1st LATAM-SHM 2023 Conference
A. Güemes, L. Mujica, J. Sierra (Eds)

FIRST EXPERIENCES OF CIVIL STRUCTURAL HEALTH MONITORING IN URUGUAY

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Key words: Bridge, instrumentation, damage, concrete, steel, assessment, stress.

Abstract
Structural health monitoring (SHM) of civil infrastructure is a world-renowned field for remote and continuous condition assessment. The use SHM systems optimize the resources available for traditional inspection campaigns and allows efficient preventive maintenance. In Uruguay, the use of SHM systems is still incipient. This article comprises recent work carried out by the academia, Facultad de Ingeniería, Univesidad de la República, and by two engineering companies, where different systems of SHM have been used and evaluated in real -or real-size- civil structures. The first case study involves a one 8 m long span of a concrete bridge, instrumented with accelerometers, strain gauges and deflectometers during a 6-month period. The second study involves the static and dynamic loading test of a steel trussed railway bridge using accelerometers, followed by a monitoring period of one week. Two longitudinal beams were extracted from a similar bridge, and analysed in laboratory, where additional damage was introduced to assess the system sensibility over damage. Finally, we present an academic study using ultrasonic wave propagation to monitor the compression level of a real size concrete column. For all case studies, we present the strengths and shortcomings of these first experiences, as well the remaining challenges for our future work.

1 INTRODUCTION

Structural health monitoring (SHM) of civil infrastructure is a well stablished field of study for condition assessment and preventive maintenance. It consists in attaching sensors to a structure and collect data remotely and regularly, in a non-destructive manner. The information is then post-processed and, by considering the structural characteristics, different types of conclusions can be drawn about the structure´s behaviour. Since the rise of telecommunications and the explosive production of inexpensive broad types of sensors, such as accelerometers, thermometers, strain-gauges, etc, which can be controlled by micro-chips, the use of different SHM techniques has grown and spread worldwide; from the academy to real field applications, it many SHM techniques have been utilized by companies, governments and other infrastructure's owners and decision-makers, in their own infrastructure assessment and maintenance plans.

In Uruguay, most tasks of condition assessment are carried out in the traditional way, by direct visual inspection. The use of SHM is still incipient. There are very few experiences of applied SHM of civil infrastructure, and no published documents, as research articles or case studies. Gradually, non-destructive testing and SHM are beginning to become understood, and their usefulness appreciated. Thus, one of our motivations is to spread the knowledge of SHM among the different actors of the building industry in Uruguay.

This article contains three different SHM experiences developed by three different institutions: one academic institution: IET-Facultad de Ingeniería of Universidad de la República, and two civil engineering consulting companies: CSI Ingenieros, and RDA Ingeniería. These experiences which were carried out separately. Therefore, the article is organized in three different sections, one for each experience. The overall goal of this article is to share our experiences with the engineering and scientific SHM communities and receive their feedback, as well as to promote the use of SHM in Latin-America.

2 USE OF TORSIONAL VIBRATION TO MONITOR THE COMPRESSION LEVEL OF A REAL SIZE REINFORECED CONCRETE COLUMN

2.1 Description of the structure under study and authorship

The structure is made of a 13 cm thick square concrete top slab supported by nine reinforced concrete columns of dimensions $20{\times}20{\times}200$ cm^3, which are founded in another 13 cm thick square concrete slab. The centre column was instrumented and tested, reinforced with four longitudinal steel bars of 12 mm diameter and 6 mm steel stirrups placed every 20 cm. The concrete with which the aforementioned elements were made has a 35 MPa standard compressive strength at 28 days after casting and was provided by a local ready-mix concrete company. See the structure and its instrumentation in Figure 1.

2.2 Specific objectives

The goal set for this case study is to present and evaluate a novel method to monitor axial compressions in elongated structural concrete members. This method is based on the use the nonlinear parameter β_G for the determination of the torsional vibration of the concrete members, which are being considered as nonlinear elastic as characterized by Spalvier et al [1]. As a result of the mentioned hypothesis, it is assumed that the dynamic shear modulus G varies in accordance with the axial strain to which the element is subjected to. That relationship is defined by

$$G = G_0(1 + \beta_G)\varepsilon \tag{1}$$

where G_0 is the dynamic shear modulus in the undeformed state and β_G is an elastic parameter that characterizes the material's nonlinearity [1].

