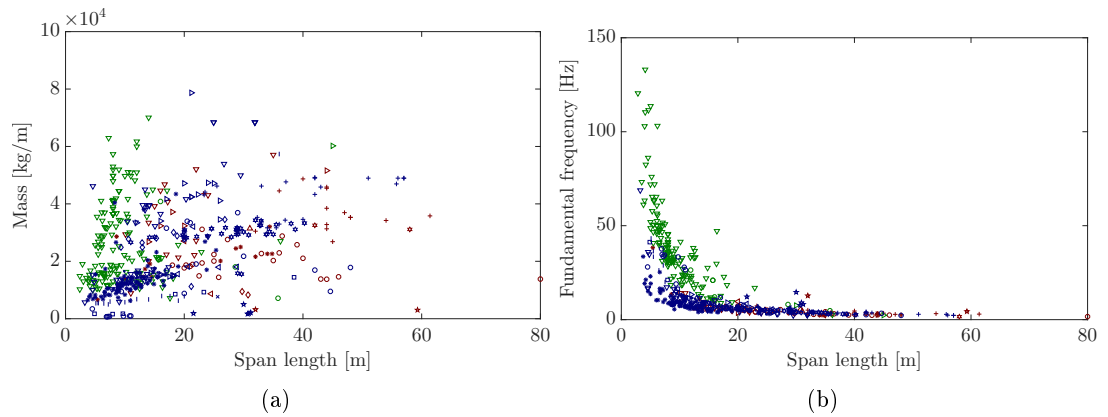


Figure 8: Distribution of (a) mass per unit length and (b) fundamental frequency versus maximum span length: (red) continuous, (green) frame, portal frame open and portal frame closed, and (blue) simply-supported bridge configurations. Markers represent deck types: $\circ$: beam, $+$: box, $*$: filler beam, $\cdot$: filler beam reinforced, $\times$: grillage, $\square$: half-through, $\diamond$: multi-girder, $\triangle$: beam slab, ∇ : full slab, $\triangleright$: hollow slab, $\triangleleft$: T beam, $\star$: truss, $\ast$: U-girder deck, $|$: U-type deck.

defined as EI/L^2 , where EI denotes the bending rigidity (the product of Young's modulus and the area moment of inertia) and L represents the maximum span length. This parameter is directly related to the ratio of span length to the maximum displacement produced by a unit load. Portal frame bridges exhibit the highest median values, which are associated with the typical structural typologies adopted, most commonly full slabs. Finally, simply-supported bridges exhibit numerous outliers with accelerations exceeding 10 m/s^2 , indicating that this configuration is more susceptible to exhibiting substantial vibration levels associated to resonance phenomena.

Figures 9(d) and 10 illustrate the influence of the damping ratios. These values are derived from EN 1990-Annex A2 [4] based on specific bridge properties; consequently, damping is treated as a dependent variable in the statistical analysis.

Figure 11 shows the structural and dynamic response of the bridge dataset, discretized into ten fundamental frequency intervals ranging from (1, 5] Hz to (30, 40] Hz. There is a discernible, although non-linear, trend where mass tends to stabilize or slightly decrease as the fundamental frequency increases. This is consistent with structural dynamics, as higher frequencies often correspond to shorter spans or stiffer cross-sections with lower total mass per length. Lower fundamental frequency bridges exhibit substantially higher bending stiffness index values. Low frequencies ($f_1 < 5 \text{ Hz}$) are generally associated with high mass and high stiffness. This is typically the case for long-span bridges ($L > 30 \text{ m}$), which, due to stiffness requirements, especially in HS lines, are usually constructed with stiff and massive sections; consequently, acceleration levels are generally not expected to be significant. At the other end of the spectrum, in the high-frequency range ($f_1 > 18 \text{ Hz}$), although the linear mass may be relatively low, acceleration levels are also not excessive because these bridges usually correspond to structural types such as frame bridges with very high flexural stiffness. The most critical situation occurs in the intermediate frequency range (between 5 and 12 Hz), where both stiffness and mass tend to be low. This is typical of simply supported slab and filler beam bridges with short-to-medium spans. On average, the bridges that exhibit

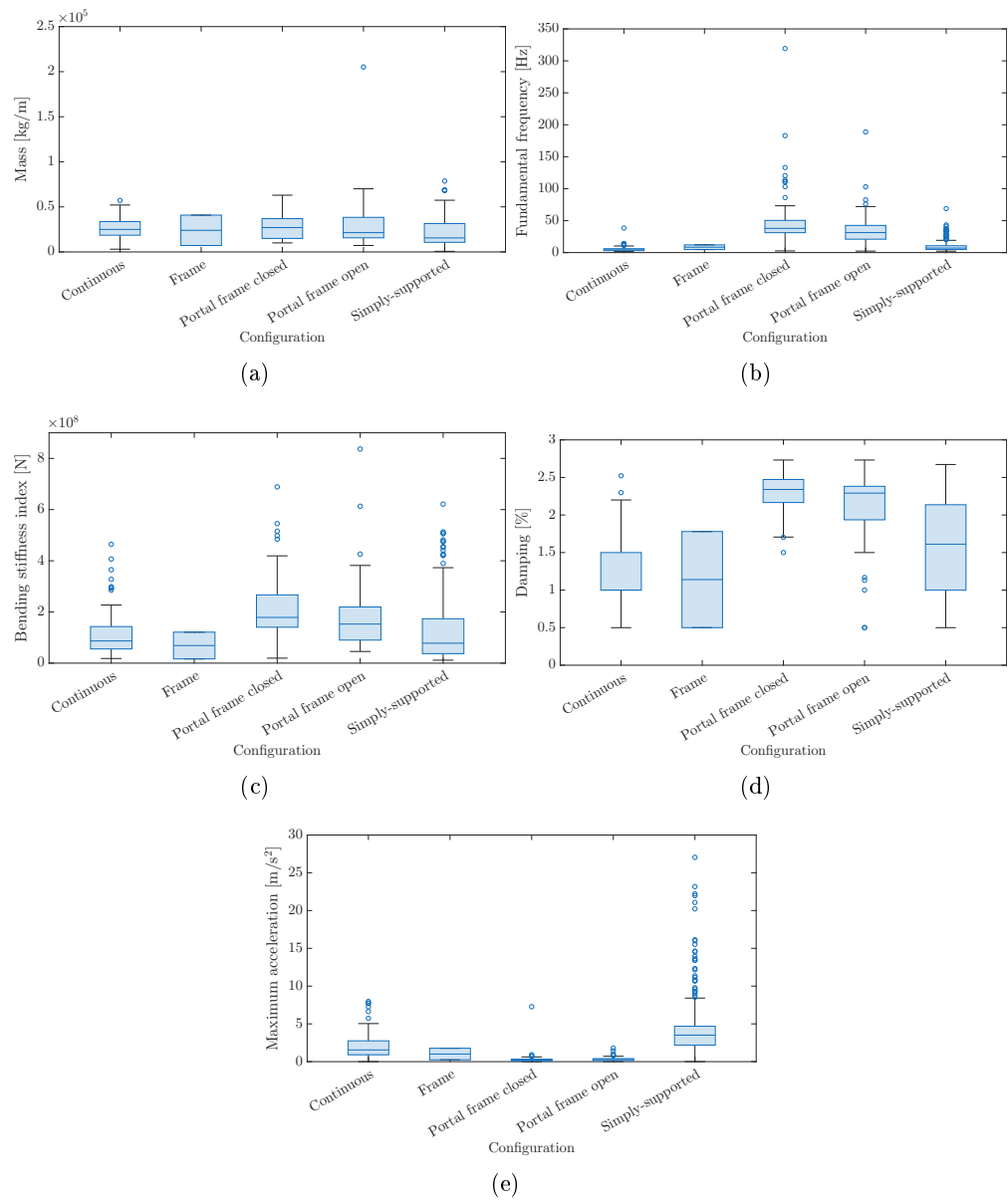


Figure 9: Statistical distribution of structural and dynamic parameters grouped by bridge longitudinal configuration: (a) linear mass, (b) fundamental frequency, (c) bending stiffness index, (d) damping[4], and (e) maximum vertical acceleration. Boxes represent the interquartile range, the horizontal line denotes the median, and circles indicate statistical outliers.

the highest acceleration levels fall within this frequency range and are characterized by low linear masses and low bending stiffness.

The interdependence between structural parameters and the resulting maximum acceleration is now evaluated through a Pearson correlation analysis. To ensure the reliability of the observed trends, a significance test is performed, masking all correlations with a p -value exceeding 0.05 (Figure 12). A statistically significant negative correlation is observed between the fundamental frequency and the maximum acceleration. This confirms that higher structural stiffness -and the consequent shift to higher frequency

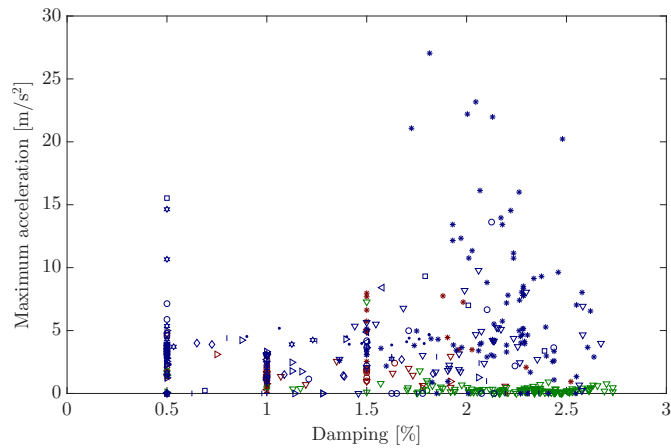


Figure 10: Distribution of maximum vertical acceleration versus damping values: (red) continuous, (green) frame, portal frame open and portal frame closed, and (blue) simply-supported bridge configurations. Markers represent deck configurations: $\circ$: beam, $+$: box, $*$: filler beam, $\cdot$: filler beam reinforced, $\times$: grillage, $\square$: half-through, $\diamond$: multi-girder, $\triangle$: slab beam, ∇ : slab full, $\triangleright$: slab hollow, $\triangleleft$: T beam, $\star$: truss, $\times$: U-girder deck, $|$: U-type deck.

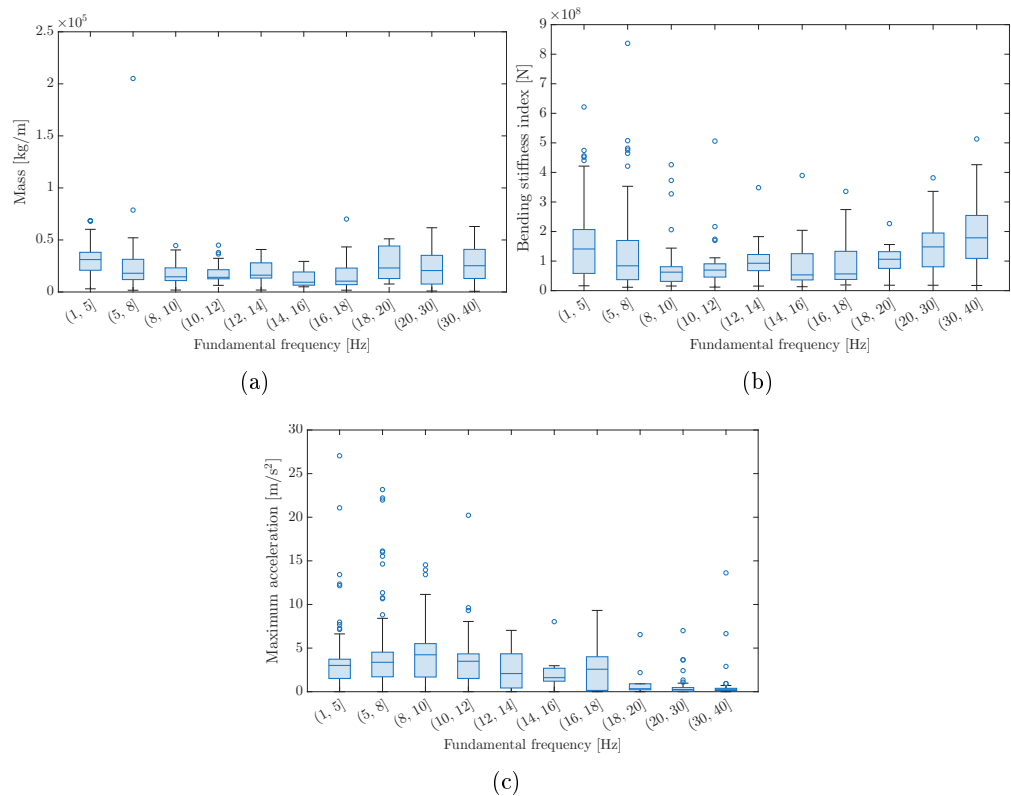


Figure 11: Boxplot analysis of bridge parameters discretized by fundamental frequency intervals: (a) distribution of mass, (b) bending stiffness, and (c) maximum acceleration.

ranges- effectively reduces the susceptibility to resonant vibration under train passages. A strong, significant positive correlation exists between mass and bending stiffness in-

Figure 10 is a heatmap showing the correlation coefficients between the input variables and the output variables. The input variables are Configuration, Deck, Material, Mass, Skewness, V_{max} , Fundamental frequency, Maximum acceleration, and Bending stiffness index. The output variables are the same as the input variables. The color scale ranges from -1 (blue) to 1 (red). The diagonal elements are all 1 (red). The off-diagonal elements show varying degrees of correlation, with some positive and some negative.

The analysis reveals that while fundamental frequency remains the universal driver for all typologies, the impact of structural mass and stiffness is highly configuration-dependent, with simply-supported spans showing the greatest sensitivity to resonance-induced accelerations. Next, a category-specific correlation analysis is performed to isolate the mechanical drivers of peak acceleration for each bridge longitudinal configuration, revealing that the influence of structural parameters is highly dependent on this factor.

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as high-priority candidates for advanced monitoring in rail networks. The box deck typologies exhibit the highest median values and broadest dispersion for the bending stiffness index. In contrast, filler beam and half-through decks show significantly lower and more concentrated stiffness, which correlates with their highest accelerations.

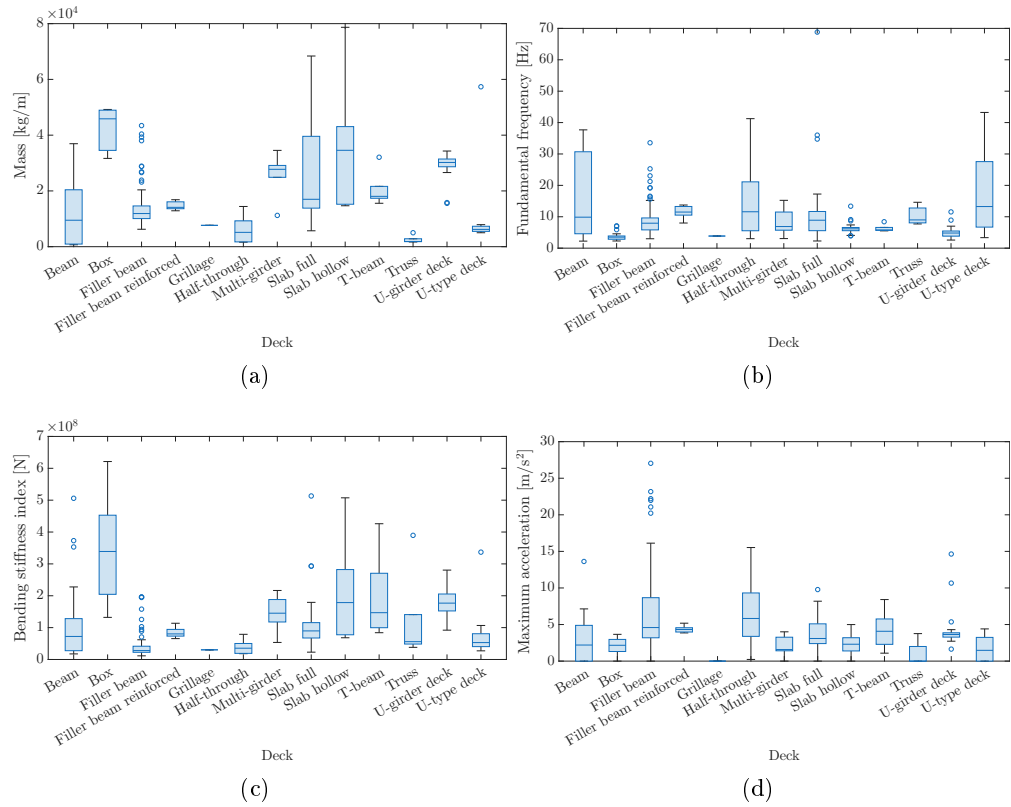


Figure 13: Comparative analysis of structural parameters for the simply-supported bridge population categorized by deck typology: (a) linear mass, (b) fundamental frequency, (c) bending stiffness index, and (d) maximum vertical acceleration.

The distribution of structural parameters into fundamental frequency bins (Figure 14) shows a clear inverse relationship between bending stiffness and fundamental frequency, where the highest EI/L^2 values are associated with the lowest frequencies, typical of long-span bridges (see Figure 8). While mass generally decreases as frequency increases, significant outliers are noted in the 8–12 Hz range representing specific short-span deck typologies. Peak vertical accelerations exhibit a drastic increase in both magnitude and dispersion within the lower frequency spectrum, specifically between 1 and 10 Hz. Above the 16–18 Hz threshold, the median acceleration stabilizes significantly below 5 m/s², suggesting that stiffer structures are inherently less susceptible to resonant excitation from train passages.

The correlation matrix for simply-supported bridges (Figure 15) reveals that geometric parameters such as skewness are less influential than other factors; instead, the dynamic response is governed by a significant interaction between mass, bending stiffness index, material, deck type, operational velocity, and fundamental frequency.

