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STRUCTURAL MONITORING SYSTEM
The use of Accelerometers for Structural Analysis

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Abstract: due to the high investment for construction, the major disorders and risks that involve the maintenance of Special Artworks (OAE - Obra de Arte Especial in Portuguese), increases the importance of monitoring the performance of these works to ensure the project has been adequacy of the project to the actual conditions of service. Monitoring the performance of these structures ensures planning to aim to prolong the average times between maintenance and decreasing repairs downtime. This study seeks to analyze the movements of arrows at a viaduct stringer, verifying its behavior compared to that specified in structural previous design. The applied methodology consists in the implantation of sensors in part of the structure, aiming to capture the displacement caused by the bending moment. The results are aimed at the creation of computational modeling with the aid of a specific computer program to compare the behavior of the viaduct loaded with data from the calculation memorial with the data presented in the readings performed by the sensors of monitoring in a given charging period.

Keywords: SHM, monitoring structures, viaducts, bending performance in beams, costs, preventive maintenance.

1 Introduction

The OAE, a denomination for bridges, viaducts, tunnels, etc., are works aimed as transposition of obstacles that hinder the continuity of a road. One of the great functions of these structures, along with highways in Brazil, is to assist in the disposal of products and goods to their destination. They are made so that the flow of vehicles does not cease and the traffic is as continuous as possible.

How to evaluate if the performance of the structure is in accordance with that predicted by the project?

Due to the high investment cost and the importance of the viaducts, a follow-up should be made to verify that the performance in its state of service is as expected and specified in the project, to ensure greater safety and durability. One way to facilitate this is by monitoring its use by an electronic structure monitoring system.

2 Literature Review

2.1 Structural Monitoring

The useful life of a structure is related to the repetition of dynamic actions, these actions are a progressive process that occurs in the structure when cyclically subjected to stress variation. Normally the OAE receive loads below the yield limit of the materials that were designed, but due to the very large number of cycles, it is essential to constantly evaluate the fatigue.

Recently, for the assembly and monitoring of the JK Bridge, in Brasília DF, was performed instrumentation during the construction phase measuring specific displacements: snap forces, temperature, wind speed, and accelerations.

According to the company responsible for the assembly and instrumentation, the measurement was performed in 48 stalks (Figure 1) in real-time, allowing modifications to be made throughout its assembly.

Figure 1. Stalk in JK Brigde, Brasilia, Brazil.

For Almeida (2003), after the assembly phase, the implanted instrumentation allowed the determination of the static and dynamic characteristics of the bridge, thus obtaining the bridge calibration, which will be used for comparison purposes in the next evaluations throughout the structure life.

This method allowed the values acquired by the sensors to be adjusted with the values predicted in the project, thus the staging time reduced from 3 months to 18 days.

In a study conducted by Aranha (1994), about 146 works in reinforced concrete structure (attacked by several degradation phenomena), it can be verified that the OAE are the interventions that have the highest cost (Table 1):

Table 1 -cost/m² of recounting interventions

Tipo de obra	Custo de recuperação total (US$)	Área total (m^2)	Custo / m^2 (US$/$m^2$)
Residenciais	111.139,16	2.581,80	43,04
Comerciais / serviços	129.384,08	7.313,00	17,69
Industriais	22.620,93	742,45	30,46
Institucionais	1.025.366,01	18.128,52	56,56
Pontes/ viadutos / trapiches	2.659.689,24	36.819,59	72,23
Praças esportivas	2.471.834,63	62.205,60	39,73
Total	**6.420.034,05**	**127.790,96**	**50,24**

(Fonte: ARANHA, 1994).

According to the Sitter law, or the law of five, the cost of intervention in the structure grows exponentially (Figure 1) in the ratio of 5 (five) in relation to the preventive measures (Helene, 1992).

In this context, it is evident the importance of inspection procedures in the preventive maintenance and conservation of special works of art. The maintenance of viaducts represents economic importance and strategic role, due to the high costs of investments involved in the implantation or recovery of this type of structure.

Because OAEs are of great importance to millions of people, any change in its structure can impact the day-to-day of a large number of people. Knowing that the useful life of an OAE is directly linked to its use and preservation of its structure, it is up to the manager to be aware of the complexity present in this type of structure and ensure its condition of use.

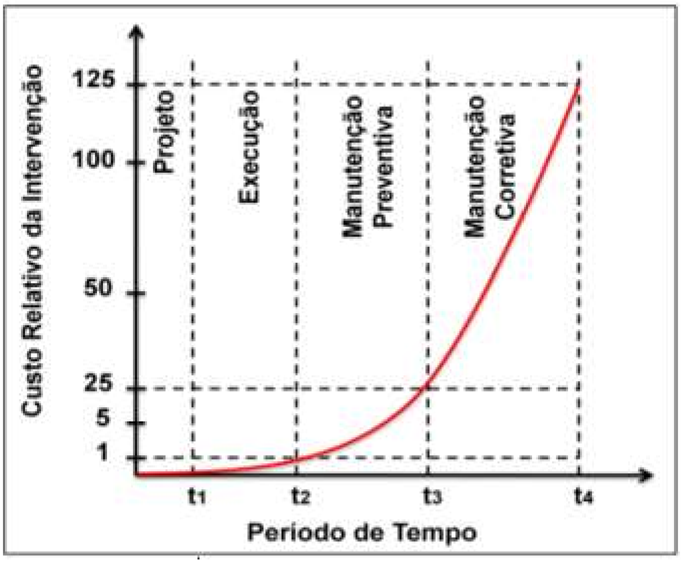

Figure 1 -Five´s law

Fatigue

According to NBR 6118 (ABNT 2014), It was evaluated that the useful life of a projected structure, adopting the parameters of resistance MPa 20, covering variable from 2.5 cm to 5cm, is 50 years. However, some parameters such as aggressive agents (CO_2, chloride ions), carbonation, despassivation and vibrations are not taken into consideration.

