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Static and dynamic analysis of a reinforced concrete rail bridge located in the Carajas Railroad

Análise estática e dinâmica de uma ponte ferroviária em concreto armado localizada na Estrada de Ferro Carajás



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Abstract

This work presents results of static and dynamic analysis carried out on 593 m length bridge along Carajás railroad in Maranhão state. The bridge's behavior became important due to some changes over the static and dynamic loads once the future train will be heavier. Thus, a mathematic model was developed using the finite element method basis to evaluate its behavior and "in loco" structural monitoring to get data as strains and vibrations when the trains go through. Several high precision accelerometers and strains gauges were placed along the slab, beams and columns. Numerical and experimental results are shown and the main conclusions validating the applied methodology are presented as well.

Keywords: bridge, monitoring, vibration, finite element.

Resumo

Neste trabalho são apresentados os resultados de análises estáticas e dinâmicas realizadas na Obra de Arte Especial número 40 (OAE 40), localizada na Estrada de Ferro Carajás. A obra consiste em uma ponte de concreto armado com 593 m de comprimento, no km 510 da ferrovia, estado do Maranhão. O estudo do comportamento da estrutura veio a assumir importância crescente frente às perspectivas de alterações das ações dinâmicas, que passarão a considerar trens-tipo operacionais com carregamento superior aos atuais. Assim, fez-se uso de um modelo matemático utilizando o método dos Elementos Finitos para avaliar seu desempenho e do monitoramento estrutural "in loco" das deformações e vibrações produzidas quando da passagem dos trens. Foram realizados diversos arranjos com acelerômetros piezoelétricos no tabuleiro, além de extensômetros elétricos de resistência no tabuleiro, longarinas e pilares. São apresentados os resultados numéricos e experimentais, bem como as conclusões relevantes que corroboram a metodologia empregada.

Palavras-chave: ponte, monitoramento, vibração, elemento finito.

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1. Introduction

Placed at the North Area of Brazil, the Carajas Railroad (EFC) is used to transport iron ore from Carajas city (Para State) to Itaqui harbor in Sao Luis city (Maranhao State). In a close future this railroad will receive greater loads and more accuracy studies about its bridge structures were necessaries.

1.1 Rolling loads

The transportation along the EFC uses compositions with locomotives (LM) type DAS9 and GDT wagons (WG). Two types of compositions are been more applied, one with 2 head locomotives and 104 wagons followed by more 1 locomotive and 104 wagons, representing 80% of the trains working nowadays. Sometimes the intermediary locomotive comes at the end of the train. The second composition presents 3 locomotives together pulling 208 wagons. Figures 1 and 2 show the longitudinal dimensions considered for these locomotives and wagons, respectively.
In a close future the EFC will be loaded by new and heavier trains. Also the number and the individual capacity of the wagons will be increased. The future trains will present 2 LM + 110 WG + 1 LM + 110 WG + 1 LM + 110 WG or 2 LM + 110 WG + 2 LM + 220 WG. Table 1 presents the current and future loads considering DASH9 locomotives and GDT wagons.

1.2 General description of the bridge

The reinforced concrete bridge named OAE 40 crosses the Cajuapara river on the kilometer 510 of the EFC railroad. With 593 m length this structures presents 22 statically indeterminate spans with 25 m each one and two longitudinal beams linked by transversal beams measuring 300 mm width at intermediary positions and 500 mm near the columns. The longitudinal beams present rectangular cross sections with 350 mm and 700 mm width far and near the columns, respectively. The bigger width starts with 350 mm and varies linearly to 700 mm in a length of 3.95 m. Figures 3 and 4 shows the structural elements of the cross section and a typical span with columns and longitudinal and transversal beams.
The bridge's columns present equals H cross sections on top and variable web along the height in 4% till their basis which are monolithically linked to the foundations blocks with 2 m height. The number 5 column (Sao Luis – Carajas way) presents the most slender presenting 34.6 m height. All columns also present the upper length without web. The last beams finally are supported by contention cells with four transversal walls and other two to enclosure crushed limestone. The total longitudinal dimension of the abutment is 21.5 m. Figure 5 shows dimensions and details of the columns.

Figure 1 – Wagon type GDT

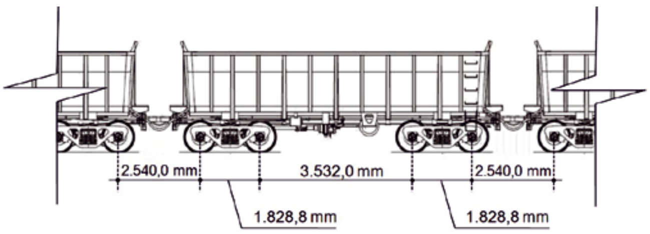


Figure 2 – Locomotive type DASH9

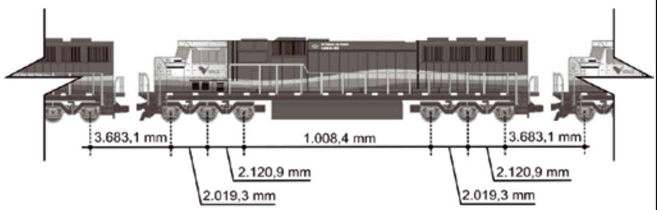
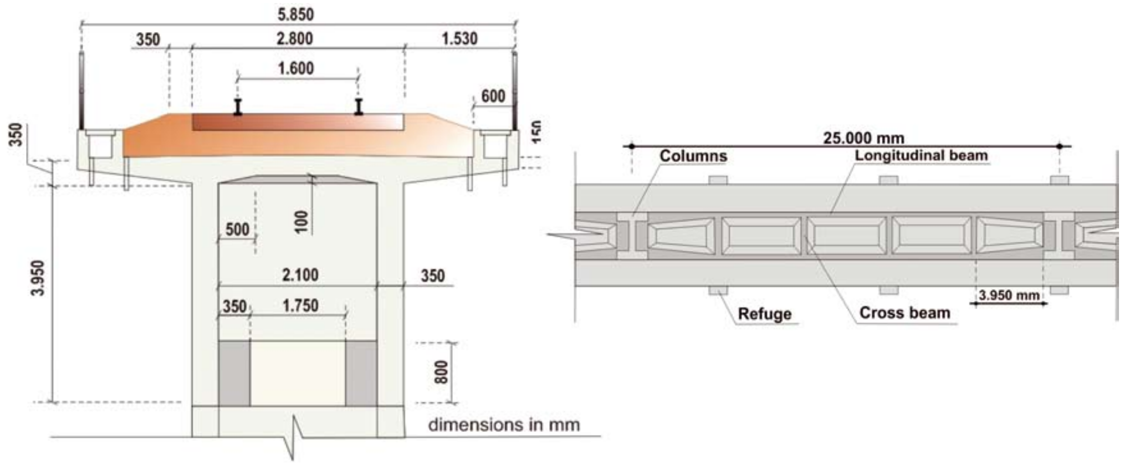


