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Computational modeling of a composite railway bridge using soil-structure interaction

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Abstract. This study presents the development of a parametric geometry model and computational modeling strategies for a steel-concrete composite railway bridge located in Brazil. The asset features a reinforced concrete deck with a total width of six meters and a variable deck thickness ranging from 30 cm to 35 cm. A parametric model was implemented using the Rhinoceros/Grasshopper environment. Custom routines in GiD were developed to automate the finite element mesh generation, assign the material properties, and incorporate loads and boundary conditions into the model. Following this preprocessing stage, the model was integrated into CSiBridge platform to perform dynamic analyses and evaluate the railway bridge performance under service conditions. The bridge dynamic behavior was assessed through onboard monitoring techniques, which recorded train-induced responses during regular operation. A time-history dynamic analysis, coupled with influence surfaces, was conducted considering soil-structure interaction models applied to the railway bridge foundations. The computational model was then calibrated using experimental dynamic responses. Additionally, the wheel load information extracted during the monitoring campaign was incorporated into the simulations. The analyses identified the vibration modes and natural frequencies of the concrete components of the asset. Finally, a comparison between experimental and computational responses is presented, providing a foundation for future resilience studies of the railway bridge.

Keywords: finite element method, time history dynamic analysis, modal analysis, soil-structure interaction.

1 Introduction

The structural response to dynamic stress is largely influenced by the interaction between structure, foundation, and soil, according to studies carried out by Kamali [1]. Therefore, the so-called dynamic soil-structure interaction analysis must combine compatibility due to the structure-foundation-soil coupling to represent the propagation of waves generated by external dynamic loads.

Previous publications by the authors of this paper have demonstrated that modeling shallow and deep foundations of railway bridges and pedestrian bridges using impedance functions directly affects the dynamic behavior of the assets. According to Gamino et al. [2], considering a modal analysis via the Finite Element Method, the use of specific soil-structure interaction techniques produced a maximum deviation of 3% in natural frequencies, while the non-use of the technique reported frequency dispersion values of around 25%, comparing the computational results with the experimental ones found in an instrumentation and monitoring campaign. These values were obtained for deep foundations of the caisson type on a railway bridge located in Brazil and built in the 1980s.

For a pedestrian bridge built on shallow foundations, the computational model produced by Gamino et al. [3] using soil-structure interaction laws achieved an excellent correlation with the natural frequencies obtained in an experimental campaign with a maximum deviation of 8%.

According to Shaik and Chandradhara [4], during the foundation design phase, it is crucial to use the soil-structure interaction model since shear forces can be evaluated in a way that is closer to real conditions, which can lead to a reduction in the amount of reinforcement bars used; additionally, they concluded that not considering the

soil-structure interaction can lead the asset to a performance, for the seismic loads, with a reduction of approximately 40% to 60%.

Considering these technical notes, this article presents a methodology for considering the effects of soil-structure interaction in deep foundations on steel piles used in a railway bridge in Brazil located in the northern region of the country. To validate computational responses, data obtained during an instrumentation and monitoring campaign of the assets were used. The vibration modes obtained for modal analysis and the dynamic response in the railway infrastructure region were compared, considering train traffic on the asset.

2 Railway bridge description and computational model

This is a composite railway bridge built in 2014 and located in northern Brazil. The analyzed asset has a total length of 550 m distributed in 25 m spans. The superstructure consists of a reinforced concrete slab supported by steel frame girders. The columns are reinforced concrete box-sections, and the foundations are composed of steel piles. Fig. 1 illustrates details of the construction system.



(a) (b)

Figure 1. Asset configuration: mid-span detail (a) and abutment region (b)