The influence of the deck type on the dynamic behaviour of continuous bridges shows a more stable response compared to the previous configuration. Figure 16 shows that, unlike SS spans, continuous bridges maintain accelerations consistently below 5 m/s²,

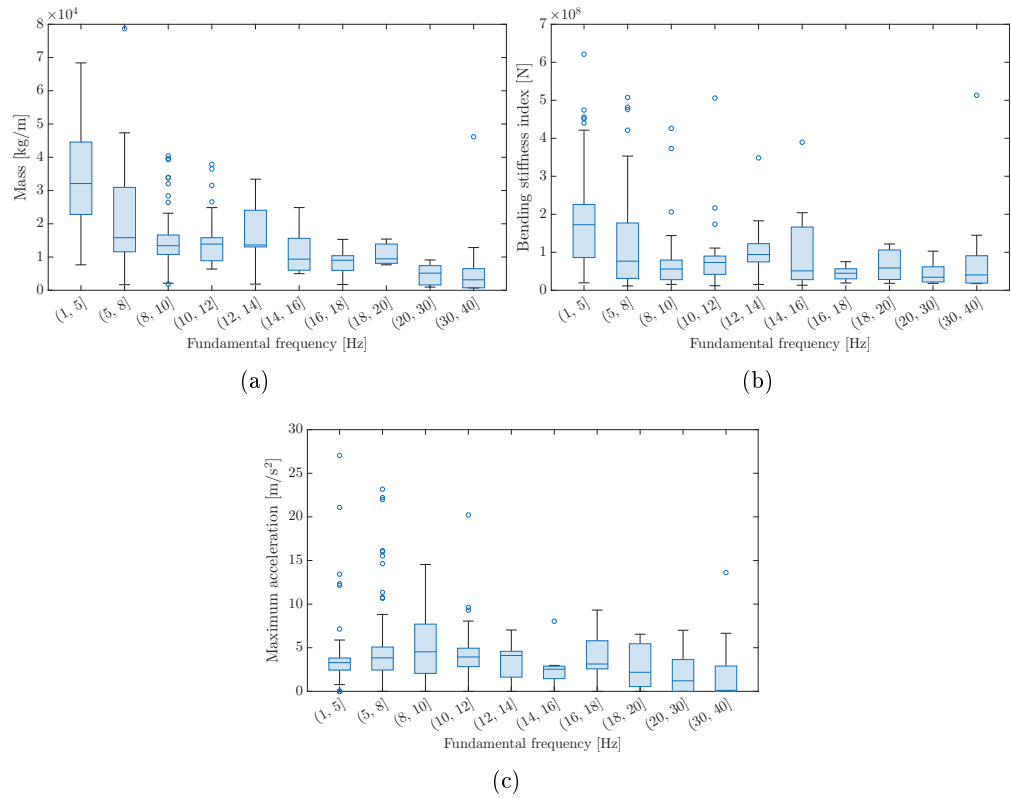


Figure 14: Distribution of bridge mass, peak vertical acceleration, and bending stiffness across fundamental frequency bins for the simply-supported bridge population.

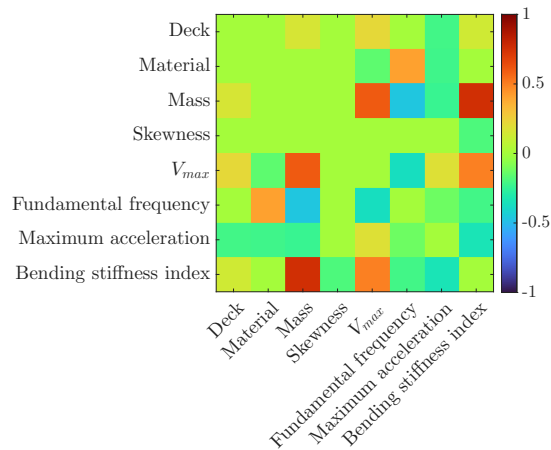


Figure 15: Full Pearson correlation matrix (R) for the simply-supported bridge population retaining only coefficients with a p -value < 0.05 . Non-significant relationships are masked (set to zero) to highlight robust structural dependencies.

suggesting that structural redundancy effectively reduces the acceleration levels.

The relationship between frequency and structural parameters confirms the general robustness of continuous bridges when compared to SS cases (Figure 17). The highest mass values are associated with the lowest frequency bin (1 – 5 Hz). As the frequency increases, the mass stabilizes at lower values, indicating a transition toward lighter

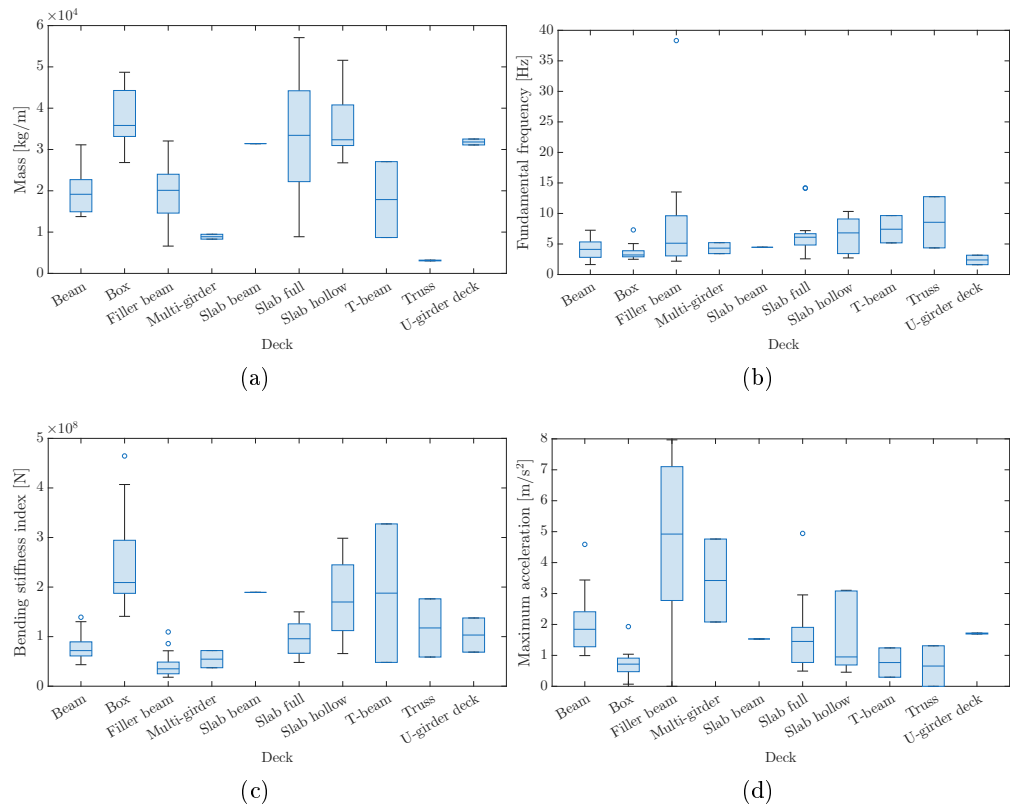


Figure 16: Comparative analysis of structural parameters for the continuous bridge population categorized by deck typology: (a) linear mass, (b) fundamental frequency, (c) bending stiffness index, and (d) maximum vertical acceleration.

structural typologies. The bending stiffness index remains fairly constant across the full frequency range. Figure 17(c) highlights that continuous bridges do not experience the remarkable acceleration levels previously showed in flexible simply-supported spans. Even at low frequencies, the maximum acceleration remains controlled, demonstrating that continuity mitigates resonance risks.

The significant correlation matrix for continuous bridges (Figure 18) shows a significant departure from the behaviour of simply-supported spans, most notably the statistical decoupling between fundamental frequency and maximum acceleration. While in simply-supported bridges the fundamental frequency is the main predictor of the dynamic risk, in continuous configurations, it leaves to be a reliable indicator. This phenomenon can be attributed to (i) in continuous bridges, the hyperstatic system allows vibration energy to be distributed across the full structure rather than concentrating at one span, (ii) the continuity of the deck which provides higher internal damping and more complex load paths, effectively mitigate resonance amplifications (this is evidenced in Figure 17(c), where accelerations remain consistently low even for bridges in the low-frequency range), and (iii) low frequency modes with closely spaced natural frequencies contribute to the response which is not governed by the fundamental mode unlike in the simply-supported case. Consequently, for HS railway bridge design, continuous bridges constitute a much more robust alternative.

Frame bridges group -including frames and both open and closed portal frames- represent the most rigid typology in the dataset. It should be noted that only beam-type

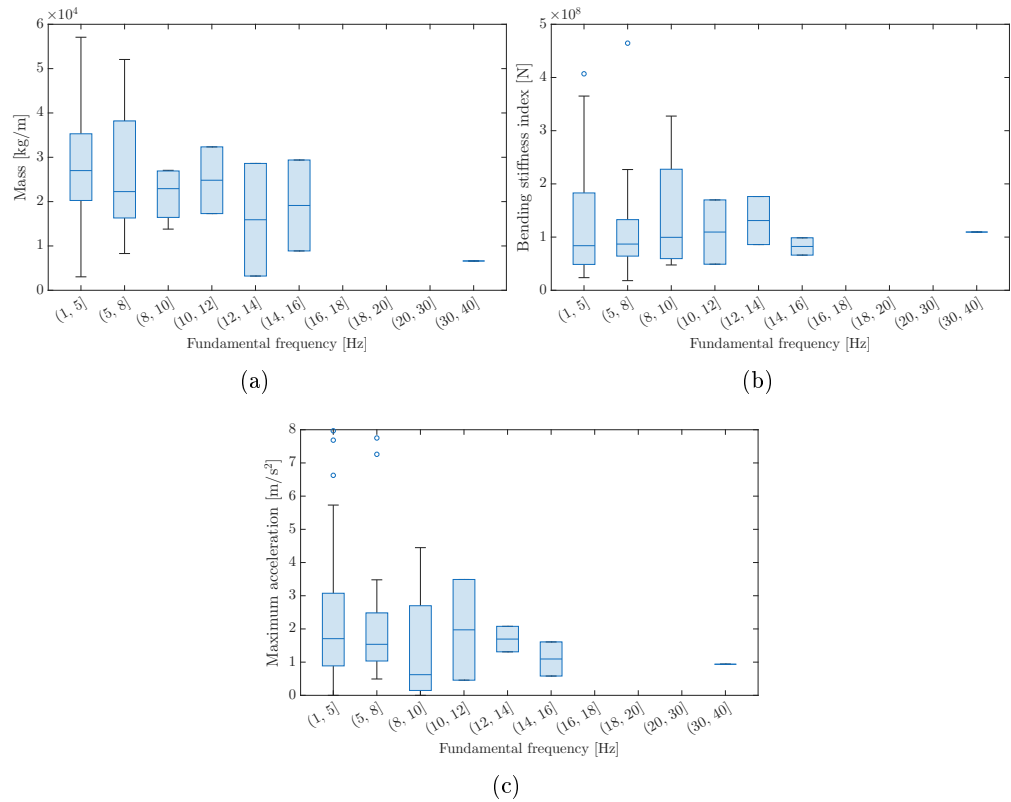


Figure 17: Distribution of bridge mass, peak vertical acceleration, and bending stiffness across fundamental frequency bins for the continuous bridge population.

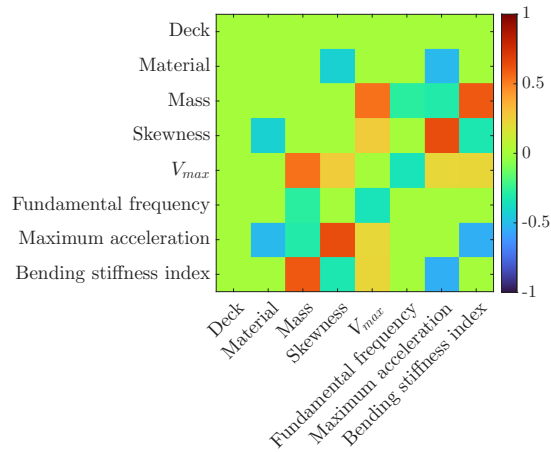


Figure 18: Full Pearson correlation matrix (R) for the continuous bridge population retaining only coefficients with a p -value < 0.05 . Non-significant relationships are masked (set to zero) to highlight robust structural dependencies.

analyses have been performed on frames due to their very low dynamic response. Figure 19 shows that these structures are characterized by low vertical accelerations, with almost all cases falling below 2 m/s^2 unlike in simply-supported or continuous bridges. The distribution of structural parameters across frequency bins (Figure 20) shows that linear mass and bending stiffness remain constant up to 10 Hz. For frequencies above this

threshold, the bending stiffness index increases significantly as span lengths decrease.

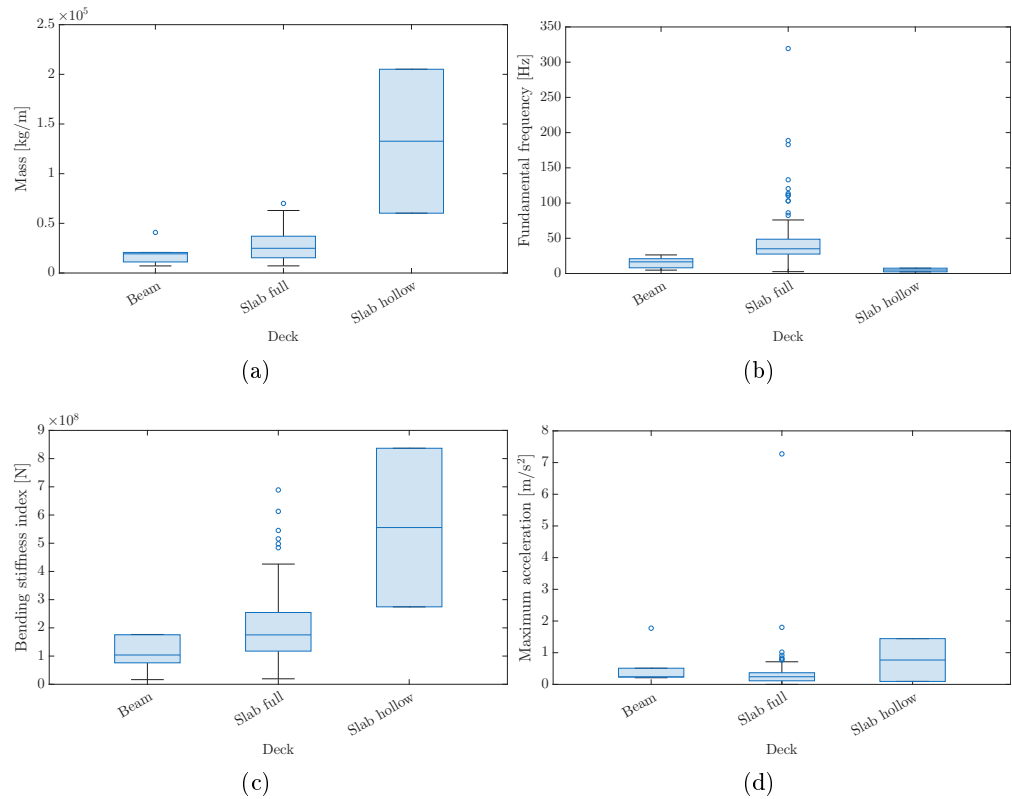


Figure 19: Comparative analysis of structural parameters for the frame bridge population categorized by deck typology: (a) linear mass, (b) fundamental frequency, (c) bending stiffness index, and (d) maximum vertical acceleration.

The significant correlation matrix for the frame population (Figure 21) reveals a highly decoupled dynamic system. The statistical analysis shows that maximum acceleration is almost independent of most structural and geometric parameters, including deck type and fundamental frequency, and that there is a strong coupling between bending stiffness, mass, and fundamental frequency, which is expected in monolithic reinforced concrete structures. Frame bridges provide enough rigidity and damping, with a relevant participation of soil radiation, to prevent significant energy transfer from the train to the structure, resulting in a low acceleration response.

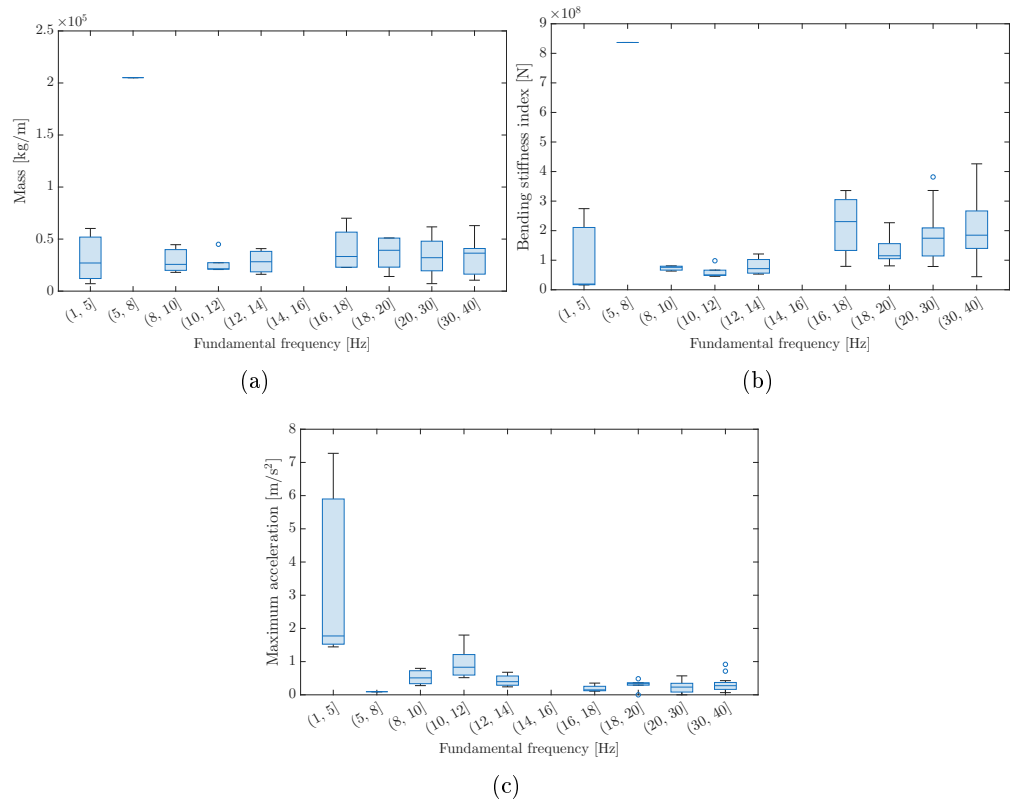


Figure 20: Distribution of bridge mass, peak vertical acceleration, and bending stiffness across fundamental frequency bins for the frame bridge population.

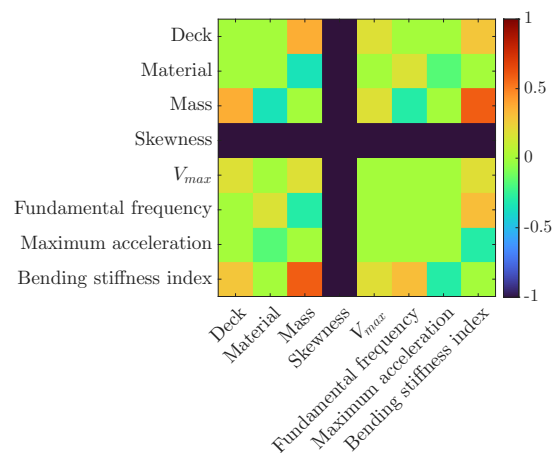


Figure 21: Full Pearson correlation matrix (R) for the frame bridge population, which contains no skew bridges, retaining only coefficients with a p -value < 0.05 . Non-significant relationships are masked (set to zero) to highlight robust structural dependencies.