Figure 3 – View of the bridge

Table 1 – Loads of locomotives and wagons

Situação	Locomotive DASH9	Full GDT wagon	Empty GDT wagon
Current	300 kN/axle (1,800 kN)	325 kN/axle(1,300 kN)	52.5 kN/axle (210 kN)
Future	300 kN/axle (1,800 kN)	400 kN/axle (1,600 kN)	52.5 kN/axle (210 kN)

Figure 4 – (a) Cross-section of the superstructure; (b) Typical span of the bridge



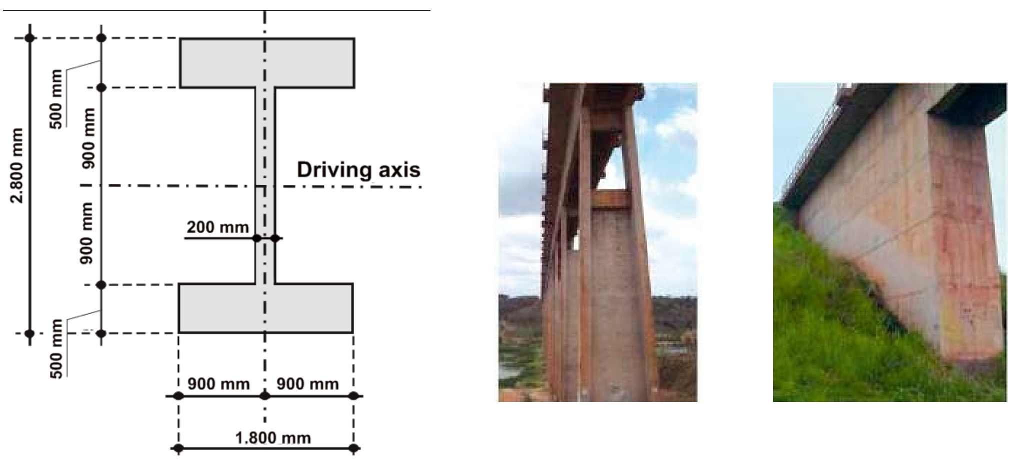
(a) (b)

2. Computational model of the structure

The bridge was modeled using the software SAP2000®[1] according to the design memorial hypothesis of the structure and after some implementations were considered aiming to simulate foundations, abutments and the new materials properties from the "in situ" tests. The cross section with slabs, beams and columns were numerically represented by frame elements. The columns were considered with fixed ends and the foundation blocks were modeled using solid elements. The abutments were simulated using plate elements (*shells*) and the elastic support devices (reinforced rubber) using spring elements with appropriate stiffness against normal, tangential and rotational forces calculated according to Pfeil [2]. To validate comparison between theoretical and experimental results a f_c = 46.0 MPa from "in situ" tests was adopted to slabs and columns, and 48.0 MPa and 38.0 MPa for abutments and foundations blocks, with E_c = 41,000.0 MPa and E_c = 34,000.0 MPa, respectively, according to 5739: 1994 [3] and NBR 8522: 2003 [4]

Due to the minimum interference of the soil over the foundations elements in deeper layers it adopted only 4,500 mm as interference zone from the top surface of the ground, i.e. a well fixed length according to Pfeil [5]. Along this length several springs were placed in the three main directions, with different stiffness from the design memorial, simulating the mechanical soil influence, following the recommendations of Anjos [6]. Such proceeding was based on the Winkler hypothesis [7], which considers the soil stiffness as

Figure 5 – (a) Columns cross-section; (b) Columns view; (c) Partial view of the encounter E2



(a) (b) (c)

Figure 6 – Computational model using finite elements

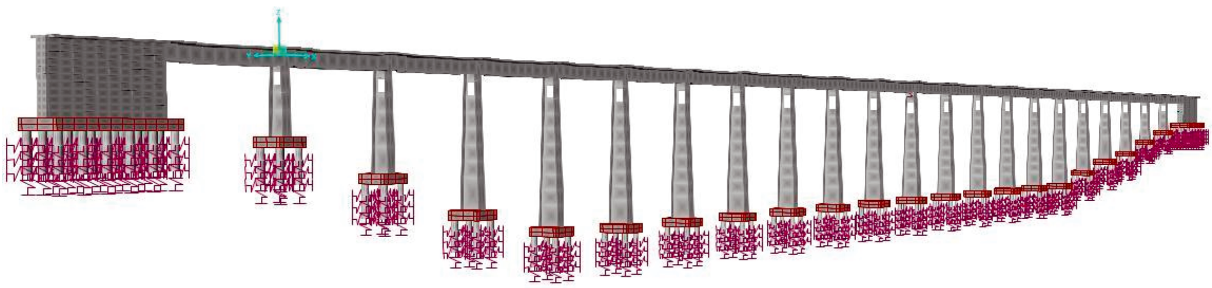


Figure 7 – Data acquisition system used for monitoring tests

a linear relationship between stresses and strains at a single point. Some little adjusts and implementations over the mass and inertia were done later aiming the modal analysis. Figure 6 shows the finite element model to study the static and modal behaviors of the bridge structure.
The vertical and horizontal rolling loads were those from the future and current compositions. The effect of the vertical rolling load along the bridge was evaluated using the impact coefficient [2], prescription of NBR 7187: 2003 [8] and described by Equation [1] where *l* is the theoretical span between the supports.

3. Strains and vibrations monitoring

Several subsystems were applied in the experimental stage, e.g. readings, data acquisition, communication, treatment and evaluation of the results, trying to obtain the variables of interest with minimum deviations, according to Mufti [9]. A communication wireless web was set *"in loco"* enabling strains and accelerations readings in strategic points of the structure. Static tests were carried out considering loads at some predefined positions and the ADS2000 AC2122 equipment was finally installed, as shown on Figure 7, with the loaded train coming from Carajas, according to the positions showed on Figure 8. The dynamic tests considered the normal velocity (100% V) or fifty percent of it (50% V).

3.1 Extensometria no pilar P20

The measured strains of the P20 column (from Carajas) were taken under several loads conditions. The cross section received the strain gages E1, E2, E3 and E4 on the concrete surface far 1.5 m from its base, as shown on Figure 9.