In his work strengthening and recovering bridges and viaducts, Teacher Vitório (2015), brings the following indicative values the lifetime of a project (Table 2). However, the service life of any viaduct can be reduced by the malfunction of some element of the OAE such as expansion joints, support devices, tray drains, etc. These elements have a less useful life than the structure and can be the gateway to pathologies.

Categoria do tempo de vida útil de projeto	*Valor indicativo do tempo de vida útil de projeto (anos)*	*Exemplos*
1	*10*	*Estruturas provisórias*[1)]
2	*10 a 25*	*Componentes estruturais substituíveis, por exemplo, vigas-carril, apoios*
3	*15 a 30*	*Estruturas agrícolas e semelhantes*
4	*50*	*Estruturas de edifícios e outras estruturas correntes*
5	*100*	*Estruturas de edifícios monumentais, pontes e outras estruturas de engenharia civil*

1) As estruturas ou componentes estruturais que podem ser desmontados, tendo em vista a sua reutilização, não deverão ser consideradas como provisórios.

Table 2 – Time of Life

Fatigue is an extremely important means of degradation, which can be accentuated in structures subjected to cyclical loading. In its study fatigue life of structural elements of reinforced concrete of road bridges, Carlos Filho performs a check of the limit's tension in the project life as shown in table 3:

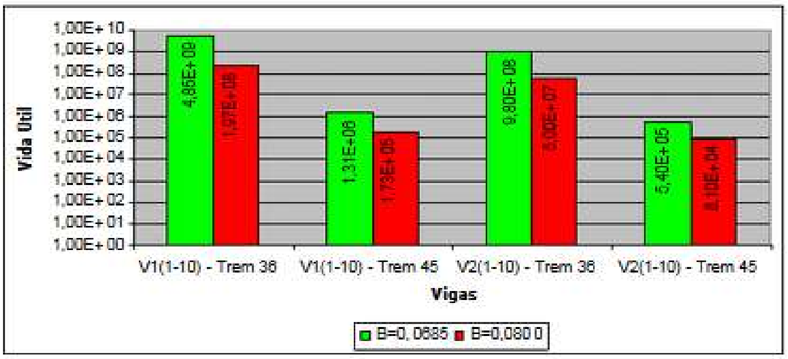

Table 3 – Fatigue life of structural elements of viaducts

This study above states that significant reductions in the useful life can occur for the requests arising from the loading of the 45 type train, which is mainly worrying for the older OAE projected under the norms of the time.

Among the various models of fatigue damage, the damage model (the simplest and the widely used) is the linear damage proposed by Palmgreen(1924) and Miner(1945), known as Miner's rule, where it suggests that the accumulated damage is Proportional to the energy absorbed by the material.

$$D_i = \sum_{i=1}^{k} \frac{n_i}{N_i}$$

D: Accumulated damage Rate
K: Number of different voltage levels in a specific charging sequence
n: Number of voltage cycles with a certain amplitude
N: Number of voltage cycles required to occur failure

Failure occurs when:

$$\sum D_i = \sum_{i=1}^{k} \frac{n_i}{N_i} \geq 1.0$$

However, according to Miner (1945), in many cases it was verified that the sum of the data at the moment of failure is different than 1, i.e., fatigue is an extremely complete engineering problem where this complexity significantly hinders the Correlation between laboratory tests and components under load in service.

3 Methodology

Among the many applications of network sensors, a particularly promising is the monitoring of the integrity of the structure, which monitors the structural health of buildings and structures of civil engineering.

The measurement of structures such as a viaduct or a building is usually enormous, and the installation of very long signal cables requires high installation costs. Besides, long cables leave the wires vulnerable to interference and weathering, so wireless data transmission is highly beneficial.

Structure integrity monitoring usually requires the measurement of vibration data, such as acceleration and speed. The measured data are analyzed by the modal analysis method to obtain the resonance frequency, damping ratio and spectrum response[1].

For wireless vibration measurements, time synchronization is very important because the vibration measurement for modal analysis requires simultaneous multi-point detection data. As a result, even if each sensor node acquires data and sends it exactly at the same instant, because the arrival time of the data usually does not match. Because when data is used for modal analysis, a time difference can be misunderstood as a phase offset. To maintain the precise time consistency between wireless nodes, time synchronization is indispensable.

Modal analysis is of great importance for bridges and viaducts, where the engineer should try to keep the natural frequencies away from the frequencies of people or vehicles passing the viaduct.

4 Case of Study

The work evaluated was a conventional reinforced concrete viaduct, molded at the site, located in the northern exit of the DF 003 highway, in the balloon passage of the Torto – Colorado balloon, Brasília – DF (Figure 2).

Figure 2 – Balão do Torto - DF 003

[1] The modal analysis corresponds to the measurement field and the analysis of the response of structural or fluid dynamics as excited throughout the frequency spectrum. As a result, we obtain the natural frequencies of the structure and its modes (forms assumed by the structure in each of the natural frequencies). The dynamic response of a structure excited by an external force is commonly called a forced response.

The viaduct has a total length of 46.78 m divided into three spans (7,51 + 29,88 +9, 23m), total width of 16.40 m and cross section in coffin girder with six transverses with the following spacing: 3,44-3,67-6,13-5,87-5,87-5,67-5,77-4,22 -4, 41m. The section height is variable in the transverse direction, with 0, 80x1,72 and 0, 65x1,72 .00 m.