3.2 Stepwise linear regression model

Following the data analysis, a Stepwise Linear Regression (SLR) model is implemented to quantify the influence of the identified structural parameters. The model follows an approach based on the Bayesian Information Criterion (BIC), ensuring that only statistically significant variables are retained to prevent overfitting. The performance of the SLR is initially evaluated using the entire dataset (Figure 22(a, b)). This global model yields an R^2 of 0.3793 and a Root Mean Square Error ($RMSE$) of 2.8585 m/s^2 . To ensure the robustness of the baseline linear model, a diagnostic analysis is conducted using studentized residuals and Cook's distance. Observations with $|r_{stud}| > 3$ are flagged as outliers, while influential points are identified using a Cook's distance threshold of $4/n$, where n is the total number of observations. Figure 22 reveals several bridges exceeding this limit. By removing these anomalous data points (Figure 22(c, d)), the performance of the model significantly improves, reaching an R^2 of 0.4815 and the $RMSE$ reducing to 1.7692 m/s^2 . To ensure a valid comparison between the different stages of the model, the residual distributions are presented as Probability Density Functions (PDF). In these plots, the area under the histogram is normalized to unity, making the distributions comparable regardless of the number of observations in each subset. This diagnostic phase is useful to demonstrate the limitations of simplified linear approximations. High-vibration events, such as resonant peaks, act as physical non-linearities rather than statistical noise. While these high-response cases are set aside during the linear regression stage to avoid biasing the baseline linear model, they are fully retained in the subsequent machine learning phase to ensure the model successfully captures the worst-case scenarios.

The SLR is applied individually to each bridge longitudinal configuration to assess the linearity of their dynamic responses (Figures 23, 24, 25). Simply-supported bridges constitute the biggest prediction challenge, with an initial R^2 of only 0.3116. After eliminating outliers, R^2 improves to 0.3401. The relatively low accuracy and high $RMSE$ (2.2565 m/s^2) confirm that resonant situations in SS bridges are highly non-linear with the fundamental frequency to train speed ratio and make it difficult to capture the response with standard regression models. Continuous bridges show better alignment with linear assumptions. The refined model achieves an R^2 of 0.4565 and a notably lower $RMSE$ of 1.0427 m/s^2 . The more concentrated residual histogram suggests that the structural redundancy of continuous bridges leads to a more easily predictable dynamic behaviour. Frame bridges initially show an R^2 of 0, indicating that the predictors had no linear relationship with the very low accelerations of these bridges. After removing outliers, the model reached an R^2 of 0.3511 with a low $RMSE$ of 0.1723 m/s^2 . This confirms that for frames, the acceleration level is consistently low and effectively decoupled from the main structural parameters.

The linear analysis demonstrates that while the SLR is effective for identifying general trends and cleaning the dataset, it is insufficient for precise acceleration prediction, particularly for the simply-supported bridges. The persistent dispersion in the residuals indicates that complex interactions between mass, fundamental frequency, and maximum operational speed require a non-linear approach. Consequently, a random forest regression model is proposed in the following subsection to better capture these high-order dependencies.

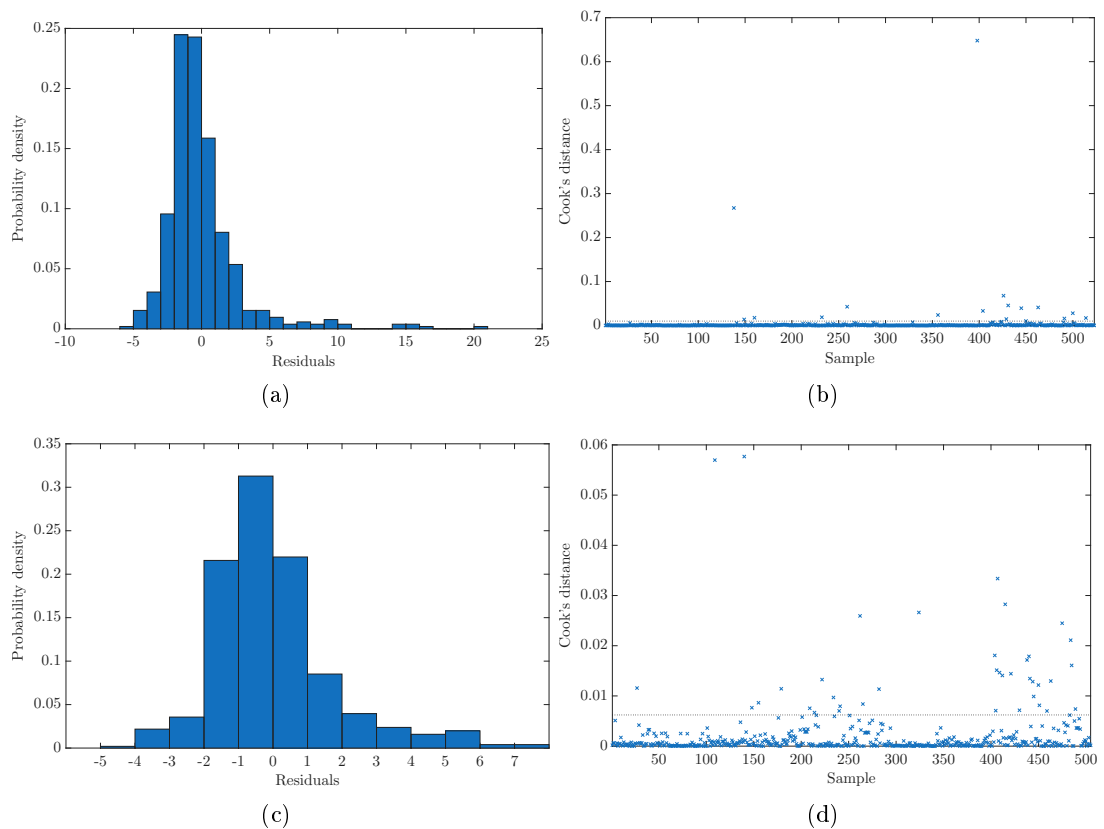


Figure 22: Diagnostic analysis of the global linear model using (a,b) the full dataset ($R^2 = 0.3793$ and $RMSE = 2.8585 \text{ m/s}^2$) and (c,d) after removing identified outliers and influential points ($R^2 = 0.4815$ and $RMSE : 1.7692 \text{ m/s}^2$): (a,c) histogram of raw residuals showing the initial error distribution and (b,d) Cook's distance plot identifying influential observations. The Cook's distance threshold is plotted by dashed lines.

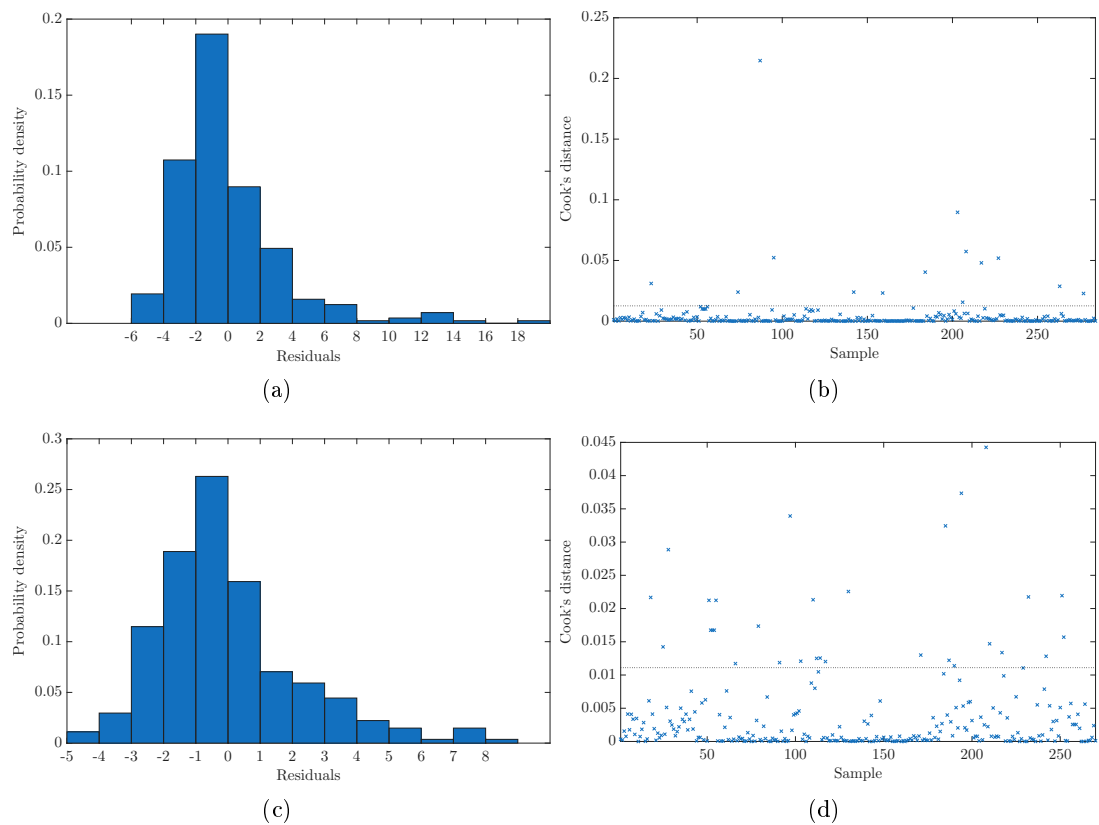


Figure 23: Diagnostic analysis of the linear model using (a,b) the full dataset for the simply-supported bridge population ($R^2 = 0.3116$ and $RMSE = 3.4388 \text{ m/s}^2$) and (c,d) after removing identified outliers and influential points ($R^2 = 0.3401$ and $RMSE = 2.2565 \text{ m/s}^2$): (a,c) histogram of raw residuals showing the initial error distribution and (b,d) Cook's distance plot identifying influential observations.

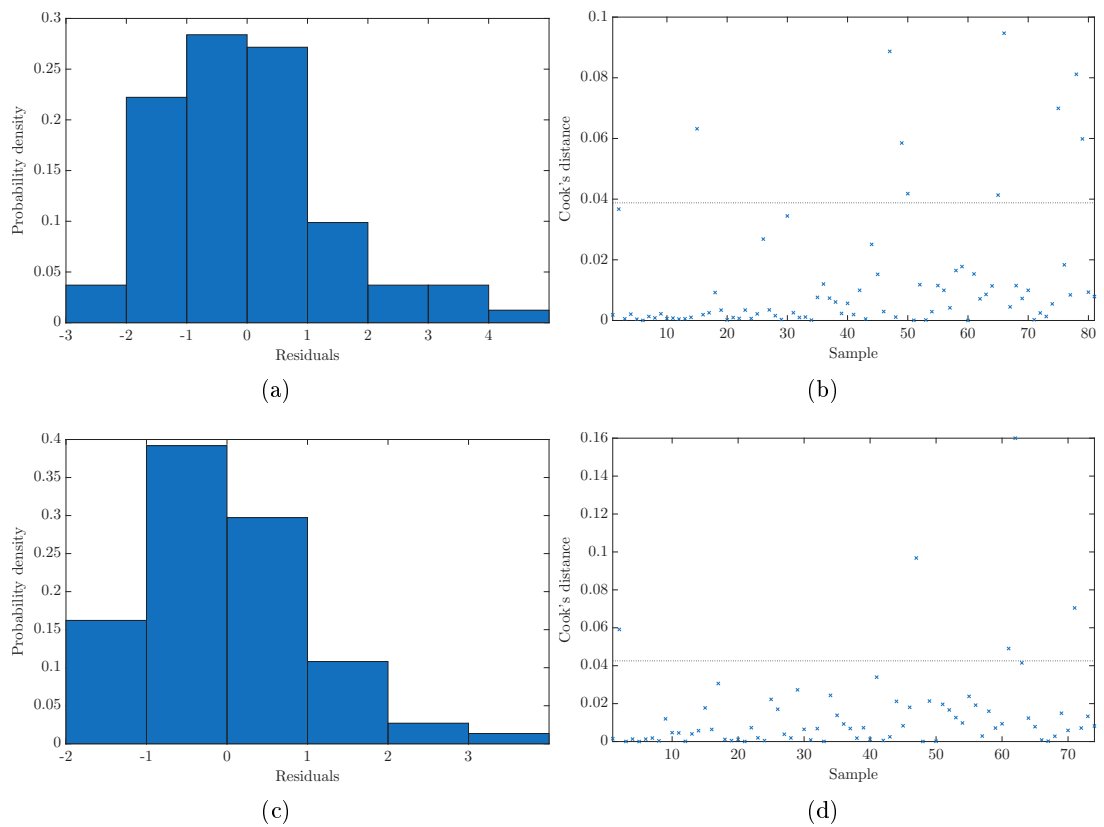


Figure 24: Diagnostic analysis of the linear model using (a,b) the full dataset for the continuous bridge population ($R^2 = 0.4228$ and $RMSE : 1.4556 \text{ m/s}^2$) and (c,d) after removing identified outliers and influential points ($R^2 = 0.4565$ and $RMSE : 1.0427 \text{ m/s}^2$): (a,c) histogram of raw residuals showing the initial error distribution and (b,d) Cook's distance plot identifying influential observations.

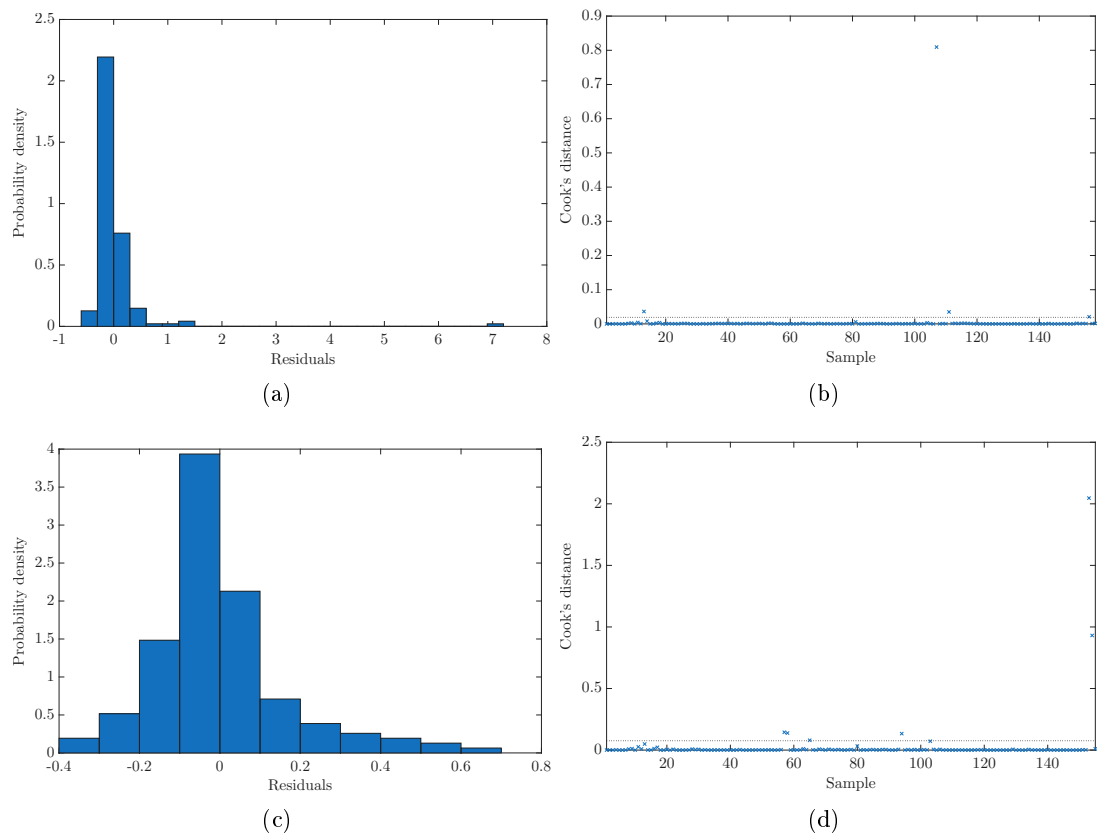


Figure 25: Diagnostic analysis of the linear model using (a,b) the full dataset for the frame bridge population ($R^2 = 0$ and $RMSE : 0.6181 \text{ m/s}^2$) and (c,d) after removing identified outliers and influential points ($R^2 = 0.3511$ and $RMSE : 0.1723 \text{ m/s}^2$): (a,c) histogram of raw residuals showing the initial error distribution and (b,d) Cook's distance plot identifying influential observations.

3.3 Random forest regression model and machine learning

To overcome the inherent non-linearity of resonance phenomena that linear models fail to capture, a Random Forest (RF) regression model is developed using a Gradient Boosting approach (LSBoost). Unlike standard black-box models, the RF is enhanced through physical feature engineering, introducing a synthetic parameter to the predictor set: Dimensionless speed $= V_{max}/(2Ln_0)$ (a non-dimensional parameter relating maximum operational velocity (V_{max}), maximum span length (L), and fundamental frequency (n_0) to quantify the characteristic speed-frequency ratio [21]).

The model is trained on 80% of the dataset, leaving 20% for independent testing. To ensure robustness and prevent overfitting, Bayesian Optimization is employed to fine-tune the hyperparameters using a 5-fold cross-validation scheme. The search space is constrained to a minimum leaf size of 5 to ensure that the model learned general physical trends rather than memorizing individual bridge samples. The RF model achieved high predictive accuracy, with an R^2 of 0.8770 and an $RMSE$ of 0.9440 m/s^2 on the unseen test set (Figure 26). As shown in Figure 26(a), most observations fall within a $\pm 20\%$ interval, indicating a strong capture of the dynamic response. The Residual Probability Density (Figure 26(b)) shows a zero-centered distribution with a high concentration of errors near zero. The fat-tailed nature of the distribution (where the theoretical normal curve is wider than the central histogram peak) highlights that while the model is highly accurate for the majority of the population, a few cases, physically complex to predict, account for the largest errors.

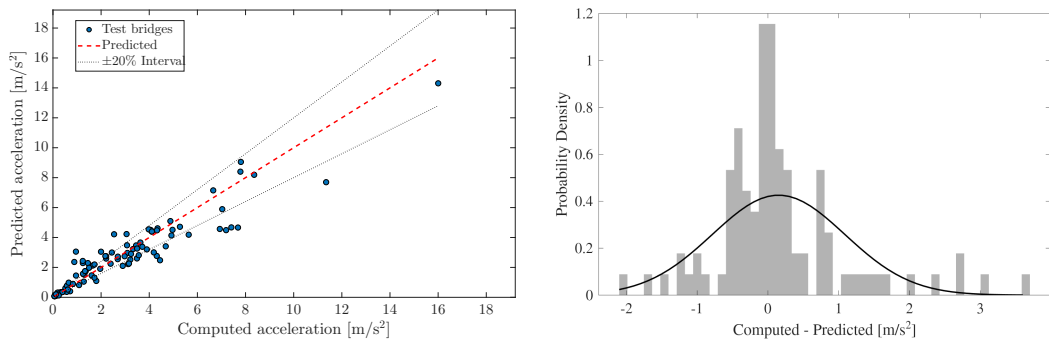


Figure 26: Performance of the optimized Random Forest model on the test set: (a) Predicted vs. computed acceleration with $\pm 20\%$ confidence intervals and (b) probability density of residuals.