Figure 1: Experimental configuration

2.3 Methodology

The concrete column was subjected with 3 loading and unloading cycles in steps of 0.6MPa (23 kN), ranging from 0 MPa to 4 MPa. To compress the column, a hydraulic jack was placed on top of the upper slab with a steel plate on top, anchored to four steel bars, which worked in tension.

Four strain gauges Tokyo TML model PL-90-11-1LJC connected to a Vishay Wheatstone bridge (configurated in quarter bridge) were attached to each column's face to measure axial strains. The resonance frequency was measured at each loading step using four accelerometers. These accelerometers were conditioned using a Dytran Signal Conditioner and the data was digitalised using a NI 9215/NI cDAQ 9174 unit.

Considering the formula detailed in Spalvier et al [1], the nonlinear parameter β_G can be obtained by fitting the formula

$$f^2(\varepsilon) = (1 + \beta_G\varepsilon)f^2(0) \tag{2}$$

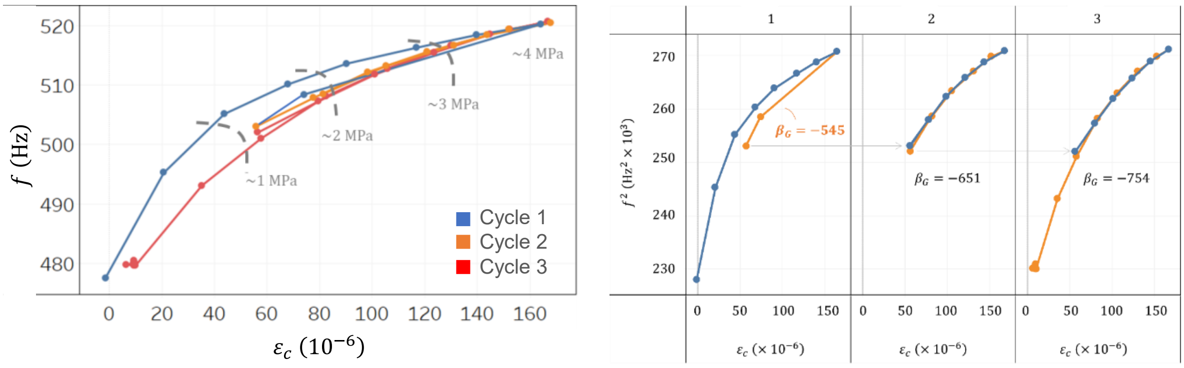
to the experimental data, where $f(\varepsilon)$ is the measured fundamental torsional frequency of vibration, and ε the axial strain.

2.3 Results and discussion

The results of the study carried out showed a very slight deviation from a linear relationship between the compressive stress and the resultant strains produced (figure not shown). Apart from this and a residual strain of approximately 10 μs, the curves for all cycles coincide showing an elastic behaviour.

Figure 2 portrays the fundamental torsional frequency of vibration (f) vs the compressive strain ε_c. Excluding the first loading cycle, the curves overlap significantly, which indicates very good repeatability. Figure 3 presents the squared of f with respect to the compressive strain ε_c for each loading/unloading cycle separately. For the latter two cycles it is observed an almost perfect elastic behaviour. While this apparent elastic behaviour is not applicable to the first cycle, it could be the result of microplastic or viscoelastic effects that occurred exclusively at that stage.

Furthermore, even thought f^2 and ε_c are positively correlated, they do not present a linear correlation as hypothesized previously. While this holds true, the hypothesis emerged from considering a polynomial nonlinear constitutive relationship ($\sigma = \sigma(\varepsilon_c)$) with strains of up to the power of 2, and, as a result, a possible explanation of the observed phenomena would be that the constitutive relationship includes higher powers of ε_c.



(a) *(b)*

Figure 2: (a) Fundamental torsional frequency of vibration wit respect to compressive strain, and (b) squared fundamental torsional frequency with respect to compressive strain.

Despite the previously mentioned discrepancy to the theoretical model proposed, the value of the nonlinear parameter β_G was calculated to -545, -651 and -754 for cycles 1, 2 and 3, respectively (for cycle 1 only the unloading stage was considered). Given that all the different values of β_G obtained are negative, which coincide with the work carried out by Spalvier *et al* [1], it is deduced that it might mean that a "hardening" behaviour occurs between the strains and stresses.