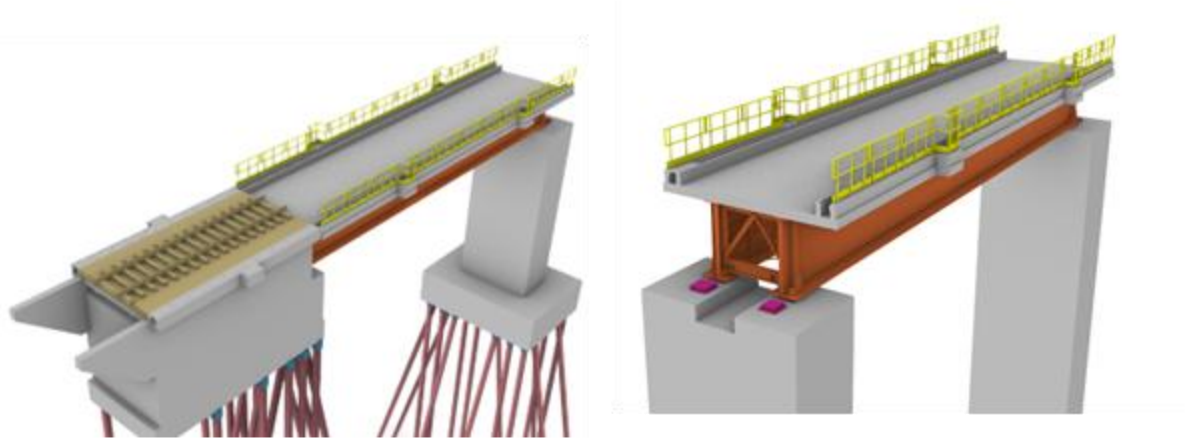
2.1 Railway bridge geometric model and preprocessing

A parameterized geometric model of the asset was developed using Rhinoceros 7 available in McNeel [5] and the functions available in Python in Grasshopper. Fig. 2 shows the rendered construction details of the railway bridge under study.

After creating the geometric model, mesh objectivity studies were carried out on parts of the railway bridge with the aim of reducing computational costs while maintaining the accuracy of the results. Thus, the asset parts were exported in ".3dm" format to the GiD 15 interface, developed by CIMNE [6], for generating the finite element mesh and applying the attributes containing the materials, loads and boundary conditions. In this aspect, GiD 15 customization routines were created for the pre-processing stage so that the computational model could be used in the CsiBridge 21 [7] interface through the ".b2k" extension.

The computational model produced refers to the section instrumented by the University of São Paulo team containing an abutment and two 25 m spans as illustrated in Fig. 3. The abutment, columns and pile caps were modeled with solid three-dimensional finite elements, the foundations and lower regions of the deck in beam elements and the deck was modeled with shell finite elements.

A lane was applied to the deck so that the moving loads originating from the railway composition (vehicle) were applied to the asset. The composition of the railway vehicle followed the values obtained in the instrumentation campaign by obtaining the wheel loads on the rails.



(a) (b)

Figure 2. Developed parameterized geometric model: abutment detail **(a)** and mids-pan region **(b)**

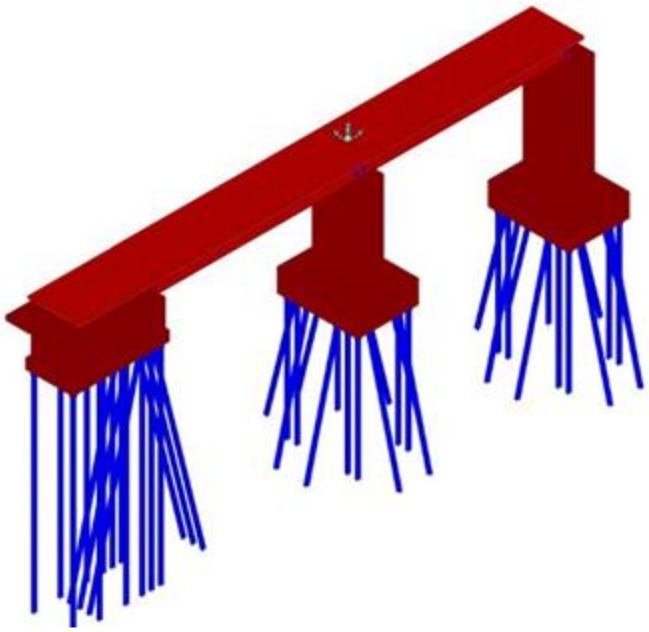


Figure 3. Computational model of the railway bridge in the CSiBride 21 interface

2.2 Computational setup

The loads were applied per loaded axle and distributed across two wheels in contact with the rails (two wheels per axle, in according to Fig. 4). The dynamic analysis employed was of the linear "time history" type, using a modal solution, with 300-time steps subdivided into proportions of 0.1 s each (total analysis of 10 s) and a constant damping coefficient equal to 3%. The vehicle speed of 60 km/h was used.

Additionally, a modal eigenvalue analysis was performed to identify the vibration modes and their respective frequencies. The model was calibrated by comparing the experimental and computational natural frequencies. Finally, the temporal accelerations in the pile caps obtained from the computational model were compared with the response obtained during the monitoring campaign.