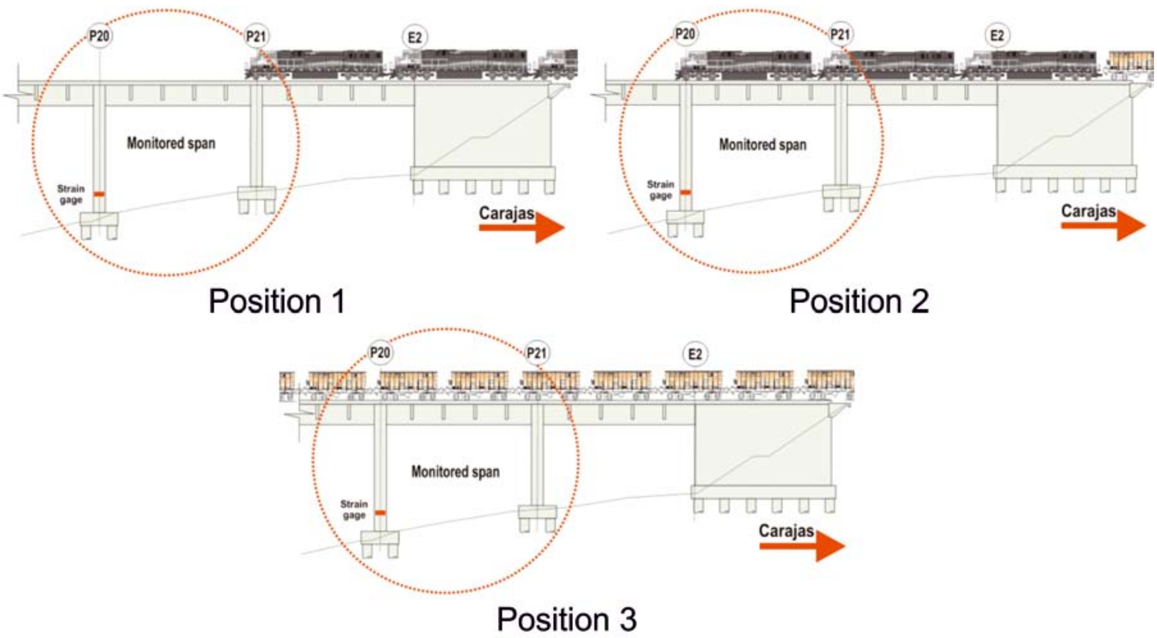
3.2 Extensometria no tabuleiro e longarina

The slabs received strain gages at two different points or sections between columns P20 and P21. One of them was placed over the left longitudinal beam (LC) (from Carajas) and a second one close the same side edge (CAN) to verify the compatibility between the concrete strains. The longitudinal right beam had one bar of its bottom flexural reinforcement monitored only (LA). Figure 10 shows all the strain gages position. The strains measured were used to estimate the stresses along the beams according to the live load on the bridge.

3.3 Vibração do tabuleiro

Sixteen piezoelectric accelerometers (low frequency) (ICP) were used to measure vibrations on the slabs. Nine arrangements of accelerometers were applied and individually tested to monitor all the

Figure 8 – Load cases for static tests



slabs under rolling loads (Figures 11 and 12). Vertical, horizontal and transversal directions received accelerometers and the temporal series were then known. These results from each arrangement were treated together as under the same loading, i.e. full or empty wagons along the trains.

Using the concept of reference accelerometers proposed by Peeters et al. [10] and Amador [11] the partial modal forms from each arrangement were grouped in order to find the complete modal from of the structure. In these nine arrangements the accelerometers were set to determine the first vibrations modes. Five accelerometer were adopted as reference and placed on the sections A and B in the arrangements along the adjacent length to Sao Luis, and for sections A1 and B1 the length monitored was adjacent to Carajas side.

4. Results

4.1 Strains on the column P20

In order to determine the acting and reacting forces from the measured strains in the structural elements a yield stress of 500.0 MPa was used for all reinforcements and a typical behavior as recommended by NBR 6118 [12] for steel bars with yield well defined. The properties of the concrete came from laboratorial tests over proofs extracted from the bridge structure. To compare theoretical and experimental results the higher strain values from the temporal curve were separated once the forces associated to these values should be close of those from the computational analysis, using the software Matlab® [13]. This procedure is worldwide known as Fellenius method and well applied by Stramandinoli [14] Khouri

Figure 9 – Strain gages position in the column: (a) Lateral view; (b) Cross-section