2.3 Conclusions

Based on the observed phenomena and analyses drawn from them, the following conclusions were drawn:

- A positive correlation can be established between the torsional frequency of vibration and the compressive axial strain. This could result in a feasible technique for monitoring reinforced concrete structures, however, further research must be carried out as the proposed linear correlation was not observed.

- When excluding the loading branch of the first cycle, the experiment yielded almost perfect overlapping of the curves. As a result, it is concluded that the results are highly repeatable and that an elastic behaviour was dominant.
- A hardening effect is observed for service levels compressive stresses [2].
- The nonlinear parameter β_G was measured, yielding the result $\beta_G = -650 \pm 104$.

3 MONITORING OF THE SAN BORJA BRIDGE BY CSI INGENIERIOS

3.1 Motivation

San Borja Bridge is in a rural area and, due to deck widening and reinforcement works being carried out in a bridge in a National Route 13 km away, it is used as a bypass for light traffic to avoid traffic jams. Because the bridge was built around 1960 the government authorities were concerned with its ability to sustain the increase in traffic flow. A proposal was made to detect any damage caused by the increase in traffic flow. The proposal consisted of doing a structural condition survey, followed by an initial load test to determine its current state, proceeded by real time monitoring to observe structural behaviour changes, if any, and concluded with a final load test. To the authors best knowledge, this was this experience was the first load test supported by subsequent SHM of a road bridge in Uruguay.

3.2 Description of the studied structure

Paso San Borja is a bridge over the Yi River in central Uruguay. The bridge deck consists of 10 slab spans, with each span being independent and simply supported on the piers using 5 cm wide neoprene strips. See Figure 3. These strips consist of two 1.5 cm thick bands with a steel plate between them. The roadway width is 5.00 meters, with two sidewalks of 70 cm each. The deck's thickness is 40 cm, and the distance between pier axes in all spans is 8 meters. On the deck's surface, there is a concrete pavement with an average thickness of 5 cm. Intermediate piers are concrete walls of 40 cm of thickness with a 50×60 cm rectangular beam on the top where the neoprene supports and the deck rest. Both abutments consist of reinforced concrete walls. The geometry of the foundations is unknown because they are underwater. A structural condition survey found that there were no pathologies compromising the safety of the structure and the average conservation state of the structure was acceptable. However, in some areas, remediation actions were recommended, and it was recommended that these pathologies be monitored.

3.3 Methodology

Six displacement sensors (D1 to D6) and nine accelerometers (A1 to A9) were installed in the bottom face of bridge deck, except for A1 to A3 and A7 to A9, which were installed in the lateral face of the deck. See Figure 4 and Figure 5.

(a) (b)

Figure 3: (a)View of Paso San Borja Bridge, and (b) tested span with auxiliary steel beams beneath the deck.

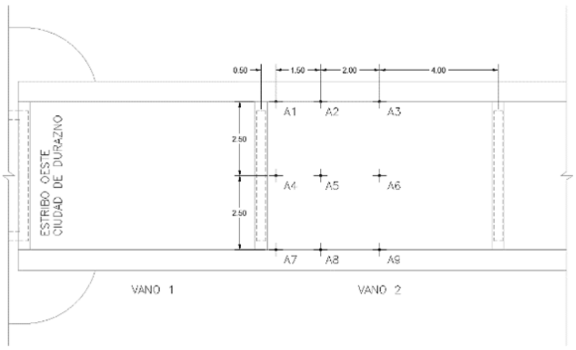


Figure 4: Plan view. Accelerometers location.

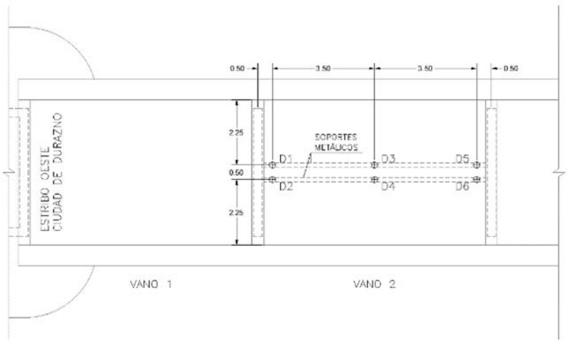


Figure 5: Plan view. Displacement sensors location.