Finally, Winkler springs were applied to the steel piles and abutments to account for the effect of soil confinement. The Morrison [8] model was used for the foundations, as described in section 2.3.

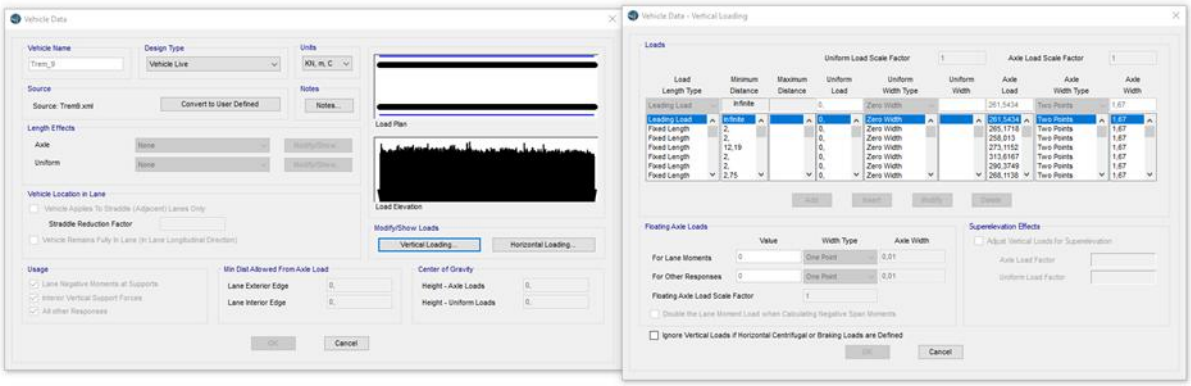


Figure 4. Composition of the railway vehicle according to the experimentally extracted wheel loads

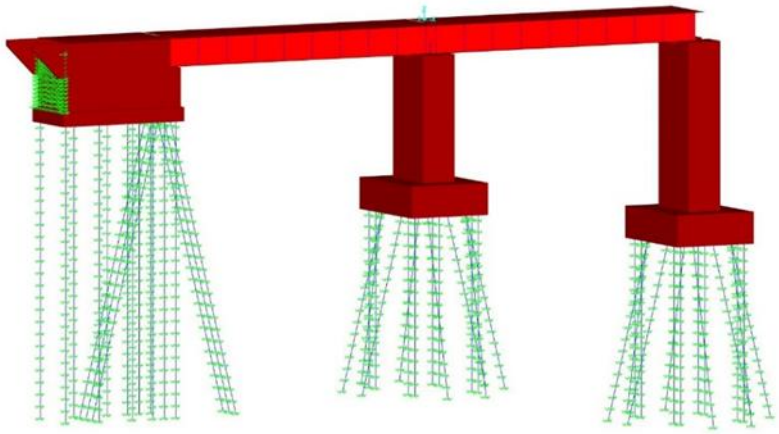


Figure 5. Application of Winkler springs on metal piles and at the abutment

2.3 Soil-structure interaction model

Basically, the horizontal stiffness corresponding to soil confinement proposed by Morrison [8] can be determined by:

$$K_H = 2\sigma_{ad}\nu SD \tag{1}$$

where K_H corresponds to the horizontal stiffness of the Winkler spring, σ_{ad} represents the allowable soil stress, ν the Poisson's ratio, S the distance between springs (finite element mesh discretization) and D represents the diameter of the piles.

Finally, the stiffness of the springs in the abutment region was calibrated according to the natural frequencies obtained experimentally during the instrumentation and monitoring campaign.

2.4 Results

The properties of the materials used in the computational model are shown in Table 1. For the soil, Poisson's ratio values were adopted according to the allowable stress, varying between 0.2 and 0.4. The piles have an "H" type cross section and were therefore converted to an equivalent diameter of 0.141 m.

Table 2 presents the computational natural frequencies compared with the experimental ones obtained from accelerometers installed on the asset for power spectral density. Fig. 6 shows the configuration of the second vibration mode found from the modal analysis in the computational model of the asset. The computational results obtained demonstrated accuracy in relation to the values found during the instrumentation and monitoring campaign of the analyzed railway bridge.