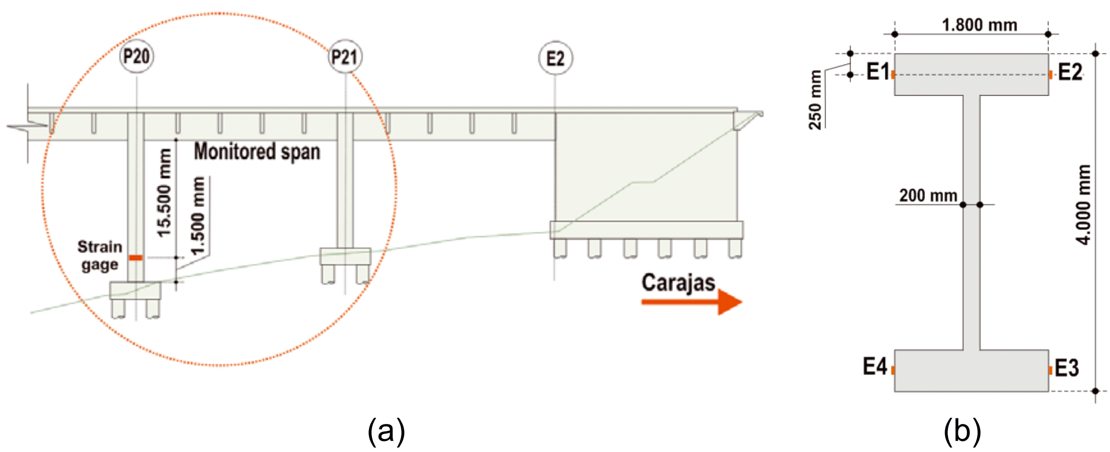
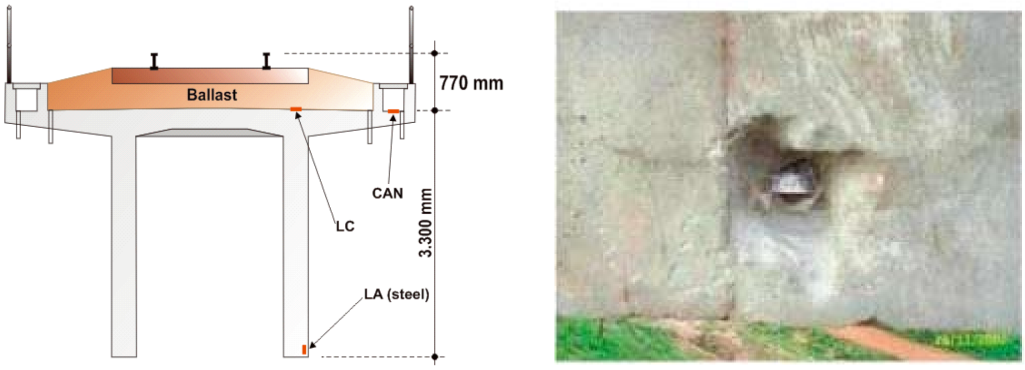
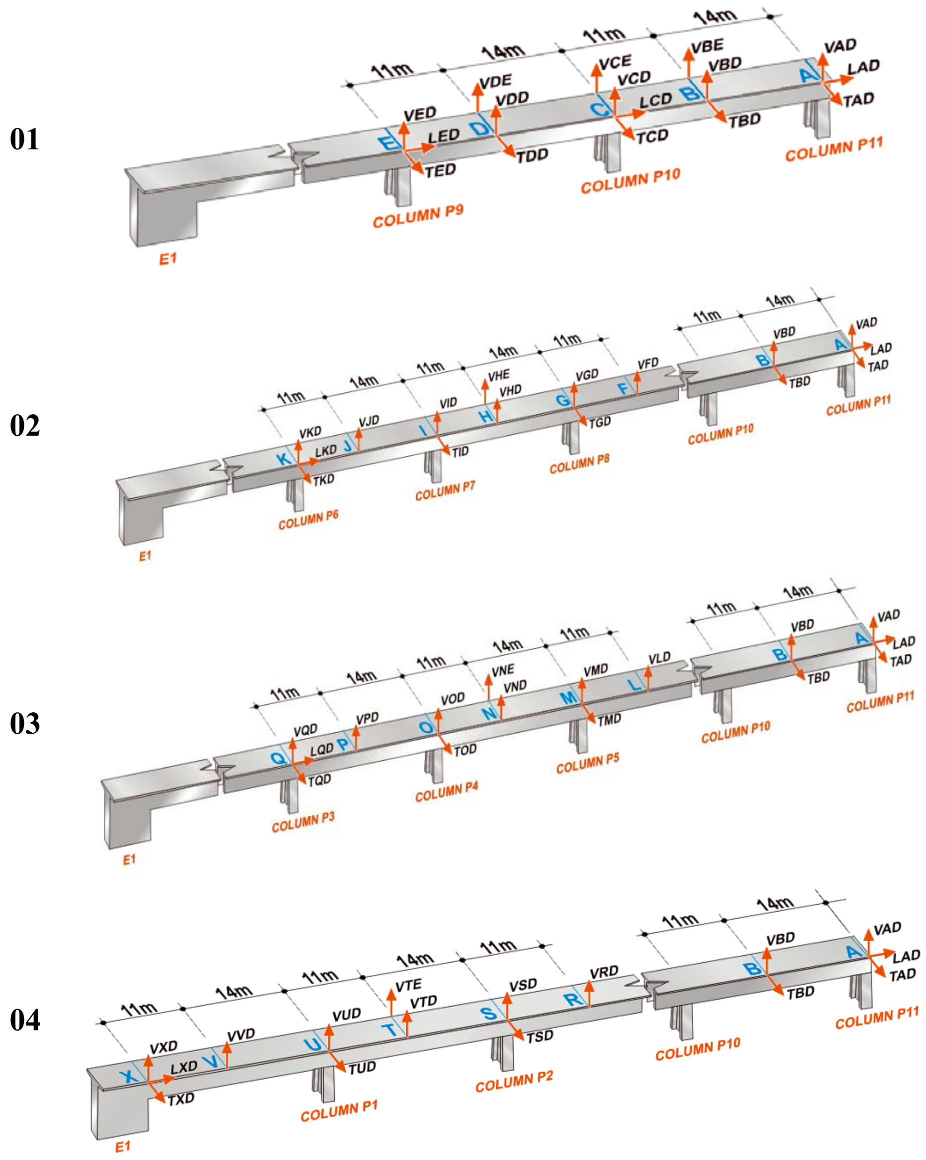


Figure 10 – (a) Position of strain gages on the superstructure; b) LA sensor already installed



(a) (b)

Figure 11 – Accelerometers in the arrangements 01, 02, 03 and 04



[15] and Rego [16], enabling to simulate the plastic behavior of the concrete elements.

Figure 13 shows the experimental strains for P20 column and the theoretical ones for two sensors, considering the actual composition with empty and normal speed. It can be also seen the effect of the impact coefficient □ of 1.356. Figure 14 shows the same results for loaded wagons and normal speed. The cross section of the P20 column was 2,400,000.0 mm^2. Table 2 presents the main characteristics of the monitored column.

The structural answers using finite element method were compared to the dynamic test ones but applying a pseudo-static analysis previously, and applying the vertical impact coefficient in order to consider the inertia effects of the loads. The strains used to calculate the forces over the column are presented on Table 3. Table 4 presents the forces from the experimental strains with loaded wagons. Figure 15 shows these results graphically and those from MEF analysis using the same loads.