To evaluate the contribution of each structural and operational parameter to the dynamic response of the bridge, a predictor importance analysis is performed (Figure 27). Furthermore, the complex non-linear relationships captured by the RF model are visualized using Partial Dependence Plots (PDP) and Individual Conditional Expectation (ICE) curves (Figure 28). The importance analysis reveals that the longitudinal configuration of the bridge is the most influential factor, followed closely by the dimensionless speed. This hierarchy indicates that the model successfully prioritizes the structural typology and the dimensionless ratio that governs the dynamic interaction regime over individual variables like mass or stiffness. The high ranking of the dimensionless speed summarizes the interaction between geometry, frequency, and speed. Mass and bending stiffness show moderate importance, acting as secondary parameters of the acceleration amplitude. Figure 28 provides a detailed view of the physics learned

by the model. Regarding the maximum operational speed, the standard PDP shows a stepped increase in acceleration, while the ICE curves reveal that for certain bridges, there are sharp jumps at specific speeds, indicating that the model has identified individual resonant situations. The dimensionless speed analysis shows a clear upward trend. The Partial Dependence Plots reveal non-linear staircase increments in acceleration as the factor approaches critical thresholds. This behaviour, observed in individual ICE curves, represents the physical transition at resonance, a phenomenon that the random forest model captures without an explicit closed-form dynamic equation. The stabilization of the median (blue line) below the mean (black line) suggests that while most bridges follow a steady trend, a subset of high-acceleration cases (outliers) increases the average. The model correctly identifies that increasing the mass or bending stiffness index of the bridge reduces the expected vertical acceleration, following an inverse non-linear relationship. Finally, for the fundamental frequency, the sensitivity is higher in the low-frequency range, where the highest accelerations are obtained, becoming almost neutral for very stiff structures.

An observation in the interpretability analysis is the presence of non-linear jumps or steps in acceleration at specific dimensionless speed values. While the dimensionless speed is calculated using the maximum operational velocity (V_{max}), resonance does not necessarily takes place at such speed. High accelerations are triggered when any of the excitation frequencies, produced by the periodic passage of axles or bogies, coincides with any of the bridge natural frequencies. Since the dataset records the maximum acceleration reached across the entire speed range $[0, V_{max}]$, a bridge will exhibit a high response if a critical resonant speed with relevant amplitude is less than or equal to V_{max} . In this context, V_{max} acts as a scanning envelope; if the resonant threshold is within the operational speed range, the peak acceleration is captured. The specific steps observed at sub-multiples of the main resonance peak suggest that the random forest model identifies multiple resonance orders (harmonics). The model ability to identify these jumps at specific factor values demonstrates that it has not only learned a statistical correlation but has effectively mapped the spectral limit states and the underlying dynamic behaviour of the structures.

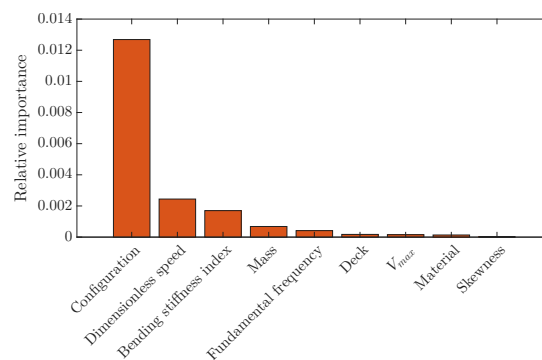


Figure 27: Relative importance of predictors for the complete bridge set.

The limitations of traditional statistical methods for predicting complex dynamic responses are evident when comparing the linear regression results with the random forest model. As shown in Figure 22, the global linear model achieved a modest $R^2 = 0.3793$ with a significant error ($RMSE = 2.8585 \text{ m/s}^2$). Even after removing outliers, the linear model only reached $R^2 = 0.4815$. In contrast, the optimized RF model (Figure 26)

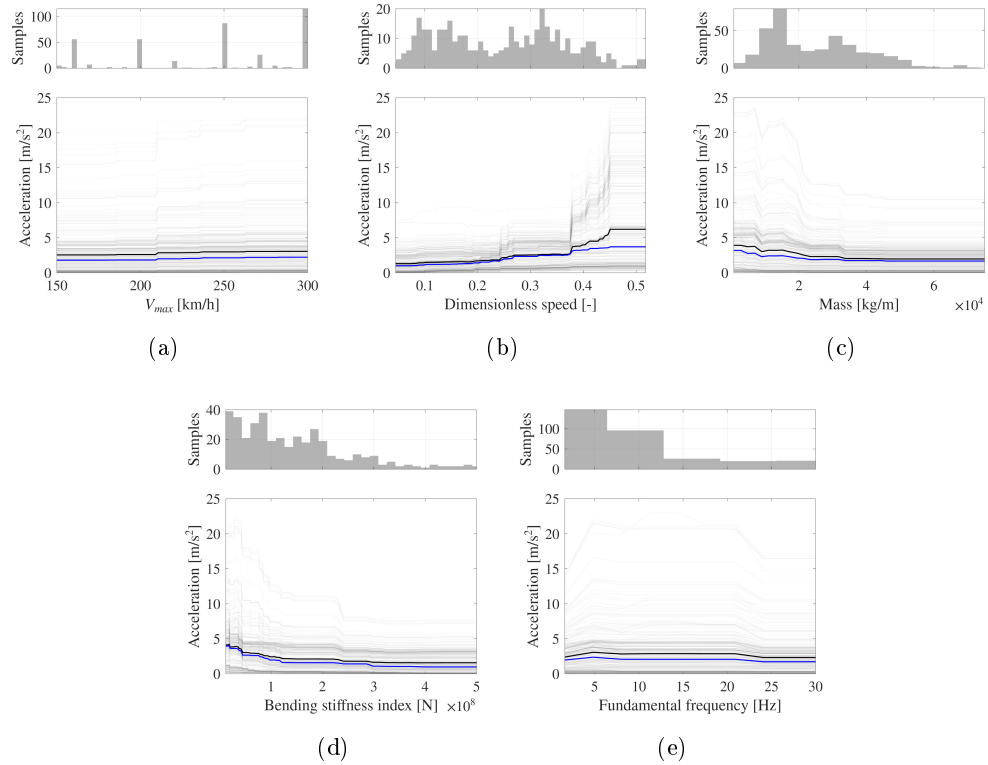


Figure 28: Interpretability analysis of the random forest model for the complete bridge set. (a-e) Partial Dependence Plots (PDP) and Individual Conditional Expectation (ICE) curves for critical parameters. The thick black line represents the average global trend (mean PDP), the blue line indicates the median response (robust trend), and the grey lines show individual bridge sensitivities (ICE). Top histograms illustrate the sample density across the training range.

achieved an $R^2 = 0.8770$ and an $RMSE$ of 0.9440 m/s^2 on unseen data. The SLR residuals (Figure 22(a)) show a wide, asymmetrical distribution, indicating that the model systematically fails to capture high-acceleration events. However, the RF residuals are centred around zero, demonstrating that the ensemble approach significantly reduces both bias and variance compared to the linear approach.

The fundamental deficiency of the linear model lies in its assumption of proportionality. In railway bridge dynamics, acceleration does not increase linearly with speed or frequency; instead, it exhibits sudden peaks due to resonance. Figures 22-25 show that even when segmented by configuration (simply-supported, continuous, or frame), the linear models consistently lead to R^2 values below 0.5. The RF model, through the introduction of the dimensionless speed, effectively linearises the problem and, while the SLR model treats speed as a simple multiplier, the RF model identifies it as part of a trigger mechanism for resonant conditions, as seen in the non-linear jumps of the ICE curves. The comparative analysis suggests that while linear models can provide a rough estimation of average behaviour, they are unreliable for applications in railway engineering. The RF model acts as an accurate surrogate that can identify the resonance response of the bridges. The transition from $R^2 \approx 0.50$ (LR) to $R^2 \approx 0.88$ (RF) represents a shift from a simple statistical fit to a model that mimics the underlying

structural behaviour.

The performance of the Random Forest model was further scrutinized by segmenting the dataset into structural configurations (Table 1 and Figures 29–37). This sub-analysis reveals critical insights into the dynamic behaviour of each typology.

Simply-supported bridges exhibited the highest $RMSE$ (2.9625 m/s^2), consistent with their high susceptibility to resonance. Predictor importance for this group (Figure 30) highlights that, apart from the dimensionless speed, mass plays a key role, acting as the primary inertial moderator of the response.

Continuous bridges showed the most balanced performance ($R^2 = 0.6811$), where the bending stiffness index emerged as a significant predictor in this configuration.

Frame structures presented an R^2 of 0.8154 and achieved the lowest absolute error $RMSE = 0.0710 \text{ m/s}^2$. This result is explained by the high stiffness of frame configurations. These structures experience very low accelerations with minimal dispersion. It is important to note that the frame category encompasses three distinct sub-configurations: open frames, closed frames, and simple frames. Despite this internal geometric variety, the model achieved its highest absolute precision in this category.

Across all cases, the dimensionless speed and bending stiffness index remained the top-ranked predictors, proving that the proposed physical feature engineering is a good descriptor of the dynamic risk regardless of the bridge longitudinal configuration. These results confirm that the global model benefits from learning the cross-categorical boundaries, while the individual models highlight the specific physical drivers of each structural type.

Table 1: Performance metrics of the Random Forest model disaggregated by structural configuration.

Configuration	$RMSE$ (m/s^2)	R^2
Simply-supported	2.9625	0.5858
Continuous	1.0213	0.6811
Frame	0.0710	0.8154
Global Model	0.9440	0.8770

The findings of this study demonstrate that the optimized Random Forest model, enhanced by physical feature engineering, serves as an accurate surrogate for predicting the dynamic response of railway bridges. In engineering practice, this model can be implemented as a robust screening tool for large-scale infrastructure assessment. By utilizing the dimensionless speed as a primary performance indicator, engineers could rapidly identify bridges at risk of exceeding acceleration limits without the need for computationally expensive time-history simulations. The methodology allows for a two-tier management approach: i) Fast Screening: the model filters thousands of bridge configurations in seconds using only basic geometric and operational data, flagging resonant-prone structures, and ii) Targeted Analysis: resources for complex dynamic analyses can then be focused exclusively on these flagged cases, significantly reducing the overhead costs and time of railway maintenance programs.

Based on the predictor importance and correlation analyses, the primary critical

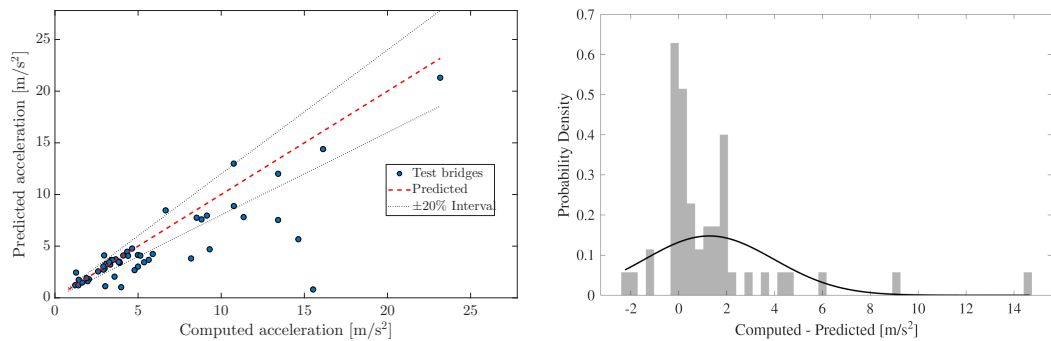


Figure 29: Performance of the optimized Random Forest model for the simply-supported bridge population on the test set: (a) Predicted vs. Actual acceleration with $\pm 20\%$ confidence intervals and (b) probability density of residuals.

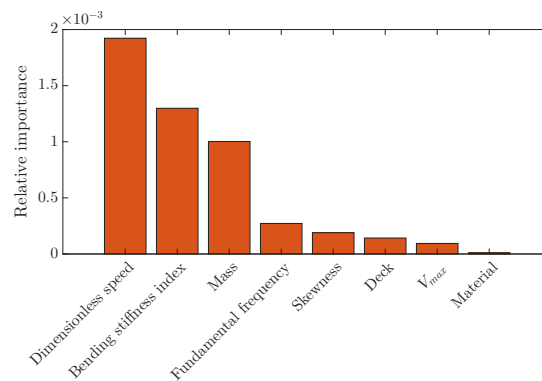


Figure 30: Relative importance of predictors of the random forest model for the simply-supported bridge population.

parameters governing the dynamic safety of the network are formally identified as: (i) the longitudinal configuration, (ii) the dimensionless speed, (iii) the mass per unit length, (iv) the bending stiffness index, and (v) the fundamental frequency of the bridge.

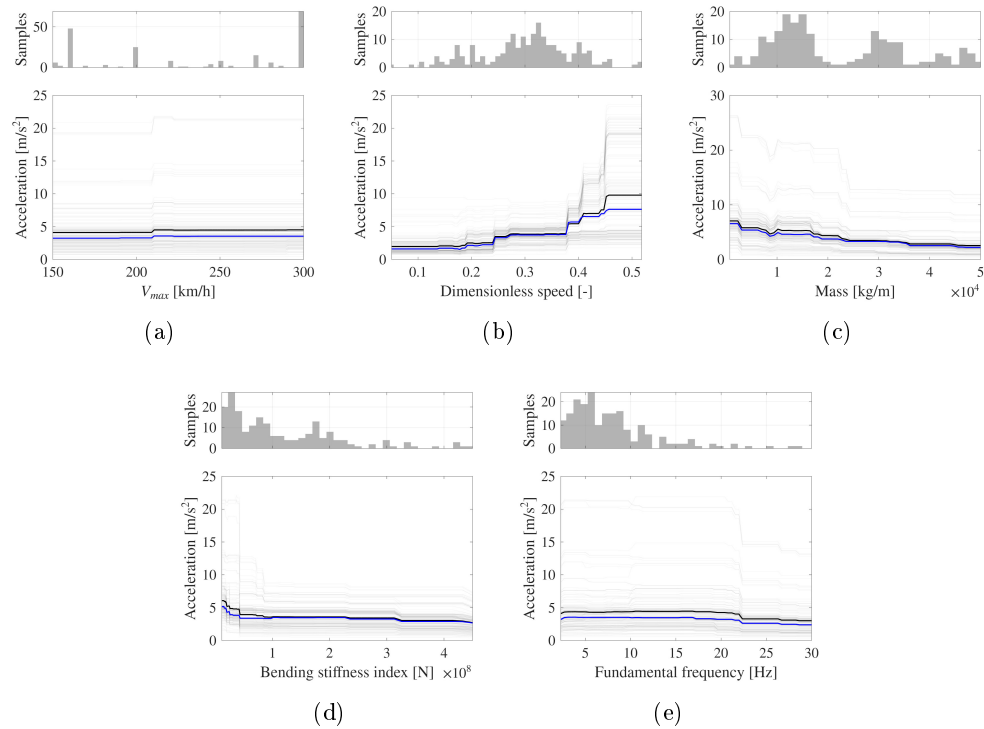


Figure 31: Interpretability analysis of the random forest model for the simply-supported bridge population. (a-e) Partial Dependence Plots (PDP) and Individual Conditional Expectation (ICE) curves for critical parameters. The thick black line represents the average global trend (mean PDP), the blue line indicates the median response (robust trend), and the grey lines show individual bridge sensitivities (ICE). Top histograms illustrate the sample density across the training range.

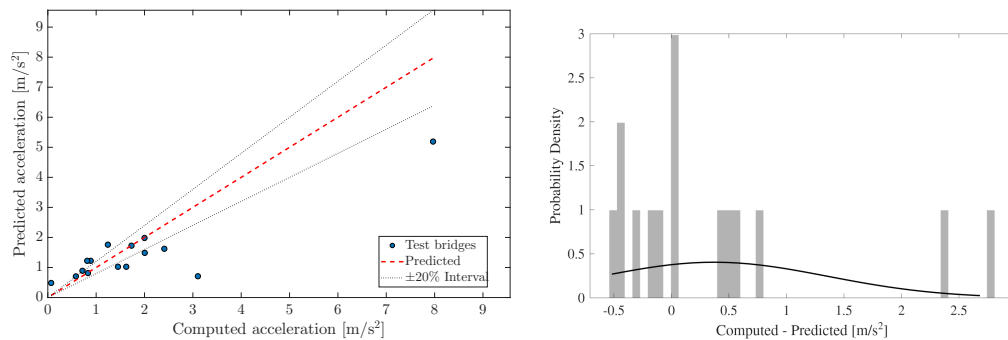


Figure 32: Performance of the optimized Random Forest model for the continuous bridge population on the test set: (a) Predicted vs. Actual acceleration with $\pm 20\%$ confidence intervals and (b) probability density of residuals.

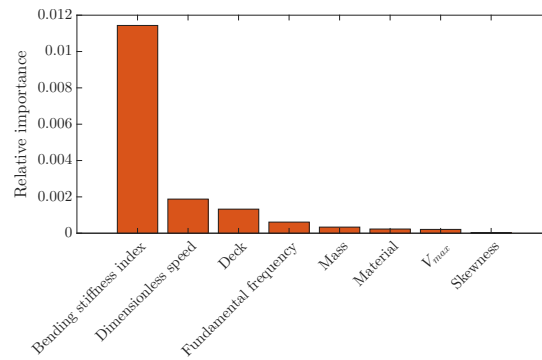


Figure 33: Relative importance of predictors of the random forest model for the continuous bridge population.