Table 1. Materials and cross-sections applied for the computational model

Region	Material properties	Stiffness/Cross-section
Deck	f_c = 35 MPa. E = 31 GPa	e_{shell} = 30 cm
Meso and infrastructure	f_c = 30 MPa. E = 29 GPa	Solid FE
Superstructure	f_y = 350 MPa. E = 210 GPa	Beam FE
Infrastructure	f_y = 250 MPa. E = 210 GPa	Beam FE
Soil	σ_{ad} = 40 kPa to 1000 kPa	65 tf/m to 1128 tf/m

Table 2. Natural frequencies obtained for the railway bridge

Vibration mode	Experimental Hz	Computational	Deviation %
I	4.4	4.0	10
II	9.2	9.3	1.1
III	11.7	10.4	12.5

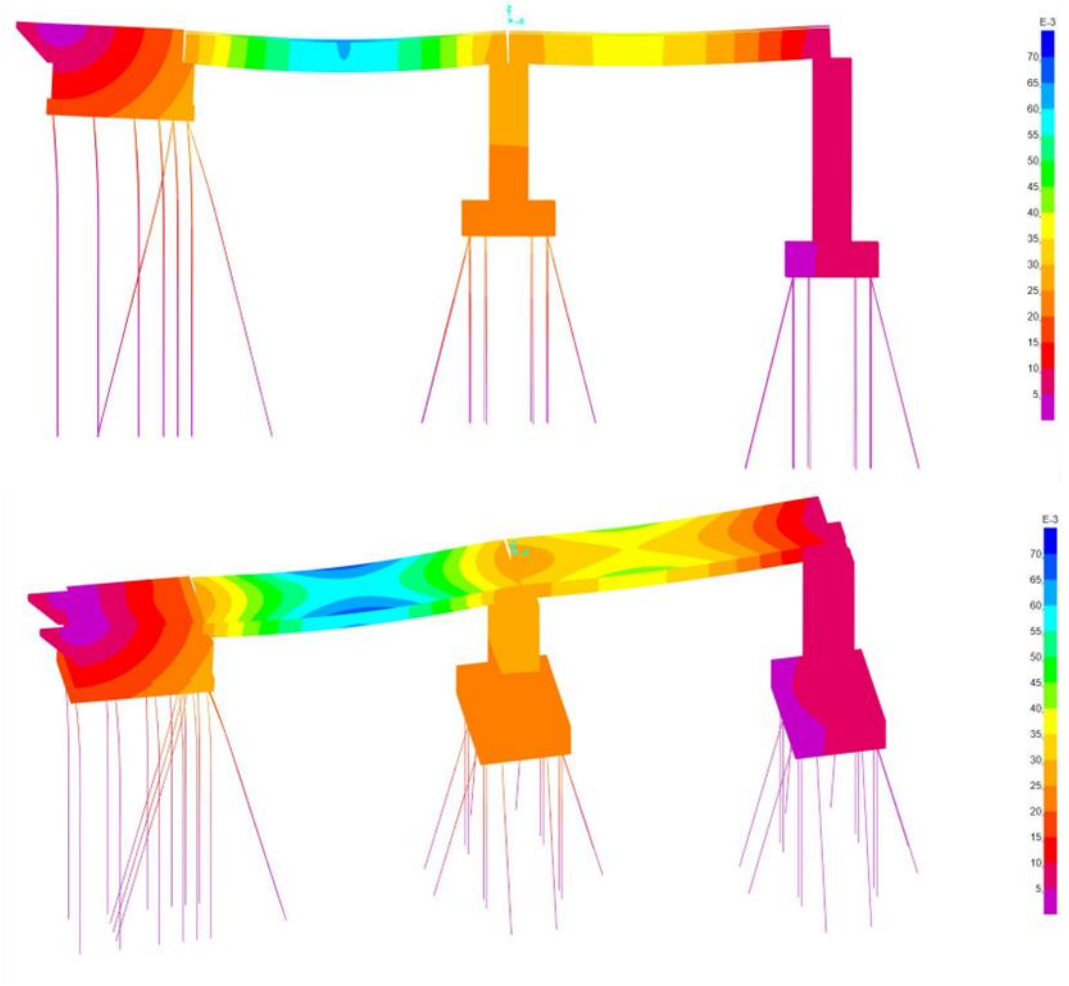
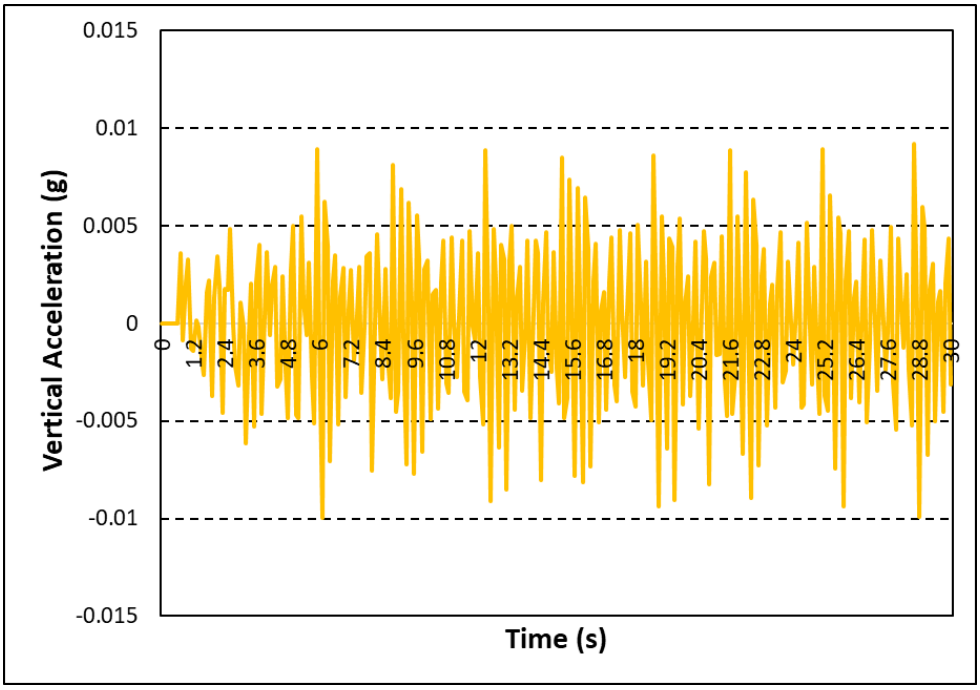
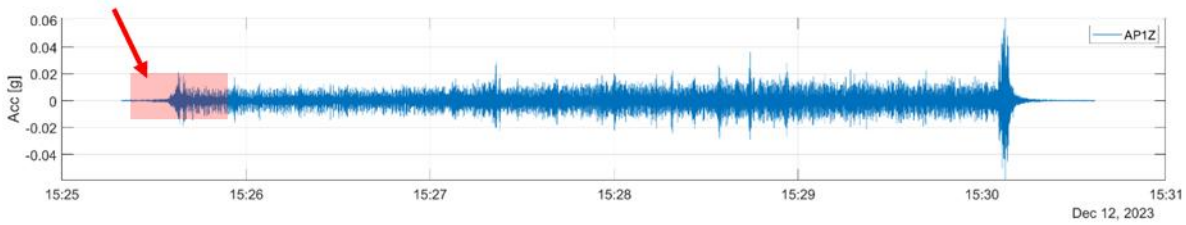