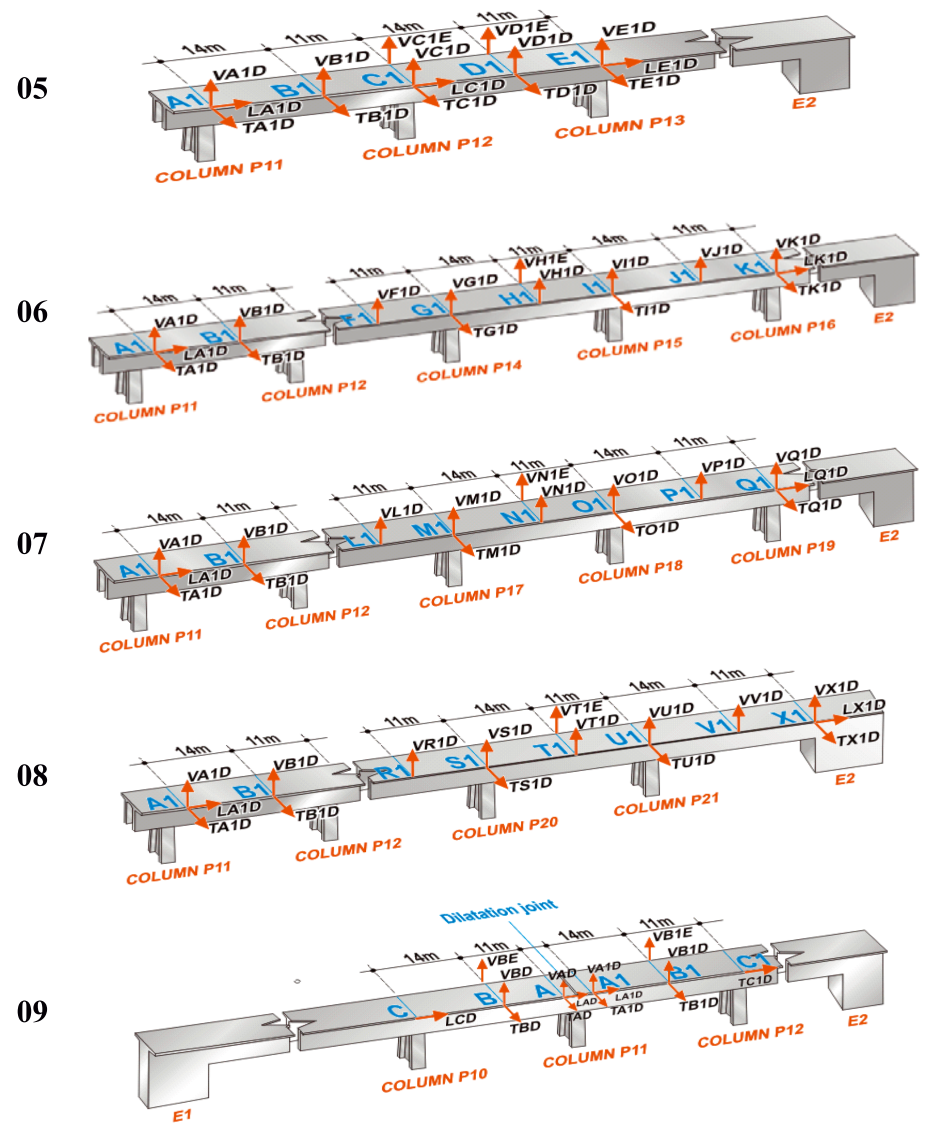
4.2 Strains on the slabs and longitudinal beams

The left longitudinal beam (Sao Luis-Carajas way) had its midspan between P20 and P21 monitored. All strains were very low for the compression block of the slabs. Figure 16 shows the flexural reinforcement strains under the loaded train passage and normal speed. The used height was 3,100 mm and the concrete compressive strength was 46.0 MPa. Table 5 presents the results during the static tests with loaded train and Table 6 brings the calculated forces without consider the self weight of the structure. A results comparison is shown on Figure 17.

4.3 Comparison between dynamic tests with different speeds

The results showed that no relevant differences were found even when the speed of the train was reduced to 50 %. Figure 18 shows the strains on the P20 surface during the head locomotives pas-

Figure 12 – Accelerometers in the arrangements 05, 06, 07, 08 and 09



sage. The reduced speed did not decrease the strains significantly ant the same behavior was observed for normal speed.

4.4 Vibrations

The accelerations recordings lasted 2,5 minutes and used the time frequency of 400 Hz domain. The results from each arrangement were processed to remove interferences which consequences do not allow satisfactory interpretations and identifications of parameters. The next process was a Chebyshev type I low-frequency filtration with a cut frequency of 10 Hz. After that the parameters were identified using Stochastic Subspace Identification methodology based on the covariance of the reference answers – SSI-COV (*Covariance-driven Stochastic Subspace Identification*) – through a software called SISMEC developed on the Matlab® platform with all algorithms presented and tested by Amador [11]. According to Peeters [17] this technique can be applicable to identify nodal parameters of civil structures without know the excitation forces, as commonly done.
Although time-history series were short it was possible to identify several vibration modes of the structure in each arrangement. Figure 19 shows a example of time-history series for empty wagons ant the correspondent spectrum from accelerometer placed on section A of the arrangement 01. Software SISMEC was used to turn the time domain into the frequency domain using mathematical tools as Fourier solutions for each sensor.
Figure 20 shows the stability diagram, according to Allemang [18], using the SSI-COV methodology over the treated series. This diagram is a practical way to get modal parameters varying the stochastic model degree. For each value natural frequency, damping and vibration mode were calculated and compared to those in the previous order checking the acceptability range established by the user for stability [11]. When the stability level is satisfactory for all parameters (frequency, damping and vibration), the symbol ⊕ is attributed, and as many symbol like this is found in the columns higher will be the probability to find the natural frequency and the associated mode can represent a true dynamic signature of the structure. The blue circle marked points refer to the chosen order by the user.
After parameter identification for all nine arrangements it was possible to join the vibration modes of them. The applied strategy was find the same vibration mode in all arrangements, compare to the modal amplitude from the reference accelerometer, and establish a correlation between modal vectors using minimum square regression. It's a simple manner to get a scale factor between reference amplitude values and normalize the whole vector. The modal forms were analyzed using the Modal Assurance Criterion (MAC), according to Allemang [18], where the identity between the analyzed modes is satisfactory when close to 1. Figure 21 shows a comparison between the modal forms from arrangements 01 and 02. It is observed that the MAC values near to 1 which were the vibration modes repeated for two arrangements and the associated frequency. Table 7 presents results from the computational and experimental analysis for three vibration modes after combination of the modes from each arrangement. Comparing these results it is clear that the studied vibration modes are similar. Figure 21 shows this similarity between results. It must be remarkable that the modal forms derived from auto-vectors and was not possible compare the amplitude values of displacements.