For the installation of the displacement sensors, two auxiliary steel beams were installed under the deck, supported on the bridge piers. Three displacement sensors were installed for each beam, one in the centre of the span, and two others towards the beams' ends, next to each pier. The characterization was made for one representative span, which was selected based on conservation (structural condition survey) and accessibility. The truck used for the test had 6 axes, one simple axis on the front, a double axis in the middle, and a triple rear axis. Two load steps were performed. The weight on each axis for each load step is shown in Table 1. For each load step static and dynamic tests were carried out.

Table 1: Weight on each axis for each load step.

	Axis 1 (kg)	**Axis 2 (kg)**	**Axis 3 (kg)**	**Axis 4 (kg)**	**Axis 5 (kg)**	**Axis 6 (kg)**	**Total (kg)**
Step 1	5950	4900	4650	3590	4780	5210	**29080**
Step 2	6020	5840	5600	5060	6770	7400	**36690**

<u>Static Tests</u>

For each load step, initially, the triple axis was positioned at approximately one-quarter of the span (x/L = 0.20, where x is the position of axis 5, and L is the length of the span). This position was maintained until a stabilization of the displacement measurements was observed, with a minimum of 5 minutes wait. Subsequently, the procedure was repeated for the triple axis at the midpoint of the span (x/L = 0.50). The truck was centred to the longitudinal axis of the bridge in all cases. Measurements of displacement were obtained at the centre of the span, relative to the displacements at the ends (to characterize bending only).

Dynamic Tests

For each load step, two tests were made, one with the truck driving at 20 km/h and another at 40 km/h, to obtain records of vertical accelerations. The truck was centred to the longitudinal axis of the bridge in all cases**.**

Structural Health Monitoring

The SHM was divided into two stages: (1) during the months of November and December of 2021, using only accelerometers A3 and A4, and (2), and from February to March 2022, using only accelerometers A2, A4, A6 and A8. Disposition of sensors is shown in Figure 6.

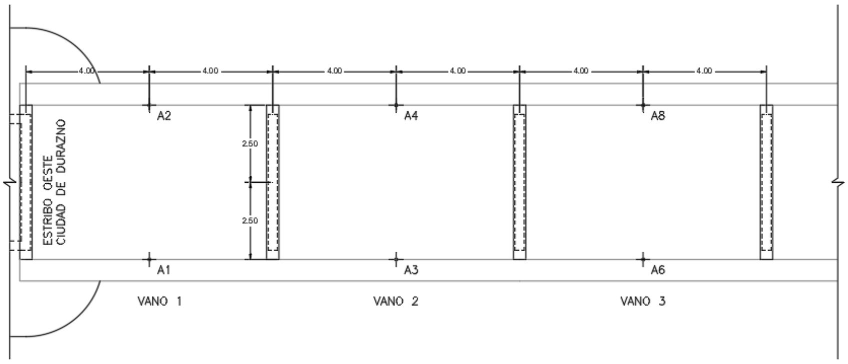


Figure 6: Disposition of sensors during Structural Health Monitoring stage.

To ensure the analysis captured only effects of traffic loads, a traffic counter was installed. This equipment allowed the detection of every vehicle that crossed the bridge, and therefore making sure that measurements made by accelerometers were related to traffic events and no other possible sources of vibration, which would add noise to the analysis.

3.4 Results and discussion

Dynamic Tests

Vertical acceleration records were analysed in frequency domain. An average natural frequency of 15.68 Hz was determined by the peak picking method. Frequency of vibration results are shown in Table 2.

Table 2: Average frequencies of vibration obtained during dynamic tests for each step and truck speed.

Frequency (Hz)	20 km/h	40 km/h
Step 1	15.9	15.7
Step 2	15.7	15.3

<u>Static Tests</u>

Table 3 presents the values of relative displacement at the span´s centre Results shown are the average measurements among both beams. These are compared to those obtained via a finite element shell model whose stiffness was fed using the frequency of the first mode of vibration (obtained by the dynamic test as shown below).

Table 3: Comparison between experimental displacements and analytical displacements

	Displacements Step 1 (mm)		Displacements Step 2 (mm)	
	x/L = 0.20	x/L = 0.50	x/L = 0.20	x/L = 0.50
Measured	-0.17	-0.34	-0.31	-0.52
Model	-0.24	-0.37	-0.41	-0.58
Error	28%	9%	25%	10%

Considering that the results are a comparison between static measurements and a finite element model calibrated by a dynamic test, some differences between values are expected. Moreover, measurements denounce a stiffer behavior of the structure than the theoretically predicted.