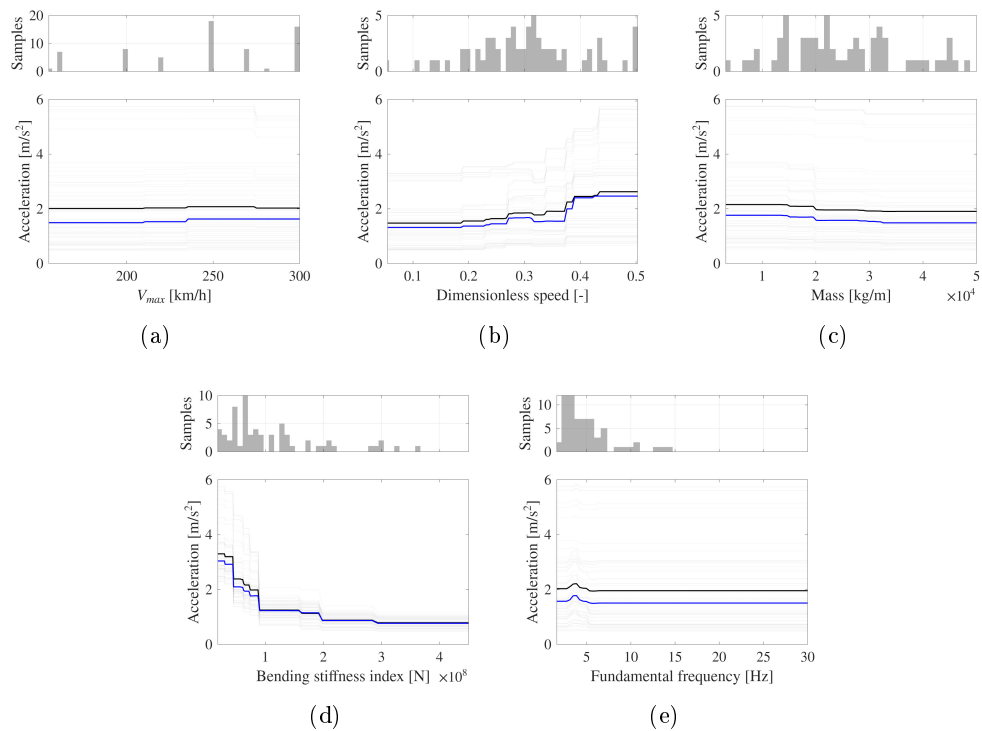


Figure 34: Interpretability analysis of the random forest model for the continuous bridge population. (a-e) Partial Dependence Plots (PDP) and Individual Conditional Expectation (ICE) curves for critical parameters. The thick black line represents the average global trend (mean PDP), the blue line indicates the median response (robust trend), and the grey lines show individual bridge sensitivities (ICE). Top histograms illustrate the sample density across the training range.

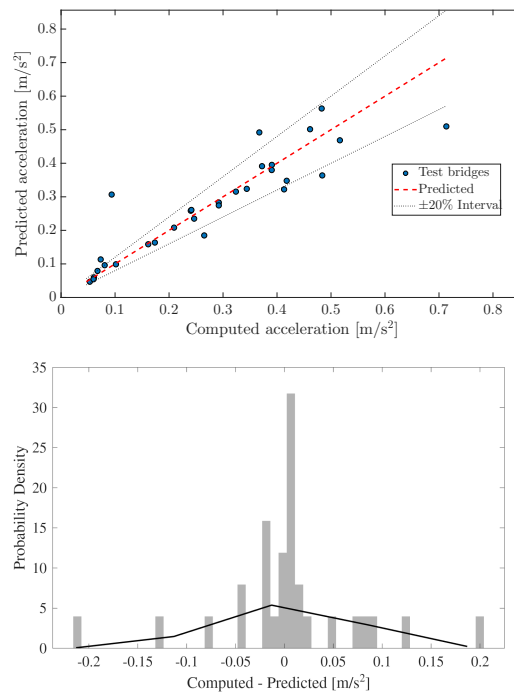


Figure 35: Performance of the optimized Random Forest model for the frame bridge population on the test set: (a) Predicted vs. Actual acceleration with $\pm 20\%$ confidence intervals and (b) probability density of residuals.

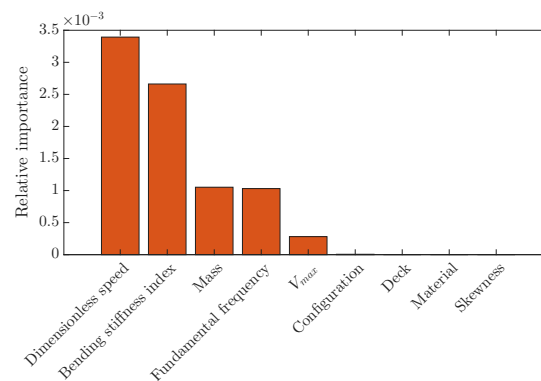


Figure 36: Relative importance of predictors of the random forest model for frame bridge population.

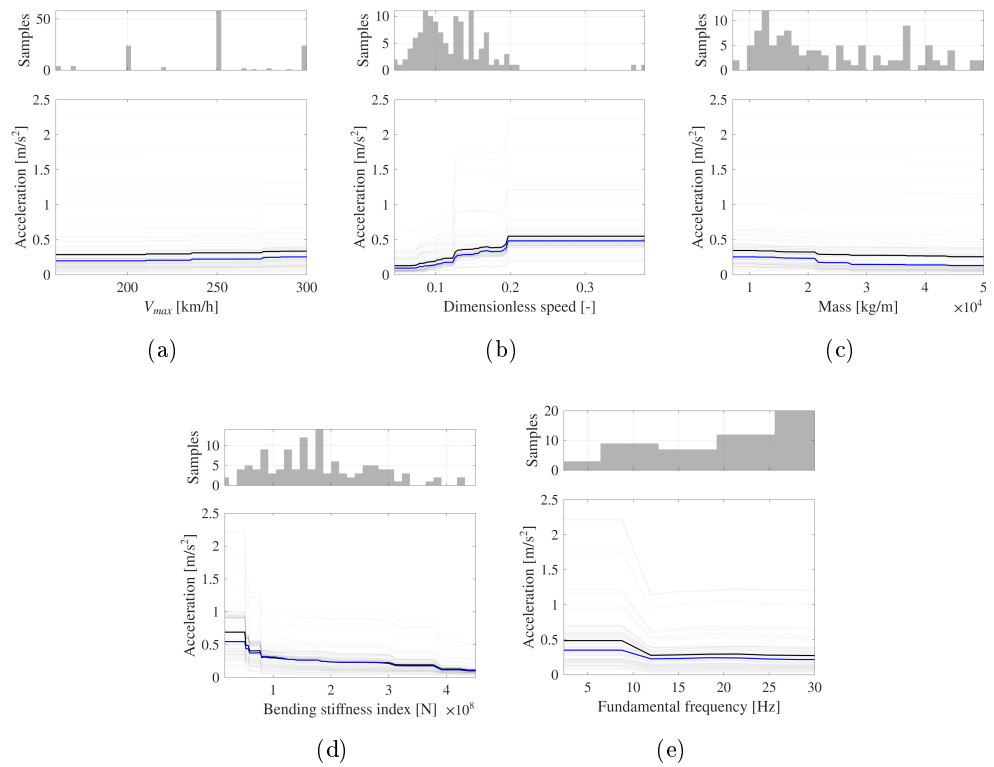


Figure 37: Interpretability analysis of the random forest model for the frame bridge population. (a-e) Partial Dependence Plots (PDP) and Individual Conditional Expectation (ICE) curves for critical parameters. The thick black line represents the average global trend (mean PDP), the blue line indicates the median response (robust trend), and the grey lines show individual bridge sensitivities (ICE). Top histograms illustrate the sample density across the training range.

3.4 Analysis over a synthetic 1 million bridge population: Deck performance evaluation

A synthetic sample of one million bridges is generated within the parameter range of the existing database, evenly distributed across the longitudinal configurations. Using the predictive model, the maximum acceleration induced by the 26 train passages is obtained and the influence of various parameters is studied by categorizing the bridges according to their longitudinal configuration and deck typology. In the following figures, the color scale is saturated at 5 m/s^2 , which corresponds to the threshold considered acceptable for serviceability in ballasted tracks railway bridges.

Figures 38 to 40 present the maximum vertical acceleration results for the synthetic population categorized into three increasing mass intervals ($m_1 - m_2$, $m_2 - m_3$, and $m_3 - m_4$) where m_1 and m_4 are the minimum and maximum values respectively, and m_2 and m_3 are the 33th and 66th percentiles of the dataset. These heatmaps permit evaluating the dynamic response across different parameter spaces: bending stiffness index and fundamental frequency versus dimensionless speed, and fundamental frequency versus bending stiffness index. The distribution shows a clear critical red zone associated with specific combinations of low stiffness and high dimensionless speeds. Furthermore, the transition across the mass intervals reveals how increased bending stiffness index shifts these regions, generally expanding the stable operational zones as the mass per unit length of the deck increases. The critical zones are predominantly concentrated in the lowest fundamental frequency range ($< 5 \text{ Hz}$) when combined with high dimensionless speeds. In the lowest mass interval, the maximum acceleration consistently remains within the saturated red zone across almost the full fundamental frequency range. This indicates that for bridges in the lowest mass range, the structural mass becomes the governing factor of the dynamic response. In this regime, the system lacks sufficient mass to attenuate the energy induced by the train passage, leading to serviceability limit exceedances even when the fundamental frequency is far from the resonance condition. In Figure 40, the interaction between fundamental frequency and the bending stiffness index reveals that a low fundamental frequency does not always lead to exceedance; however, when a low fundamental frequency coincides with low stiffness, the bridge enters the saturated acceleration zone. As the mass interval increases (moving from $m_1 - m_2$ to $m_3 - m_4$), these critical areas shift, demonstrating how the mass of the deck redefines the resonance boundaries associated with the bridge fundamental frequency.

3.4.1 Simply-supported bridges

Figure 41 provides a comprehensive parametric sensitivity and coupling analysis for the full subset of simply-supported bridges. These maps show how the maximum vertical acceleration is influenced by the interaction of structural properties and operational speed: Figure 41(a) confirms that low-mass decks ($< 2 \times 10^4 \text{ kg/m}$) are extremely sensitive to speed even at low dimensionless speeds. As mass increases, the threshold speed at which the 5 m/s^2 limit is exceeded shifts progressively to higher values. Figure 41(b) shows that at low dimensionless speeds only specific frequency ranges trigger high accelerations, but as the dimensionless speed increases beyond 0.3, the acceleration becomes critical across almost the complete frequency spectrum. Figure 41(c) shows a clear boundary: a lack of mass can be partially compensated for by very high stiffness, but there is a quadrant with low mass and low stiffness where serviceability is not met.

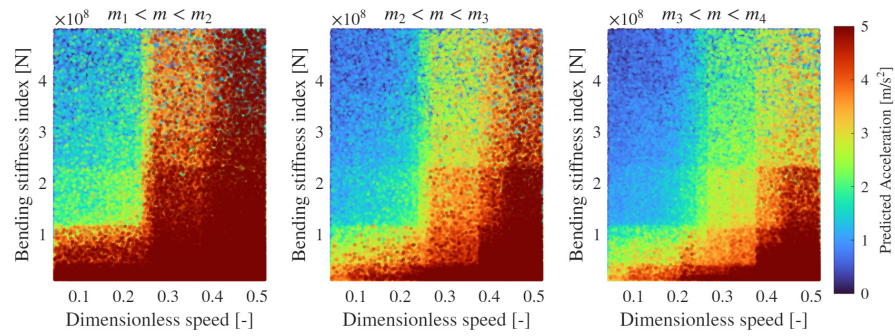


Figure 38: Parametric analysis of predicted vertical acceleration as a function of dimensionless speed and bending stiffness. The data is categorized into three mass per unit length intervals defined by the following thresholds: $m_1 = 711$, $m_2 = 7079$ (33^{rd} percentile of the dataset), $m_3 = 13446$ (66^{th} percentile of the dataset), and $m_4 = 20000 \text{ kg/m}$. The color scale is saturated at 5 m/s^2 to highlight configurations exceeding the maximum acceleration limit.

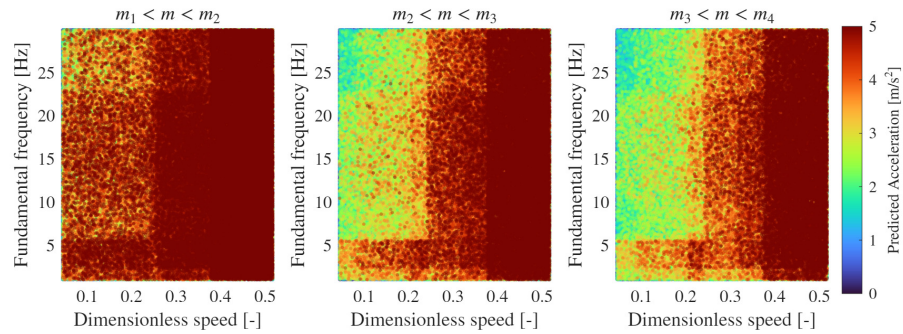


Figure 39: Parametric analysis of predicted vertical acceleration as a function of dimensionless speed and fundamental frequency. The data is categorized into three mass per unit length intervals defined by the following thresholds: $m_1 = 711$, $m_2 = 7079$ (33^{rd} percentile of the dataset), $m_3 = 13446$ (66^{th} percentile of the dataset), and $m_4 = 20000 \text{ kg/m}$. The color scale is saturated at 5 m/s^2 to highlight configurations exceeding the maximum acceleration limit.

Figure 41(d) shows a map relating fundamental frequency and mass predominantly saturated, particularly in the lowest mass region.

Figure 42 shows the distribution of maximum vertical accelerations for simply-supported bridges, categorized by deck typology, as a function of their mass per unit length and bending stiffness index. The background scatter plot represents the synthetic population, while the black lines indicate the mean values of the actual bridges in the database. The shaded grey area delimits the range defined by the mean $\pm$ one standard deviation, providing a visual benchmark of where existing structures stand relative to the critical acceleration zones. A comparative analysis across typologies reveals distinct safety margins. For instance, in box-girder and slab hollow decks, the mean values and their standard deviations are predominantly located within the blue and green regions, suggesting a robust dynamic design. Conversely, in lighter typologies such as filler beam or U-type decks, the mean values and the shaded regions sit significantly closer to the red saturated zone, indicating that a larger portion of the existing population of these

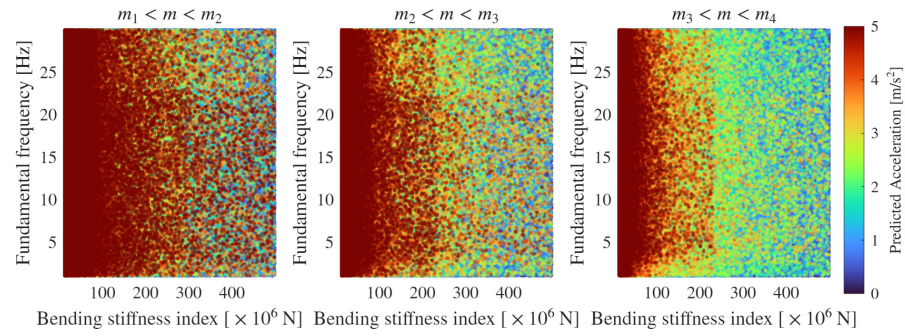


Figure 40: Parametric analysis of predicted vertical acceleration as a function of fundamental frequency and bending stiffness. The data is categorized into three mass per unit length intervals defined by the following thresholds: $m_1 = 711$, $m_2 = 7079$ (33rd percentile of the dataset), $m_3 = 13446$ (66th percentile of the dataset), and $m_4 = 20000$ kg/m. The color scale is saturated at 5 m/s² to highlight configurations exceeding the maximum acceleration limit.

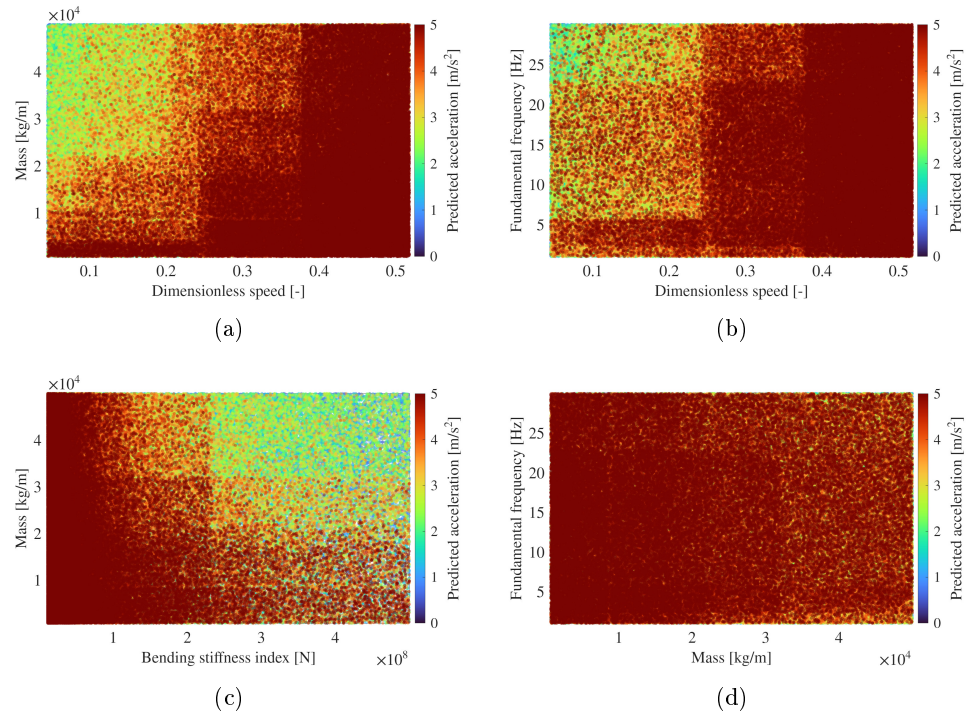


Figure 41: Parametric sensitivity and coupling analysis of vertical acceleration for simply-supported bridges: (a) influence of mass per unit length and (b) fundamental frequency as a function of dimensionless speed; (c) interaction between mass and bending stiffness; and (d) correlation between fundamental frequency and mass per unit length. The color scale is saturated at 5 m/s².

types is susceptible to exceeding serviceability limits. The plots also confirm that for almost all typologies, the critical zone is concentrated at the intersection of low mass and low stiffness. The varied shapes and locations of the grey area across subplots highlight that deck typology is a primary factor in governing the dynamic reliability of

the railway bridge network.