The viaduct was constructed with a section in a coffin girder with bracing bands on the girders (internal Transverses) being positioned along the internal stretches, with transverse girder in each support of greater rigidity, which transfers to the support of the board to ensure homogeneity over the mesostructure.

The concrete used was Fck = 30 MPA and CA-50 steel, average thickness of the slab of the board with 20 cm and coating in asphalt layer of 7 cm. For the calculations, the following coefficients were considered:

Impact coefficient: φ (CIV) = 1.35 (according to standard NBR-7188:2013)

Coefficient of number of tracks: (CNF) = 1.0 (up to two tracks)

Additional impact coefficient: (CIA) = 1.25 (concrete structure)

Input plate tilt = 10 º

Board Age – 14.00 °

φ t = (CIV) + (CNF) + (CIA): 1.6875

The load increase coefficient (Ɣf) will have two considerations:

ƔF1 = 1.35 for permanent loads

ƔF2 = 1.50 for accidental (train-type) and secondary loads

4.1 Considerations of the working efforts:

Assembly Weight: Consideration of the cross section area of the Coffin including the external wheel guard, as shown in Figure 3: (P1): g = A(area) x Ɣc = 7.756 m² x 25 kn/m³ = 193.91 kn/m

Asphalt coating (Ra):

- g = H x $Ɣ_a$ = 0.07 m x 15.60 m x 22 kn/M³ = 24.02 kn/M

Secondary overload (Ss):

- G = 5.00 Kn/m²

Figure 3-SAs number 7

According to the DER - (Department of Roads in Brasília), the load distributed along the stringer, and secondary load subjected to the impact coefficient follow the calculations:

Permanent loads (q):

- Q: (P1 + Ra) x φ t = 217.93 KN/m x 1.6875 x Ɣf1 = 496.47 KN/m

Permanent load by Stringer (QL):

- (QL): x Ɣf1 = 496.47 KN/m/2 = > QL = 248.24 KN/m

Secondary loads (QP):

- QP1: Ss x φ t x Ɣf2 = 5.00 KN/m² x 15.60 m (band) (1,6875 x 1.50) = 197.44 KN/m (all secondary band)/2 = 98.72 KN/m
- QP2: Ss x φ t x Ɣf2 = 5.00 KN/m² x 12.60 m (band) (1,6875 x 1.50) = 159.46 KN/m (band, minus train-type POSITION)/2 = 79.74 KN/m

Charges considered punctual:

- Q-1: Load of internal cross in the acting on the stringer: A x L (medium) x Ɣc = 2.848 m³ x 25 kN/m³ = 71.20 kN/2 = 35.60 KN
- Q-2: Load of the internal main cross in the, on the supports acting on the stringer according to Image 6:
- A x L (average) x Ɣc = 7.12 m³ x 25 kN/m³ = 177.99 kN/2 = 88.99 KN

Freight of the train movable type road (450 Kn):

- Concentrated load on wheel (P): 450 kN/6 = 75 kN
- QTP: P x (CIV x CNF x CIA) x Ɣf2 = 75 kN x (1.35 x 1.0 x 1.25) x 1.50 = 189.84 KN

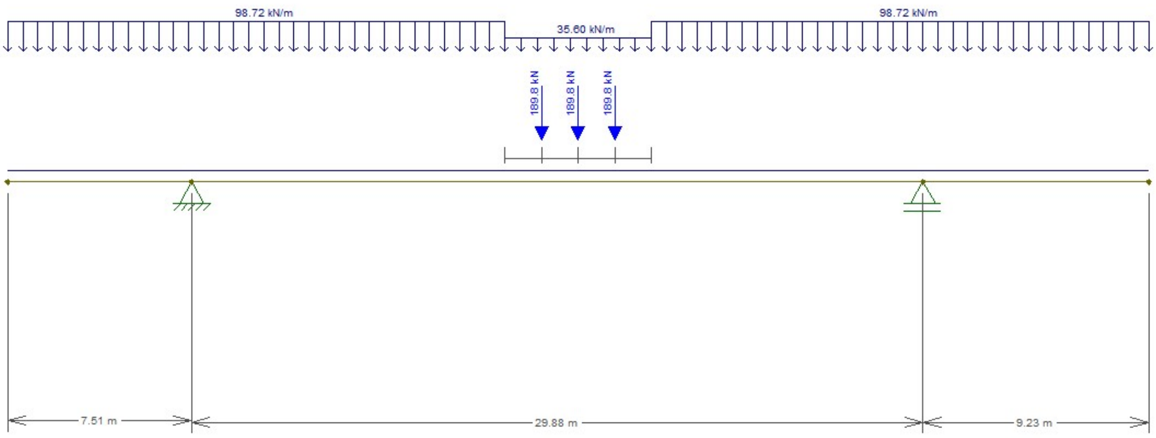

Figure 4 – Loading in Bean

5 Instrumentation of verticals displacements

It is important to highlight that structural monitoring does not only mean the action of installing sensors and checking readings, but also consists of several steps: The first of these is the clear definition of the objective.

To measure the above-mentioned viaduct, a set of sensors was installed to measure the main beams, where the simple flexion value was measured.

The measurement of vertical displacements was held on face points top of the beam located in the cutting Flights of the 1st, 2nd and 3rd, as shown in figure 5. Also to lower costs application of the sensors, only six points will be monitored in section A-A, B-B and C-C.