<u>Structural Health Monitoring</u>

Natural frequency vs. time plots were constructed. A threshold of 20 mg was defined in the accelerometers so only major events (heavy trucks) were detected. These events had a corresponding acceleration register, which was transformed to the frequency domain. The most energetic frequency was determined by peak picking. Then, frequencies were daily averaged, obtaining a variation of the frequency for each day of the monitoring period. As an example, the results of daily averaged frequency of vibration of the fundamental mode are presented in Figure 7. Similar results were obtained by the other accelerometers, and also during stage 2. No particular trends can be seen in the plot, only slight variations that can be due to the nature of the excitation or environmental factors, thus, no evidence of damage could be detected by the method.

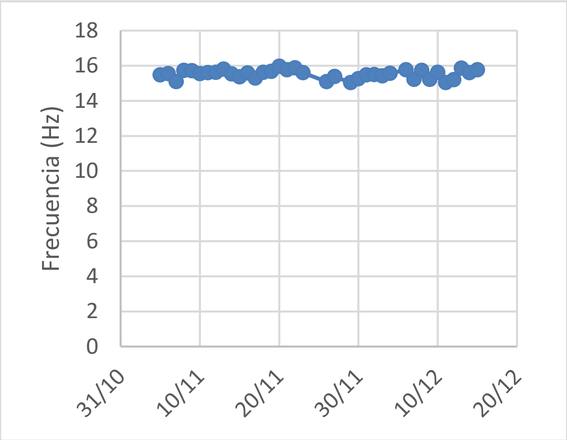


Figure 7: Register from accelerometer A3 (stage 1).

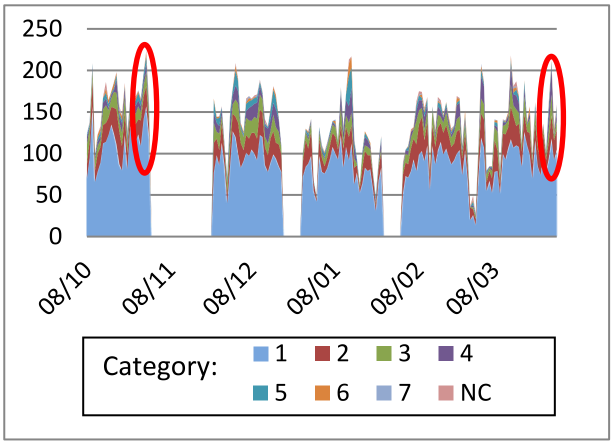


Figure 8: Daily traffic device results.

It was possible to characterize the nature of the traffic along the bridge using the traffic counter. See Figure 8. While it was observed that the periods in which no traffic over the bridge coincides with the events of flood, traffic peaks are consequence of an increase in heavy traffic (vehicle length greater than 5.50 m) and mainly on Fridays; this demonstrates that this route has been used as a bypass during traffic jams in route N°5, which are more dramatic on Fridays.

3.5 Conclusion

The load tests provided insights into the static and dynamic responses exhibited by a representative span of the bridge. This gathered information allows to assert that the stiffness of the bridge deck aligns with the theoretical stiffness, as acceptable correspondences were achieved in terms of deformations and frequencies. Regarding the Structural Health Monitoring period, the following conclusions can be drawn:

- The implemented measurement system has successfully captured the relevant traffic events occurring on the bridge.
- It can be asserted that there have been no significant reductions in the natural vibration frequencies identified in the studied span. Therefore, it is inferred that there has been no damage detected at either the global or local level.
- Moreover, the success in the San Borja Bridge project made us possible to carry out instrumentation applications of greater magnitude, such as the dynamic load test in the "Viaducto del Puerto"; a recently built bowstring arch bridge with vertical hangers.

4 VALIDATION OF A LOW-COST VIBRATION MEASUREMENT SYSTEM FOR STRUCTURAL HEALTH MONITORING BY RDA INGENIERIA

4.1 Description of the structure under study

The Santa Lucia Bridge is a 100-year-old steel truss rail bridge, located near the town of 25 de agosto in Uruguay. It was originally owned by the Uruguay Central Railway but is currently no longer in use for rail transportation. The bridge measures 26 m in span, 5.3 m wide, and 6 m height. It consists of a pair of lateral trusses connected by transverse beams (floor beams and top struts). In the plane of the transverse beams, beneath the rails, are the longitudinal beams ("stringers"), and on top of them, the wooden sleepers. The structural connections are made using plates and rivets. Two stringer beams were extracted from a similar bridge and analysed in more detailed in laboratory.