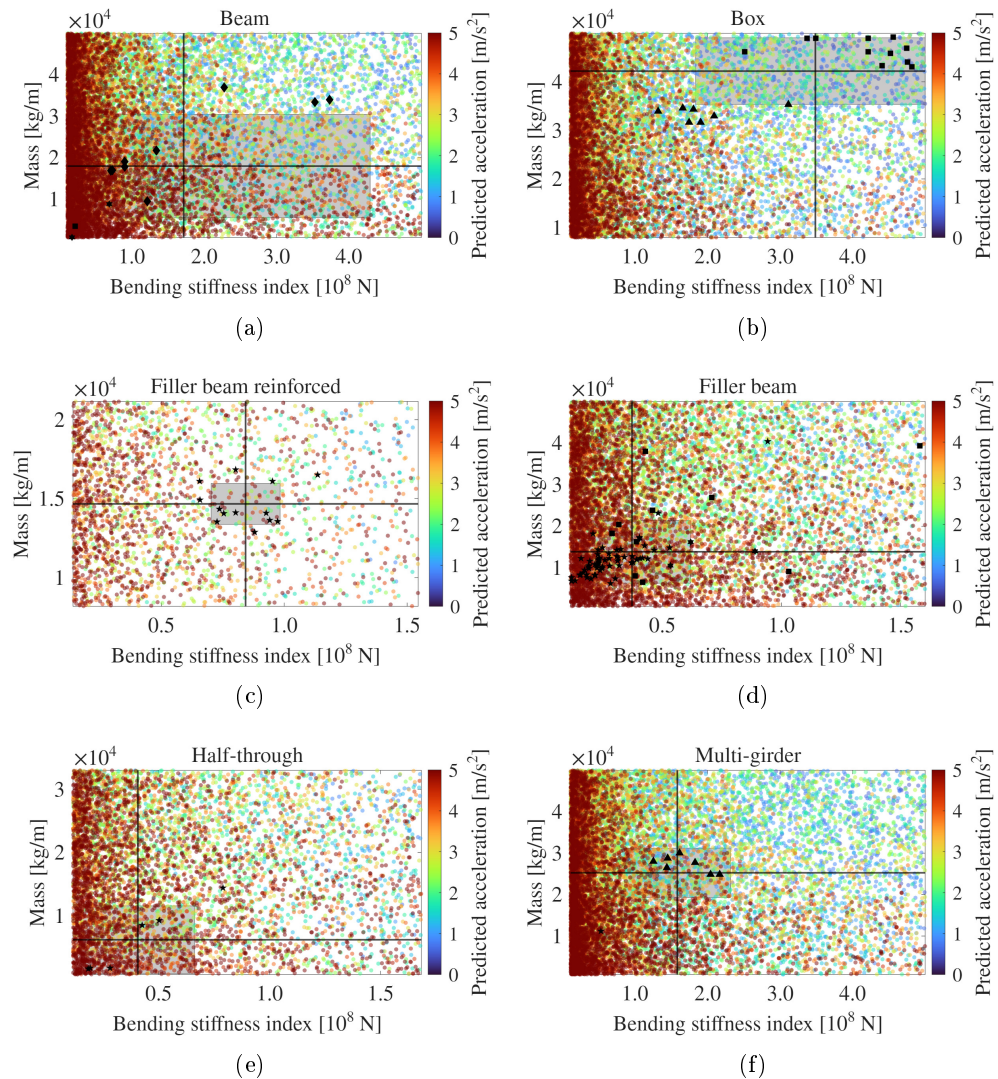


Figure 42: Analysis of vertical acceleration for simply-supported bridges as a function of mass and bending stiffness index for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 . (Continued on next page)

In Figure 43 the dynamic response is evaluated by crossing mass per unit length and fundamental frequency for each deck typology of simply-supported bridges. This visualization reveals that resonance vulnerability is not uniformly distributed across frequencies. For most typologies, the critical red zone is significantly more pronounced at lower frequencies, where the interaction with typical train speeds and axle distances is more likely to trigger high-amplitude vibrations. The positioning of the mean values and the standard deviation relative to the frequency axis provides a direct measure of the spectral safety of each bridge type. In typologies such as box-girder and slab hollow, the database population is centred in frequency ranges that, combined with their high

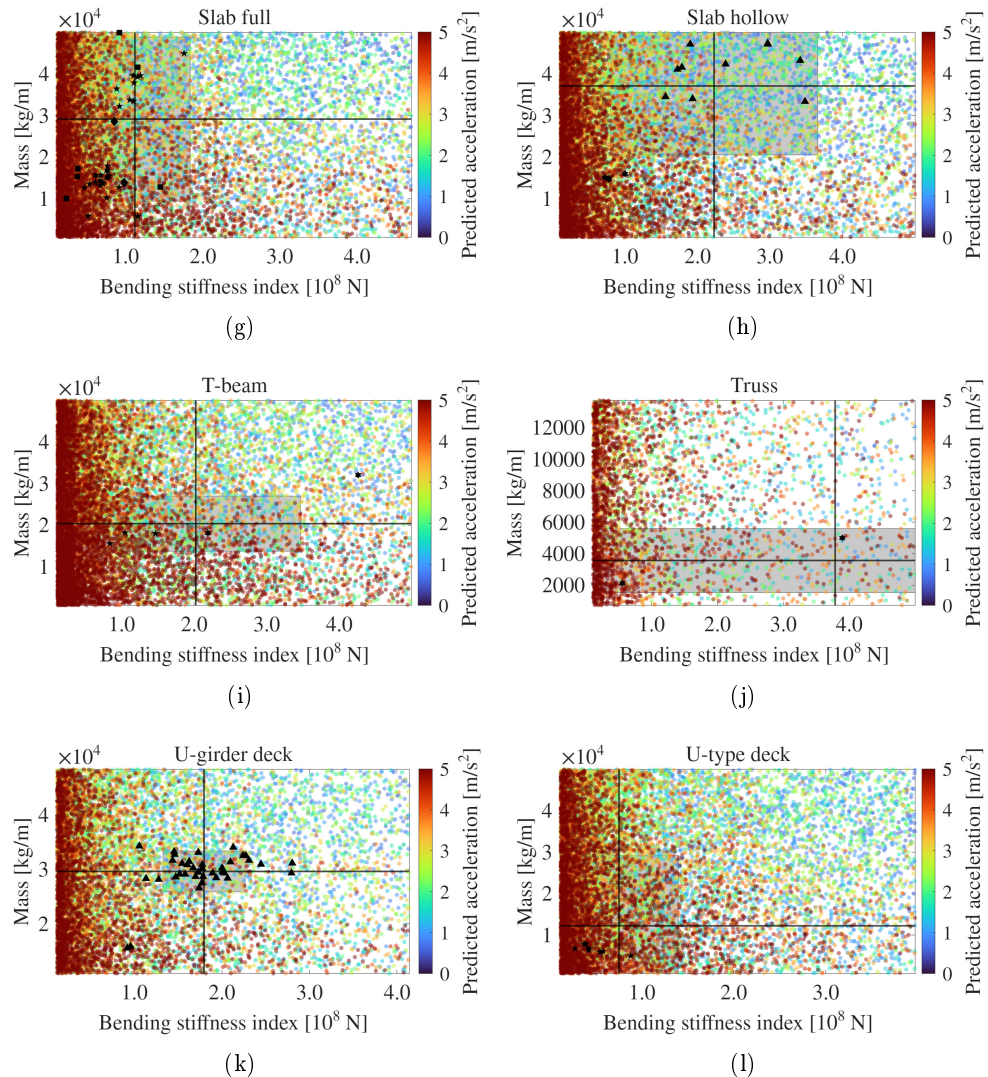


Figure 42: Analysis of vertical acceleration for simply-supported bridges as a function of mass and bending stiffness index for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 .

mass, keep the accelerations within the safe zones. In contrast, for filler beam, and U-type bridges, the grey area is not only situated in a low-mass region but also overlaps with frequency bands that are highly susceptible to high acceleration levels.

The analysis of these figures shows that low bending stiffness cannot be compensated by high mass, whereas a very stiff bridge can perform adequately even with limited mass. In general, low frequency bridges may perform well if sufficient mass is present, while high frequency ones could become problematic when associated with low masses. In short, a bridge having a low natural frequency is not necessarily a problem. The same train will induce resonance at a lower speed; however, if the bridge has sufficient mass (generally accompanied by sufficient stiffness), the response will not be high.

Within this framework, existing bridges performance varies significantly by typology. Filler beam bridges, characterized by very low stiffness and mass, tend to perform poorly despite exhibiting moderate natural frequencies (the mean being close to 10 Hz), and would require increases in both parameters. In contrast, box-girder bridges exhibit high bending stiffness (10 times higher than filler beams) and mass, resulting in low dynamic response and a very good performance, despite exhibiting low fundamental frequencies. Beam bridges present intermediate stiffness but relatively low mass, suggesting that an increase in mass could improve their behavior, while U-girder bridges occupy an intermediate position with satisfactory performance at low frequencies. Slab bridges show mixed behavior: full slabs resemble filler beams in their limitations, whereas hollow slabs, with greater stiffness for similar mass, are less prone to dynamic issues. Finally, half-through bridges, combining low stiffness with even lower mass, are inherently unfavorable from a dynamic performance standpoint.

3.4.2 Continuous bridges

The parametric sensitivity analysis for continuous bridges, presented in Figure 44, reveals a dynamic response structure that differs significantly from the patterns observed in simply-supported schemes. In the relationships involving dimensionless speed (Figures 44(a)) and 44(b)), structural continuity is shown to generate a much broader zone of low accelerations at moderate speeds, effectively shifting acceleration saturation towards more demanding operational ranges. From the interaction between mass and the bending stiffness index (Figures 44(c)), it can be concluded that continuous bridges exhibit a nearly vertical stiffness threshold that suggests the existence of a critical stiffness value below which the bridge is unable to meet serviceability criteria, regardless of its structural mass. Once this threshold is reached, the system undergoes an abrupt transition toward lower acceleration levels, showing that in the continuous bridges the bending stiffness index is the primary filter for dynamic control. Finally, the relationship between fundamental frequency and mass (Figures 44(d)) shows that, despite the advantages of hyperstaticity, mass per unit length remains a governing factor for robust behaviour across the frequency spectrum, as the saturated red area persists dominantly in the lowest mass ranges.

The analysis by deck typology for continuous bridges, shown in Figure 45, confirms that the structural continuity provides a significantly higher safety margin across the full database. Unlike the simply-supported cases, the mean values (black lines) and the standard deviation areas (grey shaded zones) for almost all typologies are well positioned within the regions with low accelerations levels. This shift is particularly evident in the mass and bending stiffness interaction plots. For typologies like the multi-girder and U-girder deck, the existing bridges are designed in a range of stiffness that avoids high acceleration levels, even when their mass is relatively low. For continuous systems, the combination of extremely low mass and low stiffness represents a localized risk design area much smaller and less frequent than in simply-supported bridges.

The relationship between mass per unit length and fundamental frequency for continuous bridges is detailed by typology in Figure 46. An initial observation is the significant migration of the database population away from the lower-left critical zone. For nearly all typologies the average structural configuration sits within regions corresponding to accelerations well below the serviceability threshold. In contrast to the simply-supported case, where low mass causes exceedance regardless of frequency, the continuity produces low accelerations at higher frequencies.

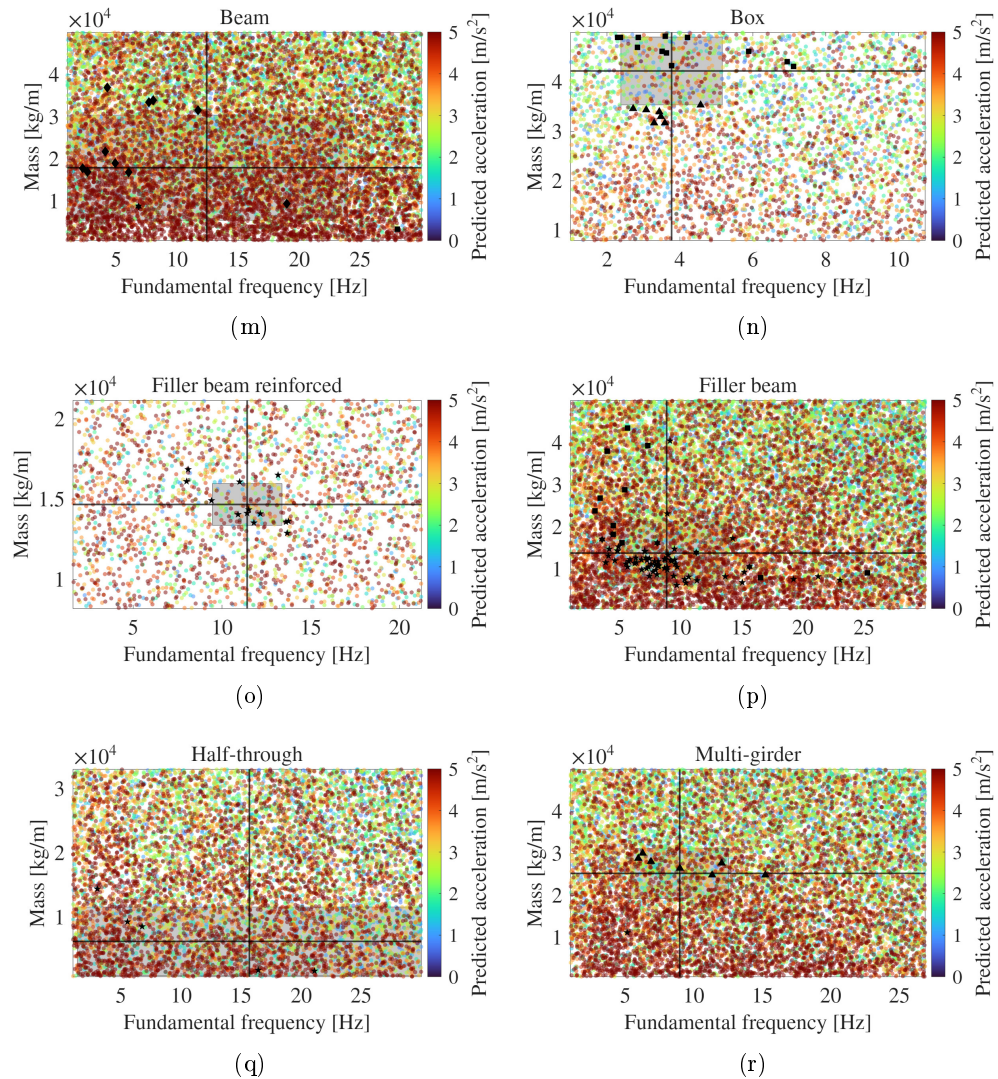


Figure 43: Analysis of vertical acceleration for simply-supported bridges as a function of mass and fundamental frequency for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 . (Continued on next page)

3.4.3 Portal frame bridges

The parametric analysis for frame bridges, shown in Figures 47 to 49, reveals the highest level of dynamic robustness among all investigated structural systems. The rigid connection between the deck and the substructure increases both the global stiffness and the fundamental frequencies of the system. In figures 48(c) and 49(c), the structural designs imply that the database mean values are located beyond the visible limits of the axes. As shown in the global sensitivity maps (Figure 47), frame bridges exhibit lower acceleration levels compared to simply-supported and continuous bridges. The interaction between mass and bending stiffness (Figure 47(c)) identifies a critical stiffness

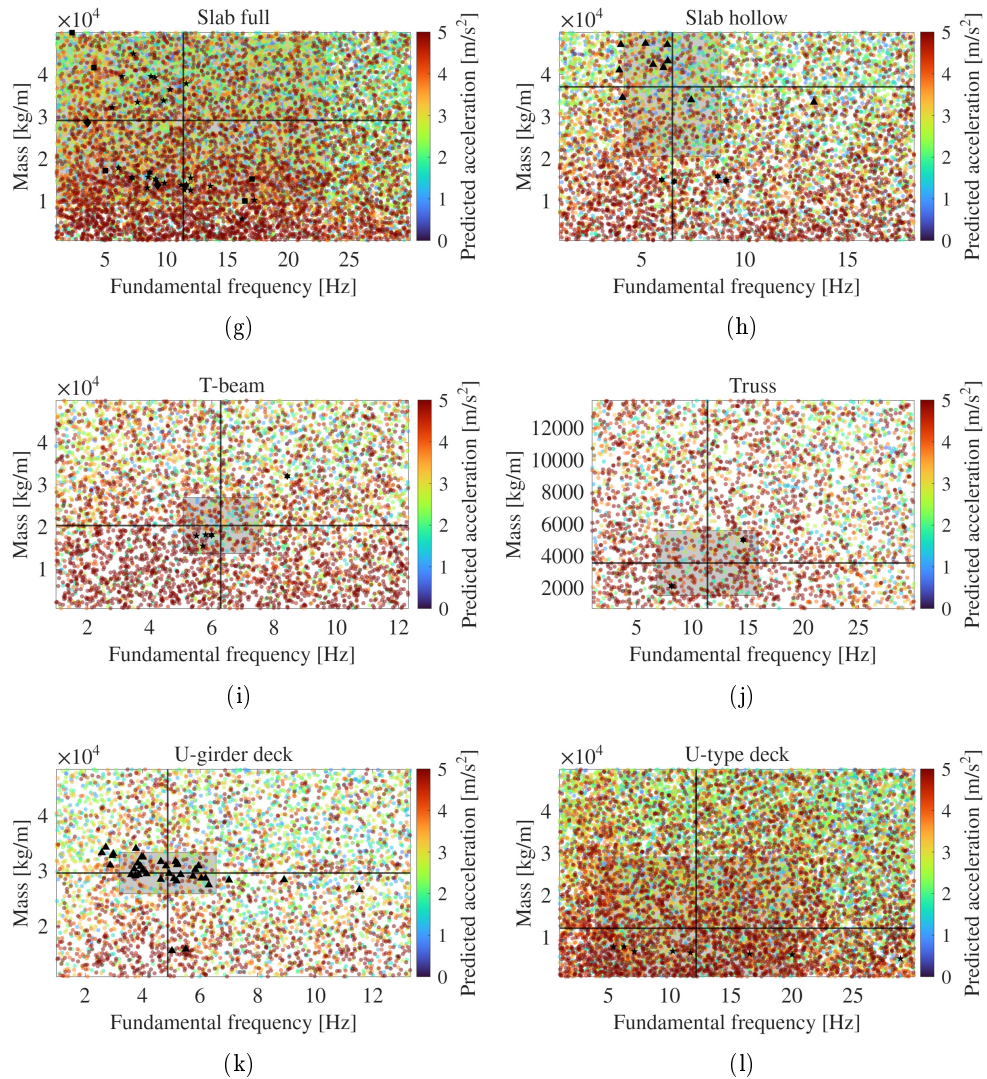


Figure 43: Analysis of vertical acceleration for simply-supported bridges as a function of mass and fundamental frequency for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 .

threshold even lower than that of continuous bridges. The analysis by typology (Figures 48 and 49) shows that the designs are consistently located in the low-acceleration regions.