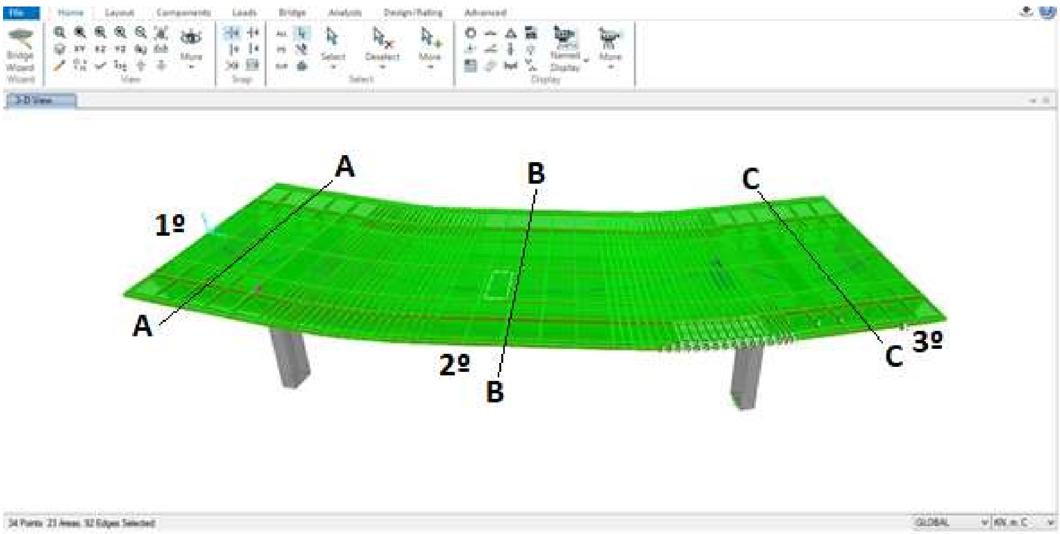

Figure 5 – Installation Points

For the measurements, it was used, in addition to the sensors, the central module for capturing and processing data. This module receives the data sent by the sensors and organizes it in the way it was programmed. These data undergo a treatment (filter) and were stored in a database to be used in the feedback, serving as a basis for comparison.

The data acquisition system will be responsible for processing and writing A read value to a representative value. The conditioning of the most common signal values are:

- Amplifier if signal;
- Signal Attenuator;
- Filter;
- Linearization.

For comparison purposes, theoretical reference values, obtained through simplified modeling of the viaduct structure, will also be presented. The finite element method was applied by the CSIBridge program.

The viaduct board is represented by shell elements, an object of the area used to model plates and membered, and in the pillars, considered perfectly attached to the foundation, were used Bar elements.

The process steps will follow the organizational chart proposed by Gilberto Nery (2013), in his article Construction Monitoring where he mentions the dependencies between the monitoring steps, and there may be more or fewer steps depending on the complexity of the structure and the monitoring desired to apply as shown in Figure 6.

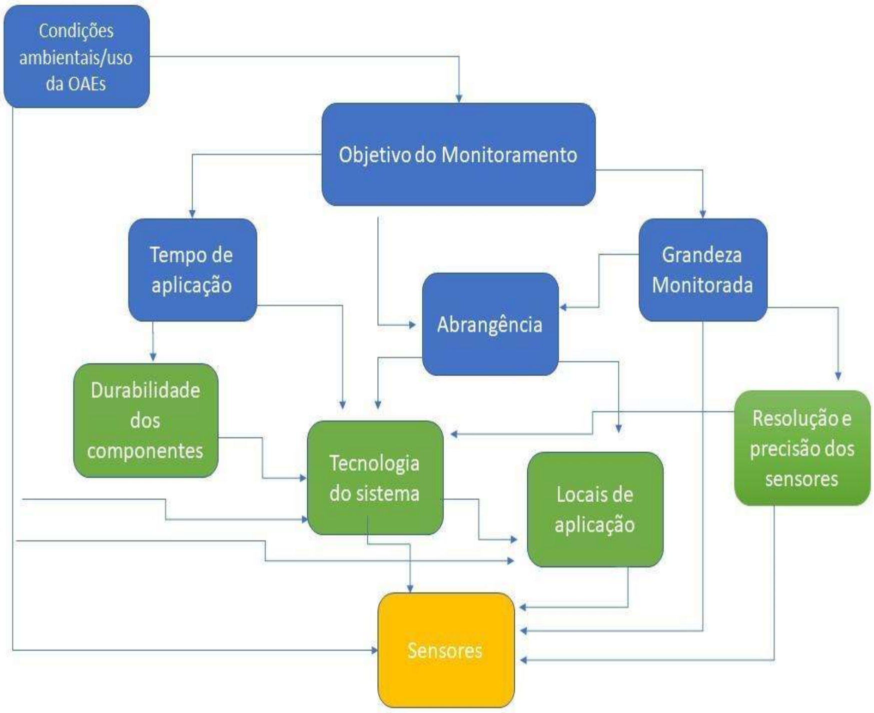

Figure 6 – Flow chart

6 Dynamic Rehearsal

The technique used consists of performing dynamic tests with registration and analysis of vibrations produced by truck traffic over the viaduct. Thus, the dynamic properties of the structure were experimentally determined. Concomitantly, a simplified mathematical model of the structure was elaborated using the properties of the materials and bonds, indicated in the original design.

The theoretical modal analysis was then made, determining the natural frequencies and the deformed modals. The comparison between the theoretical and experimental values provided indications about the behavior and the current state of the structure.