4.2 Specific objectives

This case of study had two specific objectives. The first and most important one was to validate the developed low-cost MEM-based system by comparing it to a state-of-the-art commercial system in a real-life test. In addition to this, the tests were used to dynamically characterize the structures under study and, in the case of the bridge, to stablish a reference base line to monitor the evolution of the system.

4.3 Methodology

As previously mentioned, two parallel systems were used for measuring vibrations in the studied structures. The state-of-the-art system utilized by IET-Facultad de Ingeniería (FING) was composed by a series of PCB one axis accelerometers, a Dytran signal conditioner, and a National Instruments digitizer/data acquisition system. The sampling frequency was 2 kHz for the bridge test and 1 kHz in the case of the stringer beam. In both cases the sensitivity varied for each accelerometer, but it ranged from 10 to 500 mV/g.

On behalf of RDA Engineering, the company used a low-cost system developed by CMS, based on previous works on the matter [3-5]. Its main component is the Invensense MPU 6050 MEMs accelerometer. It is a three-axis sensor with a measurement range of ±2g and a 16-bit resolution, resulting in a sensitivity of 0.06g mg/LSB. The sampling frequency used for both tests was 1 kHz. Prior to testing, all the transducers were statically auto calibrated using the projection of the gravitational acceleration as a reference [6].

In the bridge test, the accelerometers were placed in the positions shown in Figure 9, in which, they were expected to be excited by the lower modes. At each of these points, a pair of sensors from the two different systems were set as closely as possible while maintaining the alignment of their axes. Most of the FING one-axis sensors were aligned with the vertical axis apart from the sensor 5 in the bridge test corresponding to transverse axis of the structure.

As for the stringer beam, it was positioned rotated 90 degrees around its axis to study flexural vibration around its minor axis. It was mounted on a pair of metal cylinders on both ends, simulating a simple supported configuration.

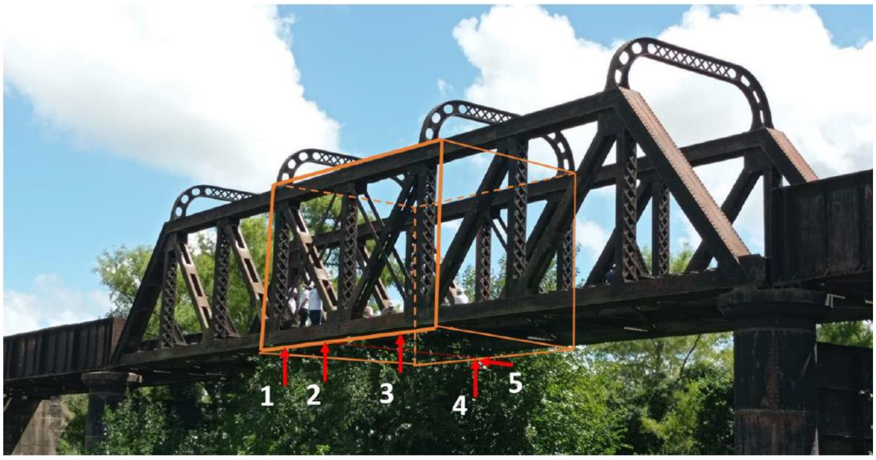


Figure 9: Santa Lucía Bridge with the location of the sensors and orientation of its axis – Bridge test.

Figure 10:Stringer beams analysed in laboratory.

The experiments involved impacting the centre of the beam's web with a rubber hammer as an impulse signal and tests under ambient vibrations. Eight accelerometers were placed on the upper face of the web, aligned along its longitudinal axis (specifically, they were offset to one side of the longitudinal axis, approximately 1 cm away, to allow for the placement of RDA sensors on the other side of the axis). The following is a diagram depicting the experimental setup, indicating the accelerometer positions and the impact location.