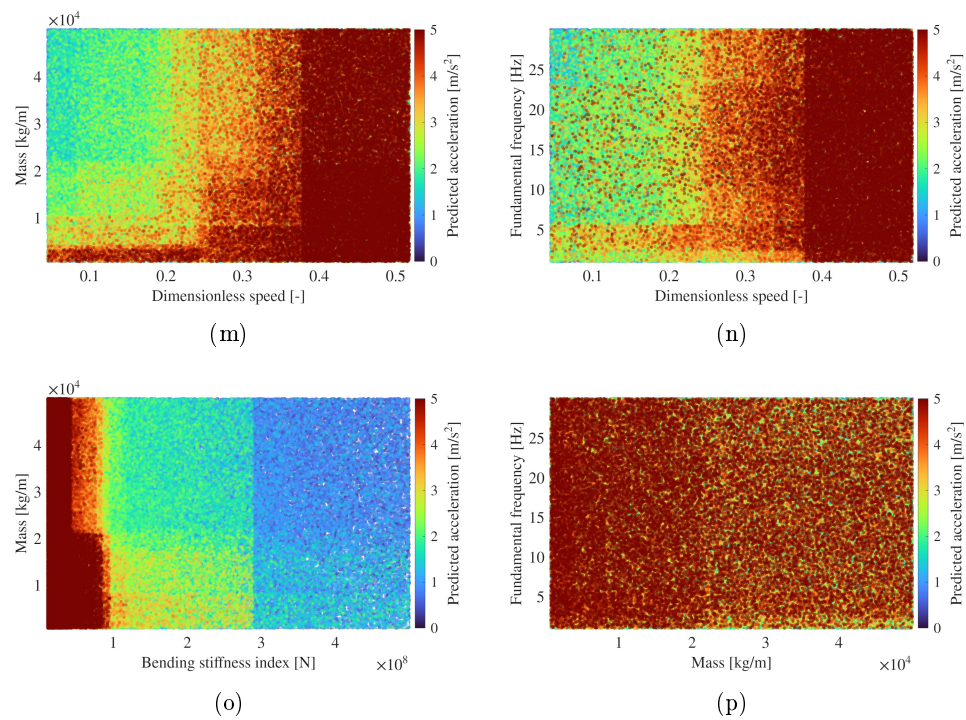


Figure 44: Parametric sensitivity and coupling analysis of vertical acceleration for continuous bridges: (a) influence of mass per unit length and (b) fundamental frequency as a function of dimensionless speed; (c) interaction between mass and bending stiffness; and (d) correlation between fundamental frequency and mass per unit length. The color scale is saturated at 5 m/s^2 .

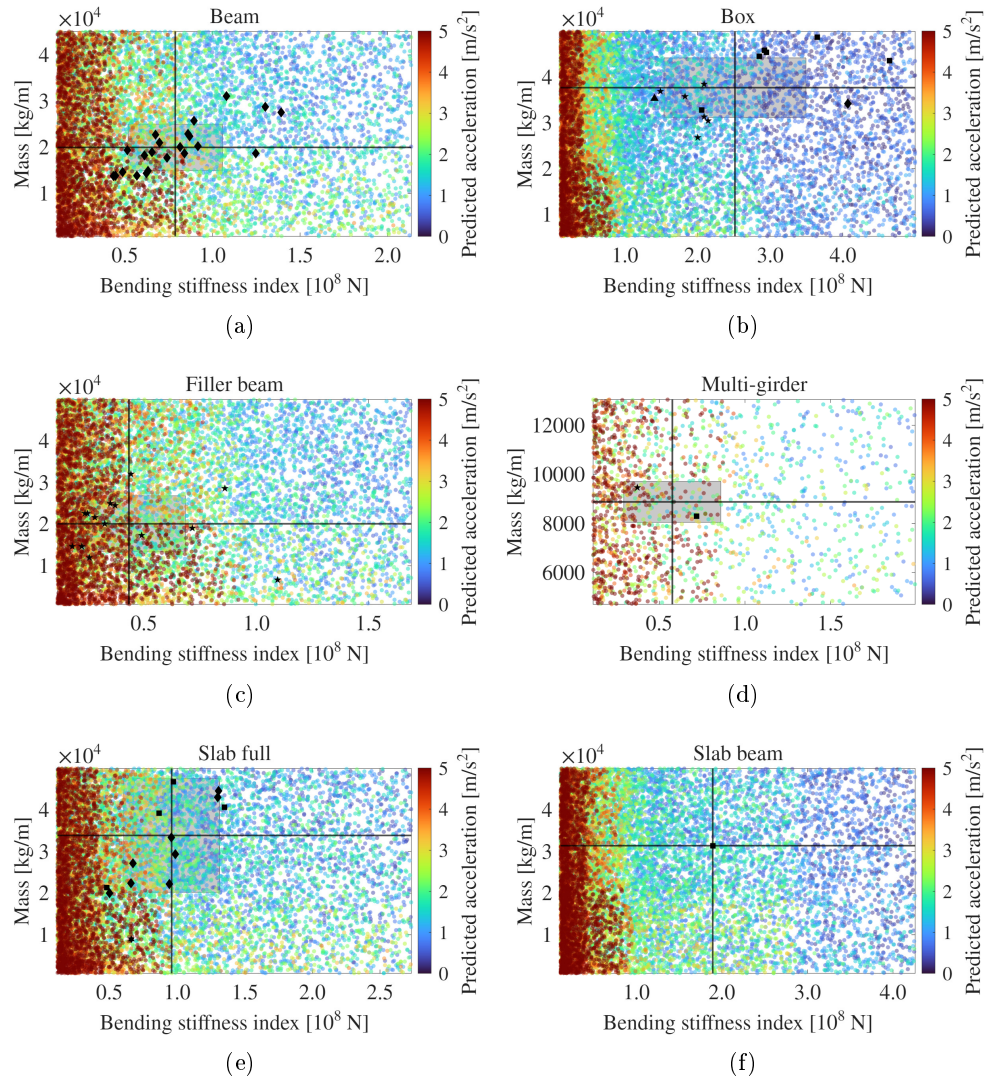


Figure 45: Analysis of vertical acceleration for continuous bridges as a function of mass and bending stiffness index for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 . (Continued on next page)

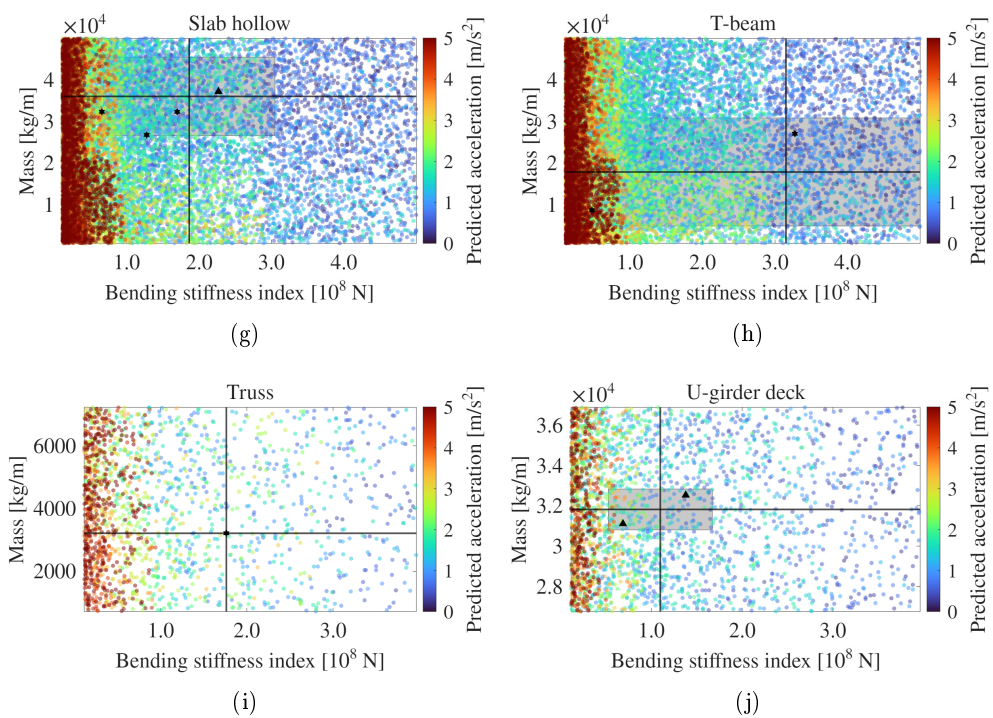


Figure 45: Analysis of vertical acceleration for continuous bridges as a function of mass and bending stiffness index for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 .

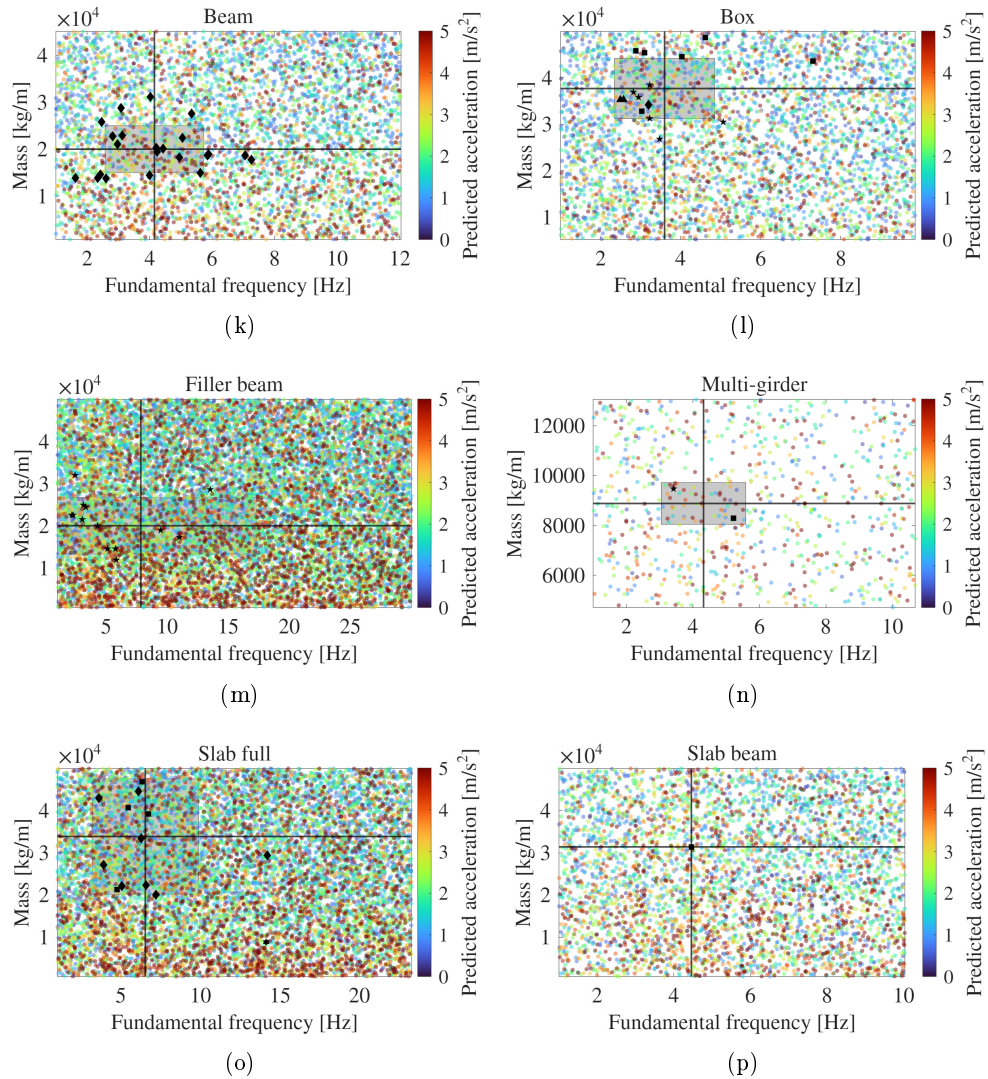


Figure 46: Analysis of vertical acceleration for continuous bridges as a function of mass and fundamental frequency for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagons (France), hexagons (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 . (Continued on next page)

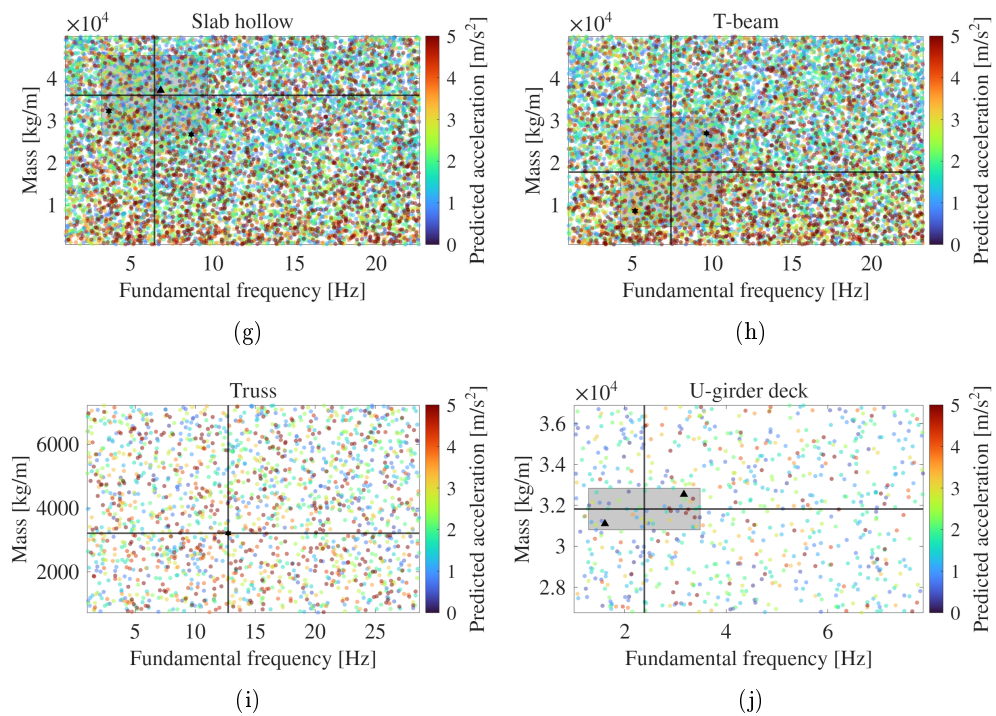


Figure 46: Analysis of vertical acceleration for continuous bridges as a function of mass and fundamental frequency for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 .

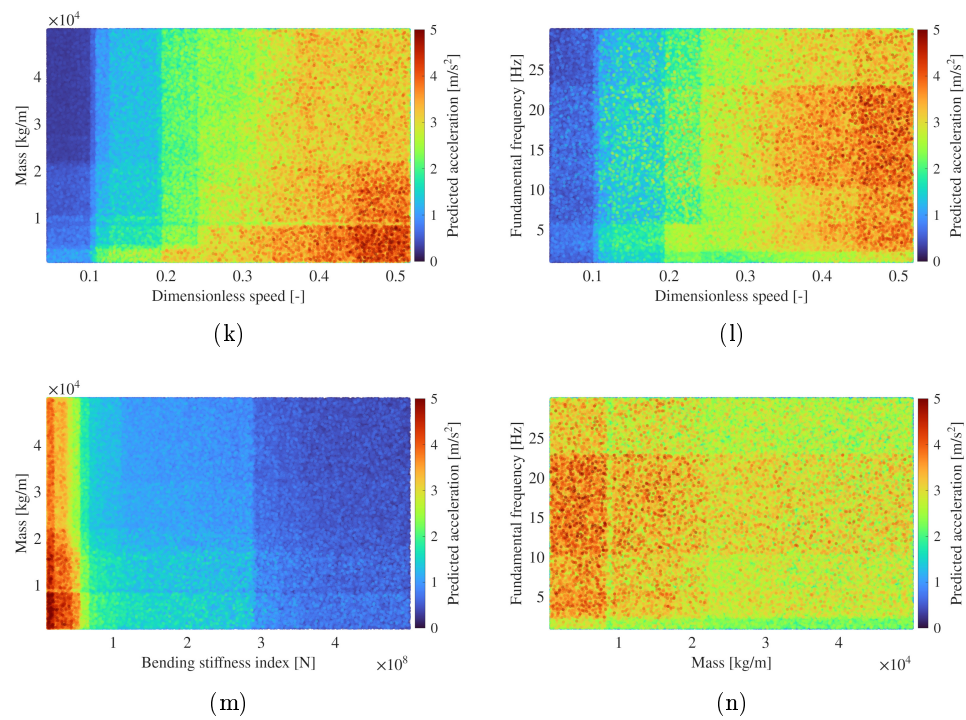


Figure 47: Parametric sensitivity and coupling analysis of vertical acceleration for frame bridges: (a) influence of mass per unit length and (b) fundamental frequency as a function of dimensionless speed; (c) interaction between mass and bending stiffness; and (d) correlation between fundamental frequency and mass per unit length. The color scale is saturated at 5 m/s^2 .

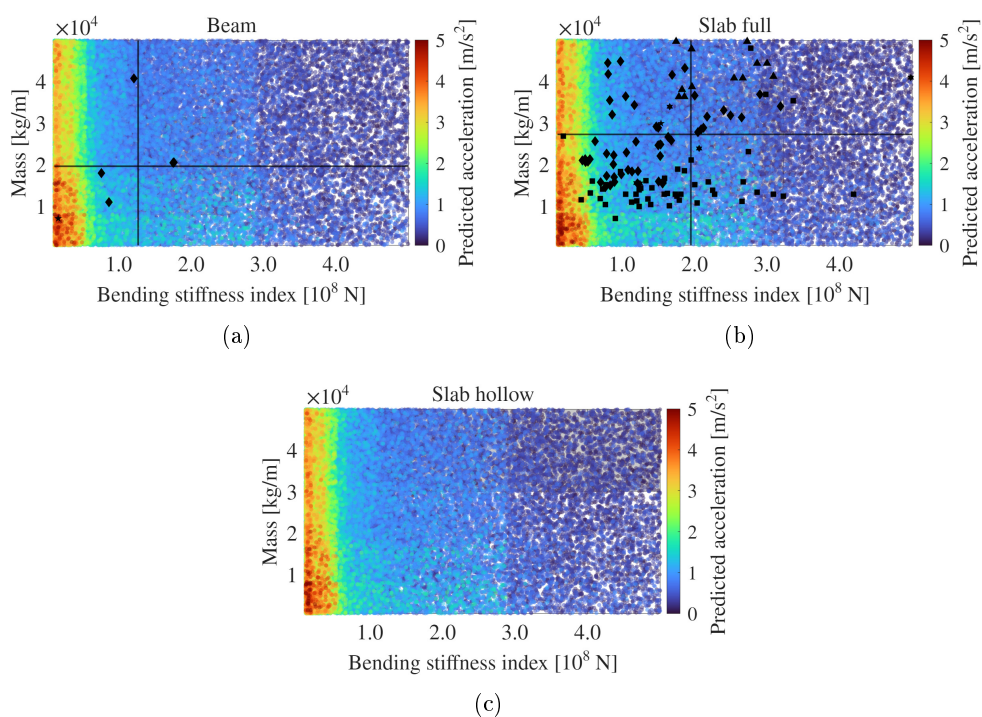


Figure 48: Analysis of vertical acceleration for frame bridges as a function of mass and bending stiffness index for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 .