Figure 13 – Observed strains on the column P20 for loaded wagons and normal speed

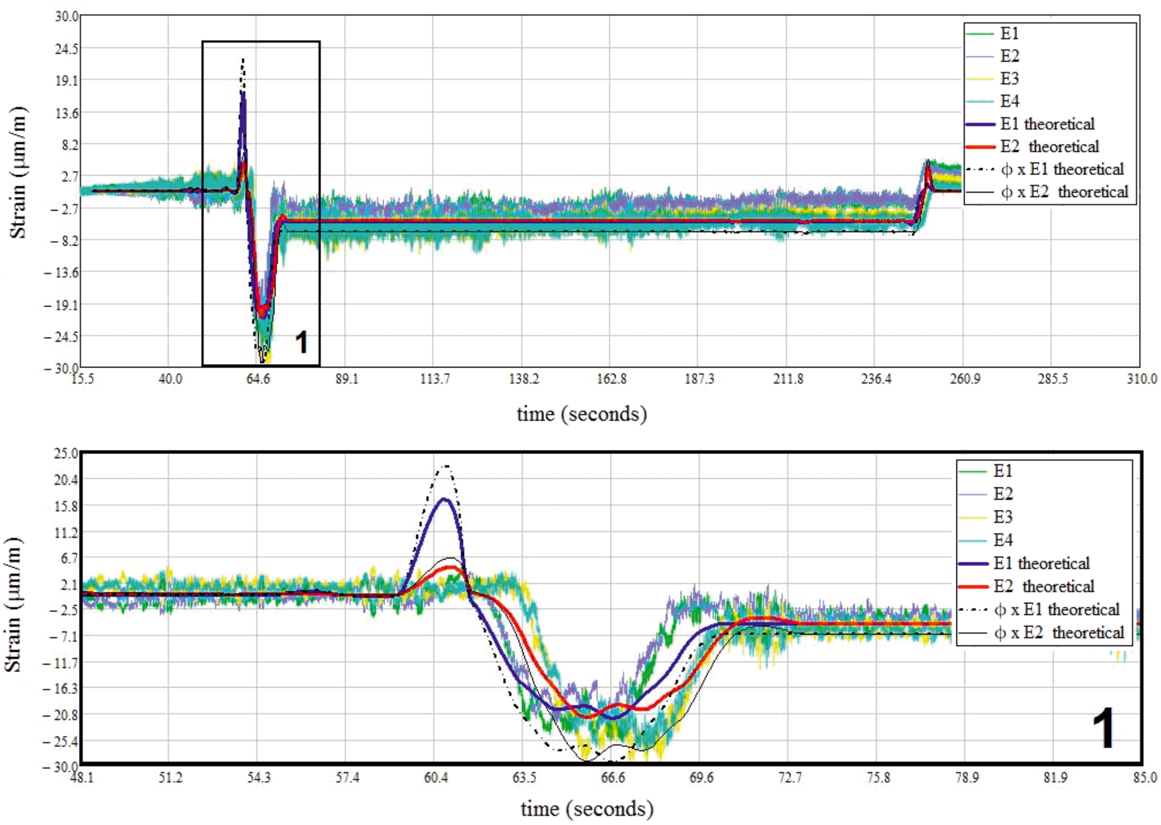


Figure 14 – Observed strains in on column P20 for loaded wagons and normal speed

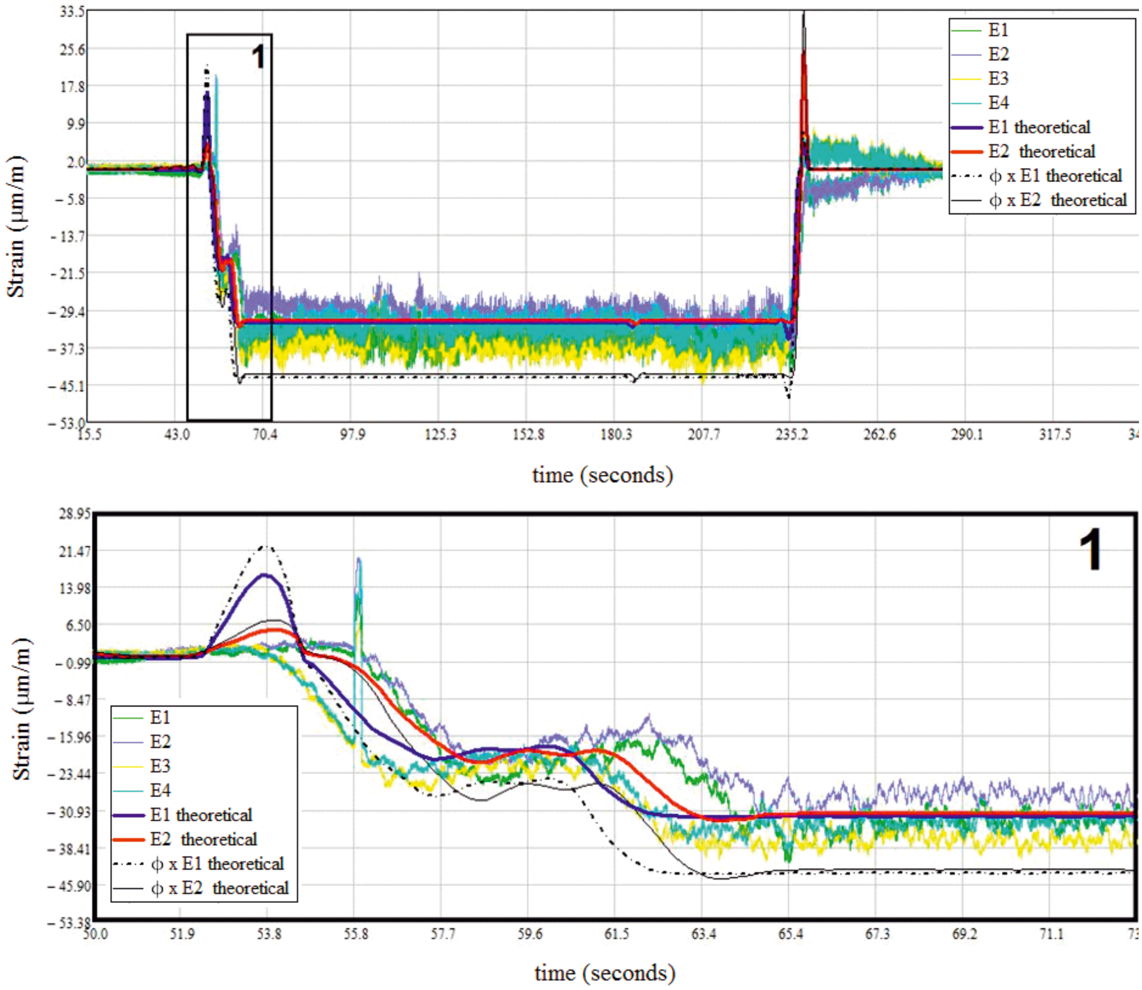


Table 2 – Main characteristic of the column monitored					
Column	A_c (mm²)	Rebars	A_s (mm²)	E_s (MPa)	f_c (MPa)
P20	2,400,000.0	84∅25.0 + 58∅12.5	48,349.6	210,000.0	46.0

Table 3 – Maximum strains on the column P20

Loaded operational train-type			
Static Test			
Sensor	Strains (‰)		
(EER)	Position 1	Position 2	Position 3
E1	+0.0012	-0.0180	-0.0350
E2	+0.0010	-0.0120	-0.0320
E3	+0.0012	-0.0100	-0.0350
E4	+0.0010	-0.0100	-0.0350