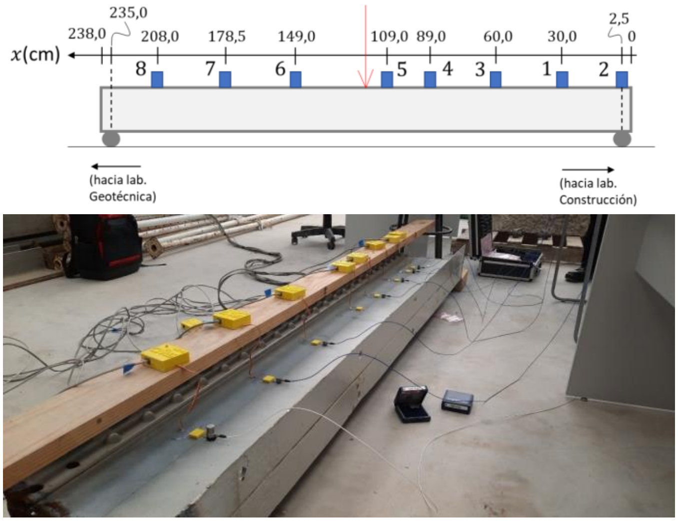


Figure 10: Location of the sensors – Stringer beam test

The analysis and characterization of the structures involved the use of operational modal analysis techniques, specifically peak picking (in the frequency domain) and covariance-driven stochastic subspace identification (in the time domain). These methods assume that the excitation is white noise, a hypothesis that tends to hold true when the excitation is broadband, meaning it has energy distributed uniformly across a wide range of frequencies [7-8]. Additionally, experience shows that impulsive signals yield favourable results with these methods, even though they are non-stationary [9]. Hence, both types of excitation sources were employed in both structures during the testing. In the initial stage, additional processing including windowing and digital filters were utilized in order to minimize the effect of secondary vibrations, windowing algorithms were used to process the measured signals.

4.4 Results and discussion

Bridge Test

From the analysis of the tests, a series of Power Spectral Densities (PSD) were obtained, and eigen-frequencies and eigen-modes were identified. Five natural frequencies with their corresponding modes of vibration can be identified: two lateral, one vertical, and two torsional. The frequencies vary from 4.09 to 57.90 Hz, which are within the expected range for a bridge of these characteristics. It is important to note that the second lateral mode could not be identified in the FING system, presumably because it is located in an antinode of the mode and then marginally excited.

Table 4 presents the results from both systems and the difference between them. It can be observed that, apart from the transverse vibration mode, the difference between them is very small, with values less than 1%.

Table 4: Comparison of natural frequencies obtained.

ID	Mode Shape	Frequency (Hz)		Difference	
		FING	CMS	Absolute	Relative
1	Lateral	4.3	4.09	0.22	5.12%
2	Vertical	9.24	9.19	0.05	0.54%
3	Lateral	-	16.84	-	-
4	Torsional	17.22	17.22	0	0%
5	Torsional	56.90	57.29	0.39	0.69%

Stringer Beam Test

The analysis of the stringer beam was carried out in a similar manner. The most consistent results were found for frequencies below 250 Hz, so the analysis focused on the dynamic study within this range. Seven frequencies were identified, and it can be inferred that the first four (29 to 70 Hz) correspond to local modes of the central plate that forms the web of the built-up beam, as they are much lower of what we would expect given the stiffness of the beam. The three higher frequencies (120 to 230 Hz) can be associated with global modes, where the contributions of the wings become predominant in the overall stiffness.

Table 5 presents a summary of the results of both systems and the difference between them. It can be observed that, apart from the third mode of vibration, the difference between them is very small, with values less than 0.50%.

Table 5: Comparison of frequencies obtained.

ID	Mode Shape	Frequency (Hz)	Difference		
		FING	CMS	Absolute	Relative
1	Local	29.11	29.14	0.03	0.10%
2	Local	33.14	33.03	0.11	0.33%
3	Local	50.01	46.39	3.62	7.24%
4	Local	70.77	70.82	0.05	0.07%
5	Global	120.1	119.8	0.3	0.25%
6	Global	164.3	164.8	0.5	0.30%
7	Global	228.5	227.9	0.6	0.26%

4.5 Conclusions

In this study, we successfully implemented an operational modal analysis technique that usually used in structural health monitoring systems. The obtained values fell within the theoretically expected range for such structures, affirming the reliability of the approach. Additionally, we achieved the validation of a low-cost vibration measurement system, consistently producing results that closely matched those obtained by a state-of-the-art commercial system, with differences generally below 1%. This successful validation highlights the potential of affordable alternatives in the field of structural analysis and monitoring. The findings from this work emphasize the feasibility of deploying cost-effective monitoring systems for structural health assessment, offering a practical and accessible solution for the evaluation and maintenance of structures.

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