Figure 6. Deformed configuration of the second vibration mode (total displacements in "m")

Fig. 7 illustrates the temporal variation of the vertical accelerations obtained in the pile caps. The forced vibration sections with average values of 0.01g for the experimental and computational series can be seen, demonstrating agreement between the results obtained.



(a)



(b)

Figure 7. Vertical accelerations in pile caps: (a) computational and (b) experimental

The instrumented pile caps demonstrated the same values of vertical accelerations and, therefore, there was no occurrence of anomalies and defects in such elements of the railway infrastructure. The experimental dynamic response demonstrated the occurrence of acceleration spikes of 0.06g that may be caused by the load distribution conditions in the wagons or even the mechanical conditions of the railway vehicle.

3 Conclusions

Considering the results obtained in this work, it is possible to conclude that:

- The use of soil-structure interaction models is essential to obtain computational dynamic responses reliable to the behavior of the asset for in-service loads.
- The dynamic modeling technique employed resulted in maximum variations of 12.5% on the natural frequencies of the analyzed railway bridge.
- The computational model produced time series of vertical accelerations in the pile caps with accuracy compared to the experimental results.

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Authorship statement. The authors hereby confirm that they are the sole liable persons responsible for the authorship of this work, and that all material that has been herein included as part of the present paper is either the property (and authorship) of the authors.

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