Finally, the mathematical model was calibrated by modifying its theoretical properties initially used, until the dynamic properties obtained in the model and the experimental ones were close. The calibrated model was considered "real" and will serve to analyze structural safety.

6.1 Equipment used and positions and editing

To obtain the accelerations of the viaduct structure, six acceleration transducers (accelerometers), the MPU6050 (figure 7) were used.

The integrated circuit CI **MPU6050** In addition to the two sensors has built-in a feature called **dmp**

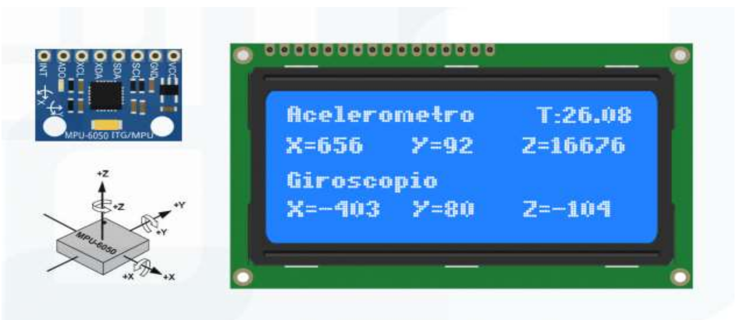

Figure 7 – Accelerometer

(Digital Motion Processor), responsible for making complex calculations with the sensors and whose data can be used for gesture recognition, navigation (GPS) systems, games, and various other applications. Another additional feature is the temperature sensor built into the IC, which allows for measurements between-40 and + 85 º C. It has high precision due to the 16-bit digital-analog converter for each channel, capturing the X, Y, and Z axes at the same time as shown in Figure 9.

Specifications:

- Chip: Mpu-6050
- Operating Voltage: 3-5v
- AD 16-bit converter
- Communication: STANDARD I2C Protocol
- Gyro Band: ± 250, 500, 1000, 2000 °/s
- Accelerometer range: ± 2, ± 4, ± 8, ± 16g
- Dimensions: 20 x 16 x 1mm

During data acquisition, readings are made up to the highest frequency allowed by the accelerometer, which is 1000 Hz according to the technical specifications. The transducers will be placed in two different configurations to check the different vibration modes of the structure. Two of these positions will be kept fixed (main of 29, 88m), due to the need to maintain a reference for all measurements performed in the calculations of the experimental modal analysis. In Figure 8 the transducer provisions and the readings positions can be observed.

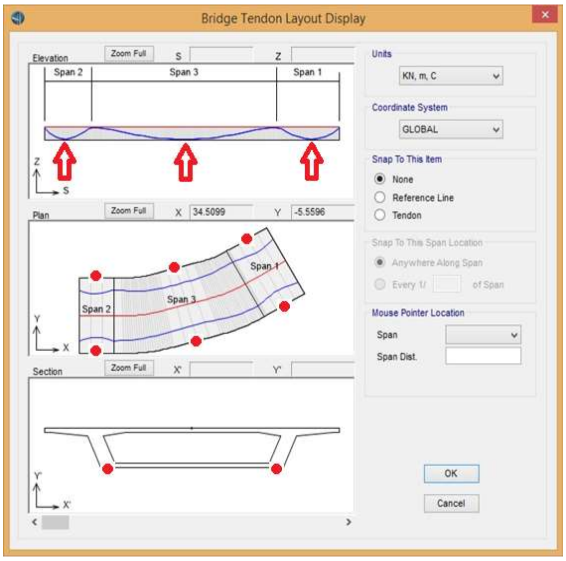

Figure 8 – Position of sensors

7 Experimental Modal Analysis

The NBR 6118 Technical Standard (ABNT, 2014), identifies the limit state of excessive vibrations (ELS-VE) as the one that occurs when "the structure, by its conditions of use, is subjected to shocks or vibrations, the respective effects should be considered in determination of requests and the possibility of fatigue should be considered in the sizing of structural elements ".

The standard recommends that the analyses concerning vibrations in concrete structures should be made in linear regime with the natural frequencies F_{Nat} being kept distant from the critical frequency F_{crit}, being specified its Limit on $F_{nat} > 1.2\ f_{crit}$.

The NBR 8800 (ABNT 2008) "Frequent combinations of service" where it is defined that "the frequent combinations are those that are repeated many times during the life span of the structure, of the order of 10^5 in 50 years, or that has Total duration equal to a non-negligible part of that period, of the order of 5% ".

According to Mohseni (2014), the length of the span and its inclination angle influence the natural frequency of the viaducts and Bachmann (1995) and Silva (1995) reinforce that in general the viaducts fluctuate in a range Frequency range from 0 to 14 Hz. This first analysis is of particular interest in verifying the behavior of the weight in the natural frequency vibrations of the viaduct. In the case of vibrations, it should be considered the possibility of resonance concerning the structure or part thereof.

Moutinho (2007), exemplifies the control of lateral vibrations on a pedestrian walkway located in the city of Toda in Japan. According to the author only with the implementation of water reservoirs inside the board the lateral vibrations suffered a decrease of about 65% concerning the same loading conditions in figure 9.

The vibration modes and the natural frequencies of the viaduct were obtained using the tests with the highest traffic intensity, that is, with greater excitations of the structure. In general, the vibration modes of viaducts have flexion or torsion deformation.

The identification of these modes was performed through calculations in the frequency domain (calculations of power spectral auto density), using the bending components for the analyzed section. The presence of peaks in the frequency response of the structure corresponds to amplification of the excitation spectrum or to the structure's own modes.