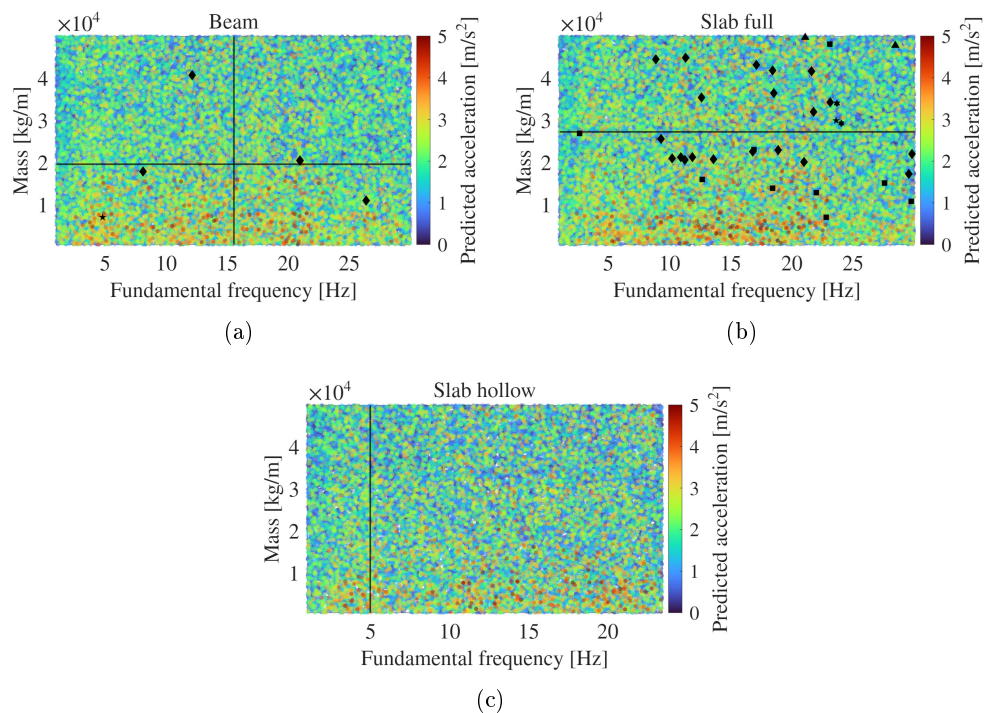


Figure 49: Analysis of vertical acceleration for frame bridges as a function of mass and fundamental frequency for the different deck types. Black lines represent the mean values for the bridges in the database (displayed as black squares (Germany), triangles (Spain), pentagrams (France), hexagrams (Portugal), and diamonds (Sweden)), while the shaded grey area denotes the range defined by the mean $\pm$ the standard deviation. The color scale is saturated at 5 m/s^2 .

3.4.4 Regression tree analysis

The regression tree analysis presented in Figure 50 summarizes the dynamic trends previously identified in the parametric sensitivity maps. The results reinforce the clear distinction between the three primary structural families: simply-supported spans, continuous bridges, and frames. For the more robust configurations, such as frames and continuous systems, the tree confirms a high degree of stability, where serviceability limits are rarely exceeded. Conversely, for the simply-supported typology, the analysis highlights a more complex interaction. Increasing bending stiffness does not yield a linear improvement in performance; instead, for certain combinations of low mass and high stiffness, the system is prone to frequency tuning with the train's excitation. The tree validates that while structural continuity provides an inherently lower baseline for accelerations, the dynamic safety of simply-supported bridges remains critically dependent on a balanced ratio of mass and stiffness to avoid spectral overlap with traffic load frequencies.

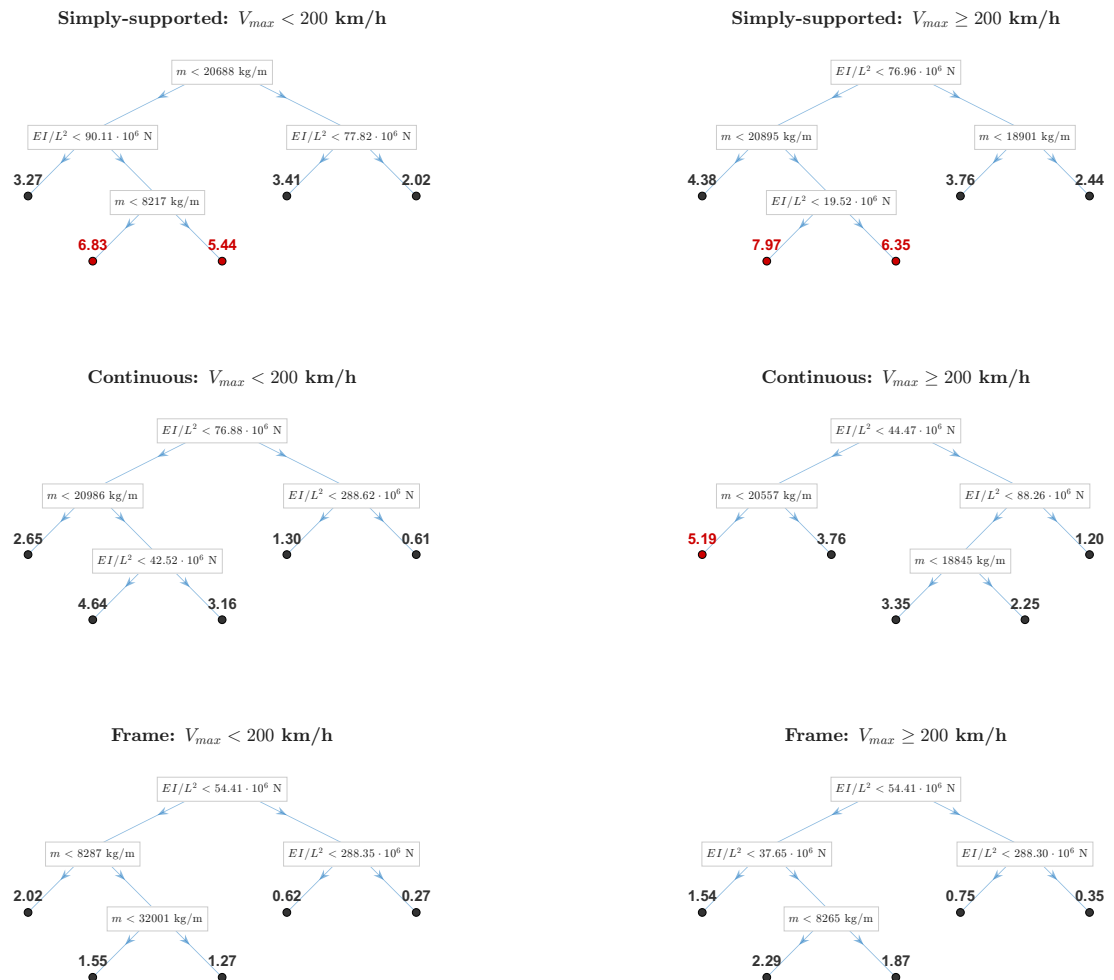


Figure 50: Regression tree for the multi-parametric prediction of vertical bridge acceleration. The terminal nodes show the mean predicted acceleration in m/s^2 . A color saturation is applied to acceleration values $\geq 5 \text{ m/s}^2$.

3.5 Predictive tool

Based on the developed RF LSBoost regression model, a design application (Figure 51) has been implemented to translate predictive results into structural engineering practice. It is important to emphasize that the predictive engine uses a single global model trained on the entire dataset, encompassing all structural configurations. This approach ensures that the algorithm captures the shared underlying physical behaviour across different bridge typologies while maintaining high generalization capabilities. To transform the raw predictions into a reliable engineering value, a conservative margin is incorporated by adding the $RMSE$ specific to each structural configuration. This allows the tool to provide a design acceleration (a_{design}) that accounts for the historical uncertainty of the model for each specific category: $a_{design} = a_{pred} + RMSE_{config}$ (Table 1).

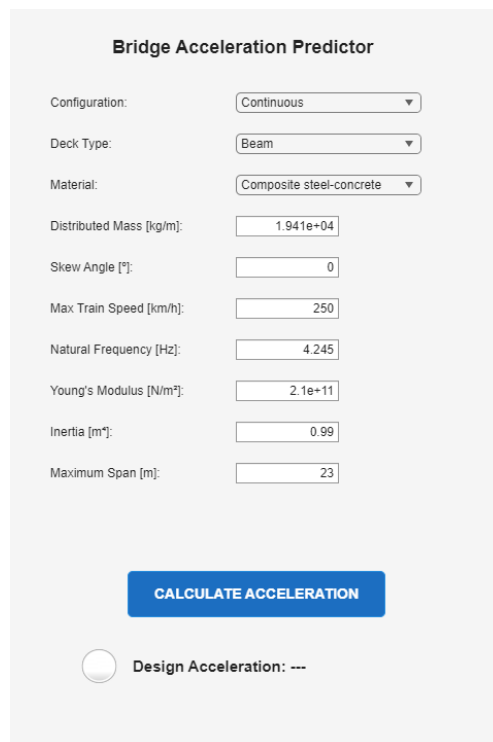


Figure 51: Matlab design application.

Figure 52(a) shows the relation between the computed accelerations and the design accelerations calculated by the tool. By integrating the configuration-specific $RMSE$, the data points exhibit a deliberate positive shift. This shift effectively establishes a conservative envelope, ensuring that for more than 95% of the considered bridges the predicted design value equals or exceeds the computed acceleration, thereby minimizing the risk of underestimation in structural assessments. Figure 52(b) shows the probability density function of the design errors (defined as $a_{design} - a_{pred}$). The distribution shows a multimodal profile, a direct result of applying discrete $RMSE$ offsets across different structural typologies. The concentration of the probability in the positive region provides empirical evidence of the tool robustness. This statistical distribution indicates that the proposed methodology effectively provides a quantifiable safety buffer, shifting the model uncertainty towards a conservative, safe design range.

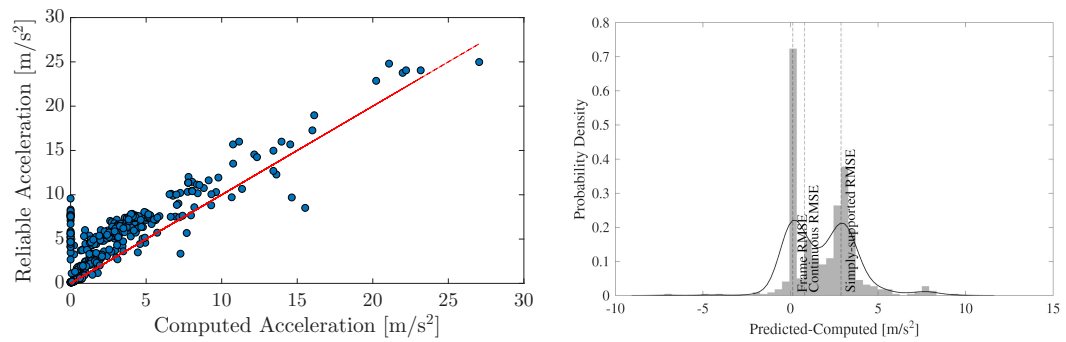


Figure 52: Performance evaluation of the proposed structural design tool: (a) scatter plot comparing the calculated design acceleration against the real computed acceleration. The red line represents the 1:1 identity line, and (b) the probability density function of the design errors. The *RMSE* offsets for different bridge configurations are indicated by the vertical dashed lines.

4 Recommendations

4.1 Worst-case combinations of critical parameters by bridge typology

The dynamic response and vulnerability to resonance of existing European railway bridges depend strongly on their longitudinal configurations and cross-sectional designs. Below are the recommended worst-case parameter combinations and ranges categorized by bridge typology:

- **Simply-Supported Bridges**

Simply-supported spans represent the most critical and resonance-prone longitudinal configuration, showing the highest susceptibility to train-induced vibrations.

- **Worst-Case Parameter Combination:** A simply-supported configuration combined with a low mass per unit length ($m < 7079 \text{ kg/m}$), a low bending stiffness index ($EI/L^2 < 10^8 \text{ N}$), and a fundamental frequency (n_0) within the critical resonant bandwidth of 5 – 12 Hz operating under dimensionless speeds higher than 0.3 ($\frac{V_{max}}{2Ln_0} > 0.3$).
- **Vulnerable Typologies:** Filler beam and half-through decks represent the worst-case representative designs because they naturally feature low mass and low stiffness. Despite having moderate natural frequencies (averaging around 10 Hz), filler beams generate the most severe acceleration levels.
- **Robust Typologies:** Box-girder and U-girder decks show good dynamic performance due to their higher rigidity and optimized stiffness-to-weight ratio. Meanwhile, hollow slab decks maintain high safety margins even at lower fundamental frequencies, relying on their substantial mass.

- **Continuous Bridges**

Continuous bridges show much higher dynamic stability than simply-supported schemes, with accelerations remaining consistently below 5 m/s^2 due to structural hyperstaticity.

- **Worst-Case Parameter Combination:** The worst-case is defined by a critical bending stiffness threshold. If the bending stiffness index falls below a specific threshold, the bridge is unable to meet serviceability requirements regardless of its structural mass. This value depends on the deck's cross-sectional properties, with the bending stiffness index (EI/L^2) falling below 10^8 N in all cases.
- **Vulnerable Typologies:** Structural configurations that combine low mass with a bending stiffness index below the indicated critical threshold.
- **Robust Typologies:** Multi-girder, Box, and U-girder decks are typically designed within stiffness ranges well above this critical threshold.

- **Portal Frame Bridges**

Portal Frame structures (both open and closed portal frames) are the most dynamically robust configurations, with vertical accelerations consistently staying below 2 m/s^2 .

- Worst-Case Parameter Combination: Maximum accelerations are virtually decoupled and independent of most structural and geometric parameters due to their high baseline monolithic rigidity and soil-structure interaction.
- Vulnerable Typologies: Although highly robust, a minimal critical stiffness threshold exists, below which peak accelerations could theoretically rise.

4.2 Practical use in train–bridge compatibility studies

The identification of these critical worst-case combinations and the development of the associated Random Forest predictive models provide several highly practical applications for railway infrastructure managers in train–bridge compatibility assessments:

- Instead of running computationally demanding finite element time-history analyses for every bridge, infrastructure managers can use the predictive model as a rapid filter. By inputting basic geometric and operational data, thousands of bridges can be scanned in seconds to flag structures operating in the critical worst-case parameter ranges.
- Engineering and computational resources can be strategically focused. Detailed finite element modelling and transient time-step calculations can be restricted only to the subset of bridges flagged during the fast screening stage, optimizing assessment costs and schedules.
- The predictive engine translates raw statistical predictions into safe engineering values by incorporating a typology-specific Root Mean Square Error offset to calculate a design acceleration (a_{design}):

$$a_{design} = a_{pred} + RMSE_{config}$$

For simply-supported spans, a conservative offset of 2.9625 m/s^2 is added, whereas continuous spans use an offset of 1.0213 m/s^2 , and frame structures use 0.0710 m/s^2 . This ensures that over 95% of real-world cases are conservatively bounded, minimizing the risk of underestimating dynamic accelerations.

- For train categories that fall within the envelope of the training train database, the dimensionless speed indicator ($\frac{V_{max}}{2Ln_0}$) acts as an efficient screening tool. It serves to efficiently determine which structures possess sufficient safety margins to support speed increases for these train classes.

5 Conclusions

The integration of statistical analysis and machine learning applied to the prediction of the dynamic response of a vast railway bridge population has led to the following conclusions:

- The preliminary statistical assessment confirms that peak vertical accelerations are highly dependent on structural typology and fundamental frequency. Simply-supported bridges exhibit the highest susceptibility to resonance whereas continuous and frame structures show more controlled responses due to structural redundancy and high bending stiffness, respectively.

- Linear regression models are insufficient for precise acceleration prediction ($R^2 \approx 0.50$). Their failure to account for non-linear jumps in acceleration near resonant speeds makes them unreliable for railway bridge engineering applications.
- The optimized RF model achieved high predictive accuracy ($R^2 = 0.877$). The inclusion of the dimensionless speed as a top-ranked predictor proves that physical feature engineering is essential for allowing black-box models to capture the underlying spectral limit states of the structures.
- Categorical analysis reveals that while the RF model is extremely precise for frame structures ($RMSE = 0.071 \text{ m/s}^2$), bridges with simply-supported spans remain the most challenging to predict due to their higher probability of experiencing significant response amplification.
- The proposed model may serve as an efficient screening tool for compatibility assessment. It allows for the rapid identification of at-risk bridges within large networks, enabling a predictive maintenance strategy.
- The dynamic response analysis shows that resonance vulnerability is mainly concentrated at low frequencies, where interaction with trains leads to higher accelerations. The relationship between mass, stiffness, and frequency is critical: low stiffness cannot be compensated by high mass, whereas high stiffness ensures good performance even with lower mass. Thus, low-frequency bridges can perform well if sufficient mass is present.
- For simply-supported bridges, performance varies by typology. Filler beam and half-through bridges perform poorly due to low stiffness and mass, while box-girder bridges perform well thanks to high stiffness. Other typologies show intermediate behaviour or could improve with increased mass or stiffness.
- The worst-case combination of critical parameters leading to maximum vertical acceleration (exceeding the 5 m/s^2 serviceability limit) is physically characterized by a simply-supported longitudinal configuration featuring a low mass per unit length ($m < 7079 \text{ kg/m}$), a low bending stiffness index ($EI/L^2 < 10^8 \text{ N}$), and a fundamental frequency operating within the critical resonant bandwidth of $5 - 12 \text{ Hz}$ under high dimensionless speeds ($V_{max}/(2Ln_0) > 0.3$).
- In continuous bridges, structural continuity significantly improves dynamic behaviour, expanding low-acceleration regions. A critical stiffness threshold exists below which serviceability cannot be met regardless of mass. Once exceeded, accelerations decrease sharply, making stiffness the dominant parameter, although mass remains relevant at low ranges.
- Finally, frame bridges exhibit the best overall performance due to their high stiffness and natural frequencies, resulting in the lowest acceleration levels and greatest dynamic robustness.
- The design application transfers the RF model by incorporating typology-specific $RMSE$ offsets to derive conservative design accelerations (a_{design}). This implementation transforms the predictive output into a safety-oriented envelope, providing a quantifiable margin that minimizes the risk of underestimation in real infrastructure assessments.

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