Table 4 – Axial force calculated from the experimental strains on the column P20

Loaded operational train-type			
Static Test			
Column	Axial Force (kN)		
	Position 1	Position 2	Position 3
P20	100.0	-1,132.0	-3,103.0

Figure 15 – Numerical and experimental results for P20 column

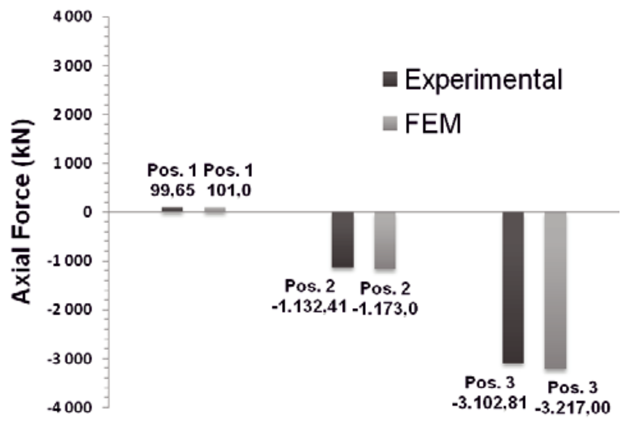


Table 5 – Maximum strains on the monitored longitudinal beam

Loaded operational train-type			
Static Test			
Sensor (EER)	Strains (‰) Position 1	Position 2	Position 3
LC	+0.0100	-0.0150	-0.0200
LA	-0.0350	+0.1200	+0.2100
CAN	+0.0100	-0.0120	-0.0200

Table 6 – Bending moments from experimental strains on the longitudinal beam

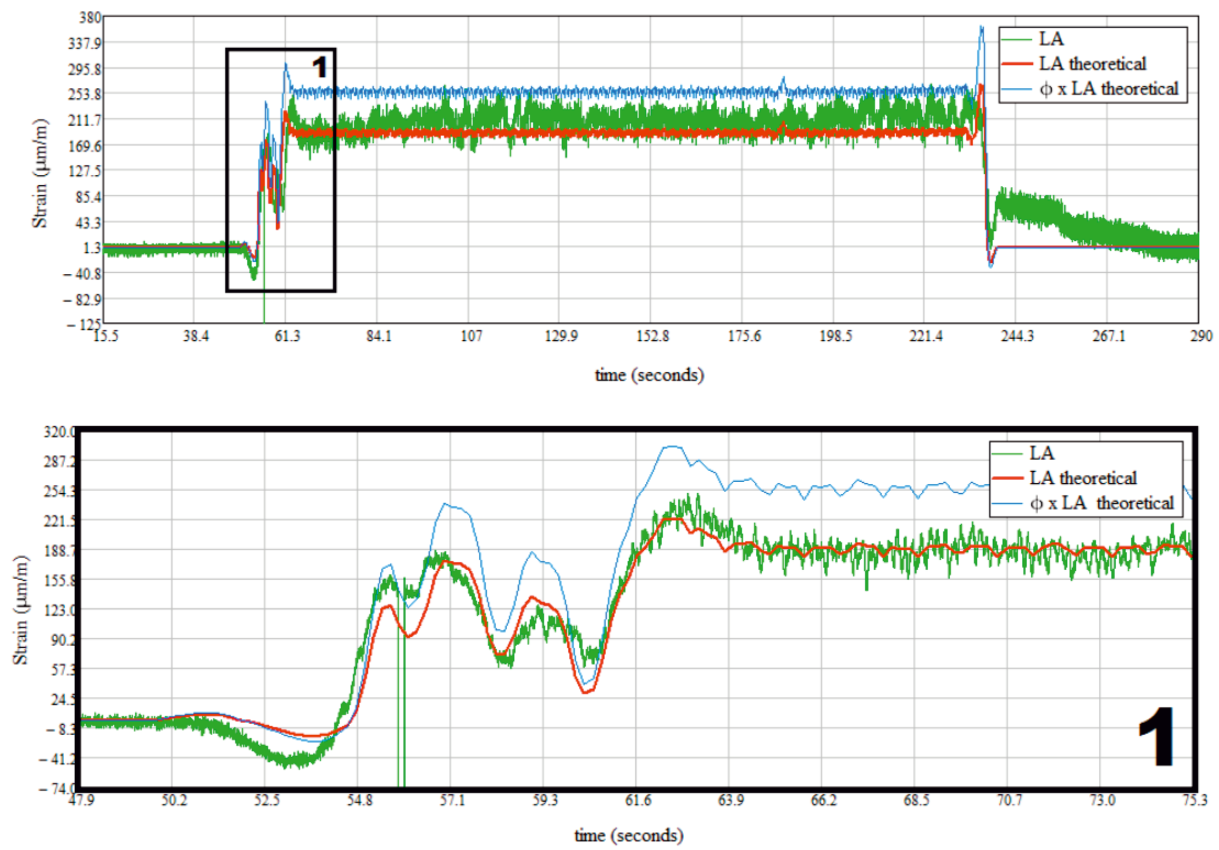
Loaded operational train-type			
Static Test			
Beam	Moment (kN.m) Position 1	Position 2	Position 3
P20-P21	-237.0	1,013.0	1,458.0

5. Conclusions

5.1 Monitored column

Strain and forces low values agreed with the computational ones enabling to state that the structure´s behavior is almost elastic. The impact coefficient of 1.356 is initially a underestimated value due to the small amplification when static and dynamic values are compared, but when this value is used over the theoretical strains it gives a mean curve involving the experimental results, leading to satisfactory results over the dynamic effects

Figure 16 – Observed steel strains in the longitudinal beam for loaded wagons and normal speed



for this study. In relation to the static test it was observed that the higher axial force was for position3 loading with the wagons on the bridge only. The experimental strains for unloaded trains were around 65 % of those with loaded train for dynamic tests and normal speed.