Figure 9. Walkway in Japan

8 Fourier Transform

One of the tools most used in the sine analysis is the Fourier transform, where a sum of harmonic components is used. According to Yanilmas (2007). This type of decomposition makes it possible to transform the periodic vibration function of the time domain to the frequency domain.

To analyze the time and frequency domains, the time domain must be repeated at intervals equal to *t* (time), and *T* is the cycle period of a frequency signal *f* (figure10). Because of this, the basic formula for mapping between two domains is obtained:

$$f\frac{1}{T}$$

Where:

f= frequency

T= time

In time domain, you need to define the function and parameters that describe it:

$$x(t) = A * \sin(2\pi f t - \varphi)$$

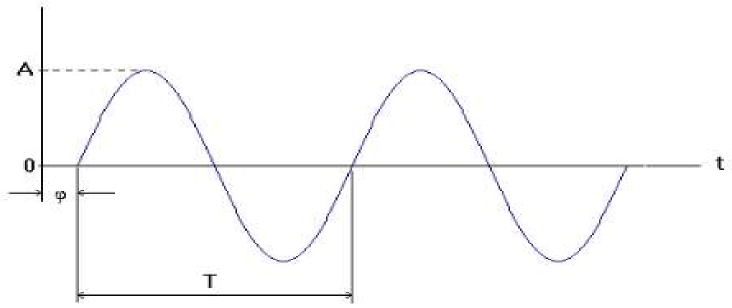

Figure 10-Relationship between time and frequency domain

Where:
A = amplitude
T = $\frac{1}{f}$ = period
φ= initial phase
t = time
sin = sine
π = Pi

However, not all functions defined in the time domain represent a constant period. For example, in the function below (figure 11), although it is described in the time domain and consists of two periodic signs, and It isn't periodic.

$$f(t) = \cos 10t + \cos(10 + \pi)t$$

t = time
cos = cosin

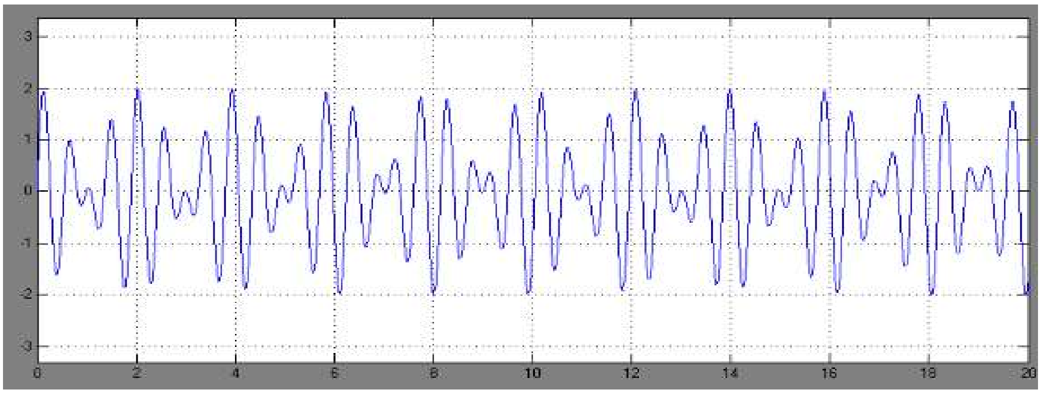

Figure 11 – Time domain

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$$f \frac{1}{T}$$

Where:
f= frequency
T= time
In Time domain, you need to define the function and parameters that describe it:

$$f(\omega t) = A_0 + \sum_{n=1}^{\infty} A_n \cos(n\omega_1 t) + \sum_{n=1}^{\infty} B_n \sin(n\omega_1 t),$$

Where:
Being:
F = frequency
$\omega_1 = \frac{2\pi}{T}$
T = time
sin = sine
cos = cosine
$\sum$ = summation
H = Lower limit
∞ = Upper Limit
Therefore, the Fourier transform consists of transforming the time domain signal into the frequency domain.

9 Theoretical modal analysis and model calibration three-dimensional mathematics

With the finite element model described, a theoretical modal analysis was performed to determine the first modes of vibration and natural frequencies of the structure. From the results, it was necessary to perform a final calibration of the model, to compare them with the information derived from the experimental modal analysis.

It was necessary to modify the model until it reached similar experimental and theoretical vibration modes. Some of these modifications should be based on the observation of the current conditions of the viaduct structure; Among them, the most important will be the lower slab of the viaduct board with its reduced stiffness.

11 Field survey

The sensors were attached to the structure with a glue epoxy bi-component as requested by the manufacturer. Being that, the Strain Gauges were attached directly in the concrete and the accelerometers were assembled in modules as shown in figure 12.

Figure 12 – Installation of sensors

The readings were performed with the sensors in pairs being B3-B4 Stationary and A1-A2 and C5-C6 as shown in figure 13.

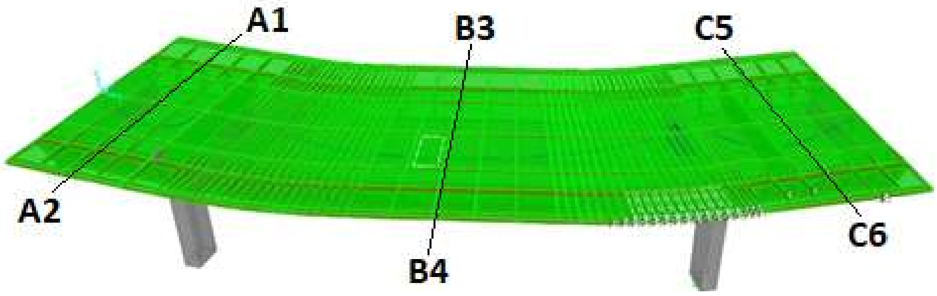

Figure 13 – reading positions

For the measurements, we used the concept of mechanical vibration where the mechanical oscillation of a body was defined concerning a reference position. This oscillation can be described by amplitude and frequency parameters. As it is a mechanical oscillation, the amplitude of the vibratory signal was chosen by the representation in quantities in acceleration in meters per second squared (m/s^2).

The frequency is corresponding to the number of movement cycles occurring in the time interval of one second, is measured in Hertz units (Hz).

The first step was to obtain the natural frequencies of the structure and to define the criteria for the amplitude of the time domain for frequency. Knowing that the maximum acquisition rate of the sensor used in reading is around 1,000 Hz, where it limits to a sampling frequency at most 500hz.

However, as in the computational analysis and in the theoretical framework the natural frequencies are in the range of 0 to 14 Hz, we opted to limit the maximum reading at 500Hz, thus, the maximum values of 250Hz are established. This limit is due to the sampling theorem of Nyquist-Shannon where it says that the sampling rate should be greater than double the most frequent component that should be analyzed in the measured signal.

The sampling period was in 60 minutes, where two measurements were made every millisecond, therefore, on average 500 readings per second on each sensor. It was possible to analyze the noise range of all the sensors where 5% of the readings were removed, which are the most distant from the mean values obtained in the tests, which in this case is the zero acceleration. In this period 1,414,548 valid readings were recorded. However, to meet the specific objectives, the analysis is detailed in reading the passage in the vehicle at 14:22:45 in figure 14.

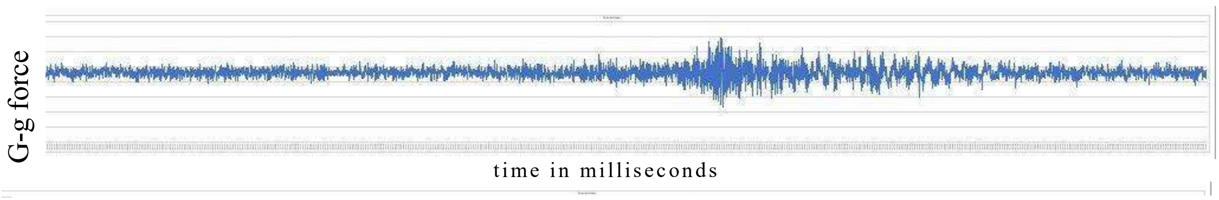

Figure 14 – G acceleration as function of the time

During the entire reading period was given priority to analyze the vehicle of train type TB-450 preset by NBR 7188 (ABNT 2013), because it is the most unfavorable position, however, in the course of the analysis this kind of vehicle was not detected, only the truck type figure 15.

Figure 15 – Truck 6x2

During measurement of the Truck were captured 944 readings on sensor B3 (figure 16), the focal point of our analysis. Of these 944 readings were removed the natural frequencies and for analysis of the response of the measurements, the output signals were converted from the time domain to the frequency domain from the Fourier transform as described in the modal analysis.

After conversion analysis, the frequencies were around 32 Hz during the passage, with a maximum acceleration of 1.02 g or 10.0 m/s^2. With these values could apply the conversion formula to obtain the displacement:

$$\text{Des} = \frac{Acc}{(2 * \pi * vel)^2} = 0{,}2\text{mm}$$

Where:

Des = Displacement

ACC = in m/s^2

Vel = in Hz

The numerical simulation during the passage of the train type obtained the maximum displacement value of -3mm at the moment as illustrated in Figure 17:

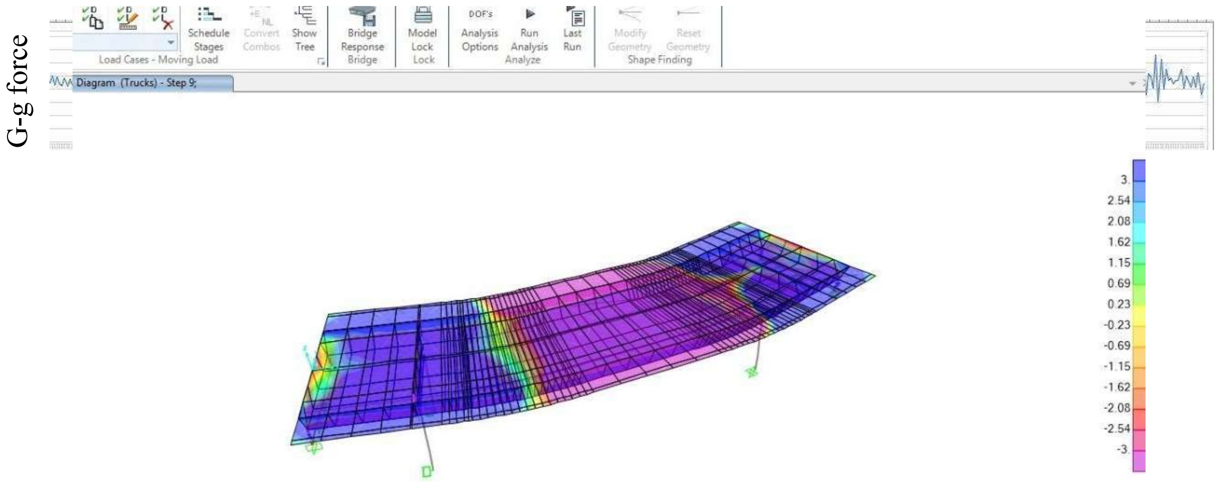

Figure 17 – Dynamic response in the middle of the board

5 Conclusion

With the readings performed, it was possible to develop a comparative study between the behavior predicted in the project with the data presented by the monitoring system. The proposed analysis methodology was used through the development of a three-dimensional numerical-computational model and the observation of installed sensors "in loco". Thus, for the purpose of comparative calculation, we considered the technical data sheet elaborated by the manufacturer, where it was observed that the displacement captured by the B3 sensor is equivalent to the computational calculation.