Figure 17 – Numerical and experimental results for longitudinal beams

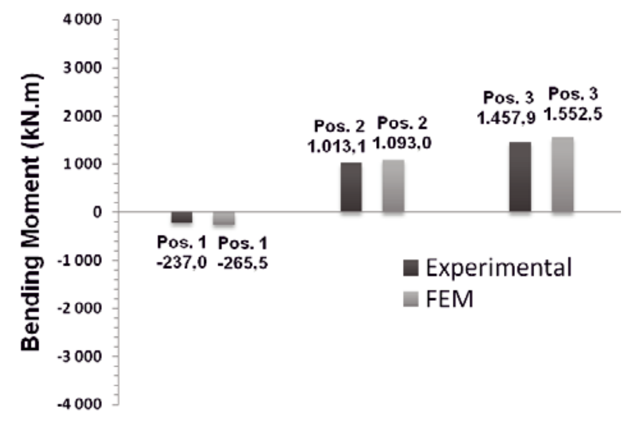


Figure 18 – Tests comparison for P20 column during leading locomotives passage

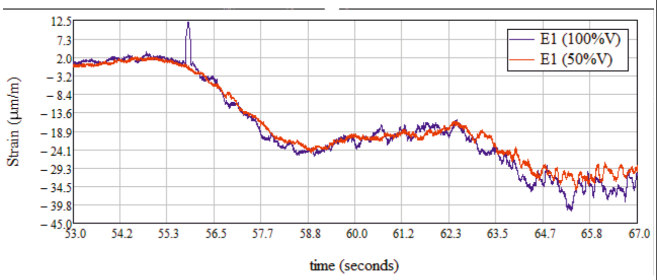
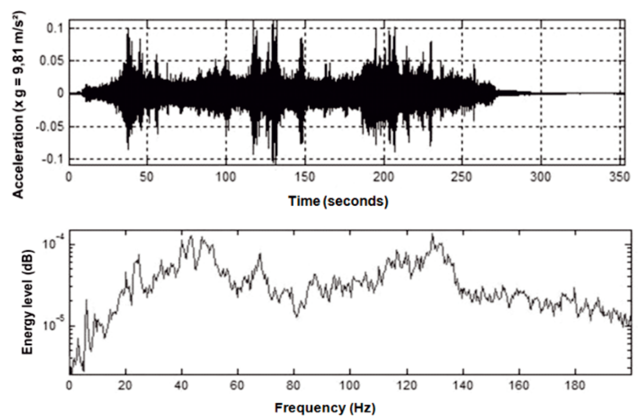


Figure 19 – Time-frequency domain response for measured acceleration from VAD, arrangement 01



5.2 Longitudinal beam Monitored

The low strain levels on the concrete surface and on the flexural reinforcement showed that the longitudinal beams are according to the national code prescriptions [12]. Thus, the dynamic effects had similar answers for tests with different speeds although these tests were carried out with different compositions. The dynamic increasing was very small when compared static and dynamic tests.

5.3 Experimental modal analysis

In relation to modal analysis it was verified that the structure can be excited by many ways depending on the train speed and condition of the wheel system. Another observed aspect from modal analysis was the majority of vibration modes with frequencies below 6 Hz are not requested by normal speed composition, becoming hard to measure frequencies from 0 to 10 Hz. The adoption of line-vehicle with unbalanced mass mechanism could be a good alternative. It was also observed that the natural frequencies of the structure are related to lateral flexure due the columns' flexibility.

Figure 20 – Diagram of stability created for parameter identification arrangement 01

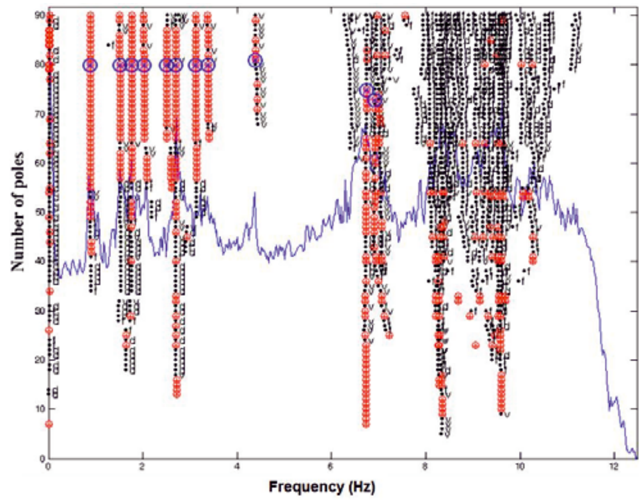
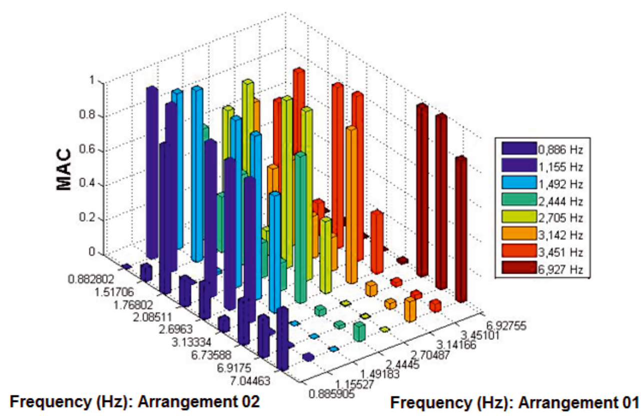


Figure 21 – Comparison between the modal shapes from the arrangements 01 and 02



5.4 Finite element computational model

The computational model was able to reproduce the static and dynamic behaviors satisfactorily for the first vibration modes of the structure. It may be used for future loads but another model using solid and shell elements is recommended, and due to the strong effect of the train mass in the modal behavior it should be reasonable consider the vehicle-structure interaction in future analysis. This interaction could reduce the train impact over the structure in the beginning and at the end of its passage along the bridge. Another alternative should be non-linear analysis or software solving the dynamic problem step by step along the time using tested methods, e.g. Newmark Method. Then, the impact coefficient could be replaced by more realistic factors.

6. Acknowledgments

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Figure 22 – Comparison between vibration modes

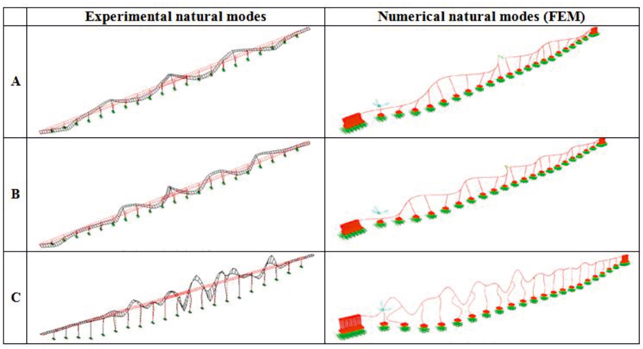

Table 7 – Results from the modal analysis

Mode	Experimental modes frequency (Hz)	Experimental modes Damping ratio (%)	Numerical modes (FEM) frequency (Hz)
A	1.562	4.631	1.541
B	2.704	2.999	2.883
C	6.904	1.426	7.893