During the entire reading period it was given priority to analyze the vehicles of train type TB-45 of 450 KN, with six wheels defined by NBR 7188 (ABNT 2013), because it is a more unfavorable position. But during the measurements did not pass any vehicle that corresponding to the characteristics of the norm. The most suitable vehicle for our test and that we take as a basis for the measurement was the mechanical horse Truck 6x2. According to the technical data of Mercedes-Benz, the empty rigged mechanical horse has the reference 4.325 tons on the front axle and 3.030 ton in the back axis.

Applying the same formula, with an average close to 3.5 ton per axle, we can take into account The Weight of 35kn per axle, Equivalent to 35kn/6 = 5, 83kn. Applying the project coefficients P * (CIV * CNF * CIA) xYf2 = 5, 83kN * (1.35 * 1.0 * 1.25) * 1.5 = 14, 75kN.

Because of the results presented in the analyzed graph, it was observed that the maximum value of displacement predicted in the project at the passage of the train type TB-45 was estimated at 3 mm. The "in-loco" Analysis of the work of OAE number 07 presented the displacement of 0.2 mm for the vehicle empty truck.

The result read was not obtained by the project load, but with a vehicle with a lighter axle. Considering a direct proportionality between loading and displacement, it can be inferred that the result read is estimated for a load of 45 T, to compare the order of magnitude between the read and the calculated. In this sense, the result read is estimated as: 0.2 x 189.84/14.75 ~ 2.6 mm. The estimated displacement result for the TB-45 type train read from 2.6 mm becomes compatible with the 3 mm. calculated.

References

[1] ¿Qué es el monitoreo en la construcción? *O que é monitoração na construção civil?* What is monitoring in construction?; Boletim Técnico; Gilberto Nery, USP; 2013.

[2] ABNT - Associação Brasileira de Normas Técnicas. NBR ISO 31010, 2012. *Sistema de Gestão de Qualidade Requisitos*. Associação Brasileira de Normas Técnicas – ABNT. Rio de Janeiro.

[3] *Avaliação de desempenho de sistemas estruturais de pontes de concreto*. Tese de Doutoramento. PENNER, Elisabeth; Escola Politécnica da USP, 2001.

[4] BACHMANN et al. *Vibration Problems in Structures: Practical Guidelines*. Birkhäuser Verlag, 1995. 248 p.

[5] BAYRAKTAR et al. *Ambient Vibration Tests of a Steel Foot-bridge*. Journal of on destructive Evaluation, v. 29, 2010; p. 14-24. http://dx.doi.org/ 10.1007/s10921-009-0061-9.

[6] DNIT, 2004. NORMA DNIT 010/2004 - PRO. *Inspeções em pontes e viadutos de concreto armado e protendido* – Procedimento. Instituto de Pesquisas Rodoviárias. Departamento Nacional de Infraestrutura de Transportes. Ministério dos Transportes. Brasil. 15/11/2017.

[7] DNIT, 2011- PROARTE – *Programa de Reabilitação de Obras de Arte Especiais* – Unidade de Programas Especiais – UPEsp, 2011. Disponível em < http://www.dnit.gov.br/noticias/ii-forum-de-infraestrutura-de-transportes-debate-entraves-juridicos-as-atividades-do-dnit/6PROARTE.pdf

[8] DNIT, 2016. NORMA DNIT 010/20016 - PRO. *Manual de Manutenção de Obras de Arte Especiais*. Instituto de Pesquisas Rodoviárias. Departamento Nacional de Infraestrutura de Transportes. Ministério dos Transportes. Brasil., 15/10/2017.

[9] *Monitoração na construção civil* – Gilberto Nery de Araújo Neto, julho 2013.

[10] *STRUCTURAL HEALTH MONITORING USING SMART SENSORS* – Tomori Nagayama, University of Tokyo, 2000.

[11] NBR 9452 *Inspeção de pontes, viadutos e passarelas de concreto*. Rio de Janeiro,21/09/2017.

[12] NBR 7188 *Carga móvel rodoviária e de pedestres em pontes, viadutos, passarelas e outras estruturas.* Rio de Janeiro,11/11/2013.

[13] ORCESI, D, A; FRANGOPOL, M, D. *Bridge Performance Monitoring Based on Traffic Data*. Journal of Engineer-ing Mechanics, v. 139, 2013, p. 1508-1520. http://dx.doi.org/10.1061/(ASCE)EM.1943-7889.0000582.

[14]<www.dec.fct.unl.pt/seccoes/S_Estruturas/Mecanica_Meios_Continuos/upgrade/I_extensometria.pdf>, acesso em *27 de maio de 2009).*

[15] MOHSENI et al. *A Simplified Method to Estimate the Fundamental Frequency of Skew Continuous Multicell BoxGirder Bridges*. Latin American Journal of Solids and Structures, v. 11, 2014, p. 649-658.

[15]https://www.mercedes-enz.com.br/resources/files/documentos/caminhoes/atron/dados-tecnicos/v3_Atron-1635-4x2-Cavalo-Mecanico.pdf acessado em 12/03/2019.