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COMPUTATIONAL MODEL UPDATING OF SIMPLE BEAMS USING EXPERIMENTAL MODAL DATA

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Abstract. *Structural Health Monitoring (SHM) is a manner to constantly evaluate a structure's state during its lifespan. Non-destructive tests are recommended in those cases once destructive ones do not allow for the repetition of the tests. The procedure of obtaining and evaluating modal parameters (such as natural frequencies and mode shapes) from a given structure consists in a good tool to perform SHM, since those modal results are an inherent characteristic of the system. Brazilian companies in the electricity distribution field have been showing interest in the use of SHM based on modal parameters control to evaluate the structural integrity of overhead reinforced concrete electricity distribution poles. The main goal is to contribute to safe operational actions and also to protect third-party agents affected by those structures. This paper is part of a program that seeks to propose a modal analysis basis to develop a SHM methodology of reinforced concrete electricity distribution poles. Experimental reinforced concrete beams, subject to different kinds and levels of chemical induced deterioration over time, were constructed and experimentally tested in order to update the finite element computational models (built at Ansys Workbench software). The computational models were calibrated and validated by the dynamic Young's Moduli of concrete. From these experimental tests and updating processes, it was possible to notice that different kinds of chemical deterioration (such as chlorite or sulfate) do not represent a variation tendency over time, however, it was noticed that one of the deteriorations (chloride) reduces the Young's Moduli while the other (sulfate) increases the same parameter. Besides that, the chloride action showed itself prevalent over sulfate action when the combined action is evaluated.*

Keywords: *Structural Health Monitoring, Modal Analysis, Reinforced Concrete, Finite Element, Vibration.*

1. INTRODUCTION

Reinforced Concrete (RC) structures, during their useful lifespan, are exposed to deterioration processes that can reduce its structural properties beyond the allowable limits. When this scenario is reached, the structure uses are compromised, which can lead to both economic losses and risks to human life. During the last few years, some private energy supplying groups in Brazil have been registering some incidents related to reinforced concrete light poles. These light pole structures have been presenting failure during the execution of periodic maintenance, where the unwanted context has been motivating increased interest on the application of periodic structural evaluation.

The process of implementing a system of damage identification strategy is referred to as Structural Health Monitoring – SHM. The basic guideline processes to apply a SHM system involves the application of constant time-spaced measurements, the extraction of interest data and the elaboration of a statistical analysis to predict the state health from the element (Farrar and Worden, 2007). By that system application the objective is to determine the structure integrity level at any period over its lifespan, considering both specific parts and the whole structure. Besides that, according to Nagarajaiah and Erazo (2016), the SHM system implementation seeks to provide an economic cost, among other scenarios, on the maintenance of civil structures. For these authors, the objective is about guaranteeing a safe operational condition during the useful life of the structure and to reducing the impact of eventual failure.

As further visual analysis, there are the destructive and non-destructive tests that are widely used to evaluate the civil engineering elements and constructions. According to Juliani (2014), non-destructive tests are more recommended than the destructive ones, once they are not invasive and can be repeated any time during a structure's useful life to evaluate properties of interest over time, leading to a correct manner of application of a regular SHM system. In this scenario, the dynamic modal analysis consists of a well-developed and used SHM tool for civil engineering problems (Farrar et al., 2001). Modal analysis can provide a non-destructive diagnose of damage (such as cracks), including its position and magnitude. This is achieved once the structural modifications that result from any kind of degradation are expected to directly interfere on oscillatory modal parameters: natural frequencies, vibration modes and modal damping factors. As presented by Ewnis (2000), one of the most popular non-destructive vibration tests is defined as Modal Testing. Where it is realized executed in controlled conditions, by performing the tests in a laboratory and obtaining more accurate and detailed information. Also, data acquisition and its analysis take part of its test.

The RC structures evaluation considering the oscillatory modal parameters is well used and is applicable from a range of small and simple beams (Ndambi et al., 2002) to complex structures such as bridges (Cunha and Caetano, 2006) and dams (Sevim et al. 2009). Additionally, the studies proposed by these mentioned works are respect to, computational analysis, experimental measurements and the correlation between them, respectively.

Therefore, the objective of this study is to induce chemical deteriorations processes in RC beams, extract experimental modal data and calibrate computational models of the RC beams to obtain a representative computational model of the real structure. Another objective is to evaluate the influence of different chemical deteriorations on the RC beams' modal parameters and its influence on the computational model calibration. The computational models' calibration is based on the evaluation of the concrete dynamic Young's Moduli behavior by comparing the RC beams, subject to different forms of deterioration, from an initial condition using laboratorial data and computational models of dynamic modal analysis. After that, the understandings acquired in this paper will be used on computational models of the actual overhead distribution pole's structure.

2. EXPERIMENTAL APPARATUS WITH ACQUIRED DATA AND FINITE ELEMENT MODELING

The experimental campaign consists of four identical RC beams. All four beams were constructed following the schemes presented on Fig. 1. Besides that, the experimental RC beams were submitted to four different cases of induced chemical deterioration over time, as follows:

Case 1 – Reference beam (without any special kind of deterioration);
Case 2 – Beam submitted to the action of chloride deterioration;
Case 3 – Beam submitted to the action of sulfate deterioration;
Case 4 – Beam submitted to the action of both chloride and sulfate deterioration.

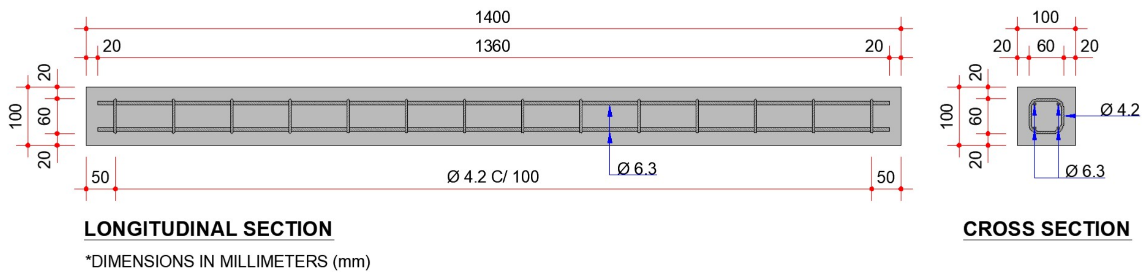

Figure 1. RC beam project considered.

The four beams were periodically (every fortnight) tested in order to measure the variation of the natural frequencies over time. The beams were suspended by their center of gravity in order to reproduce a free-free boundary condition. The beams were submitted to hammer impact excitations where four accelerometers were positioned over the beam (two in each direction from the cross-section and at the same side) in order to capture the first three natural frequencies, as illustrated on Fig. 2.

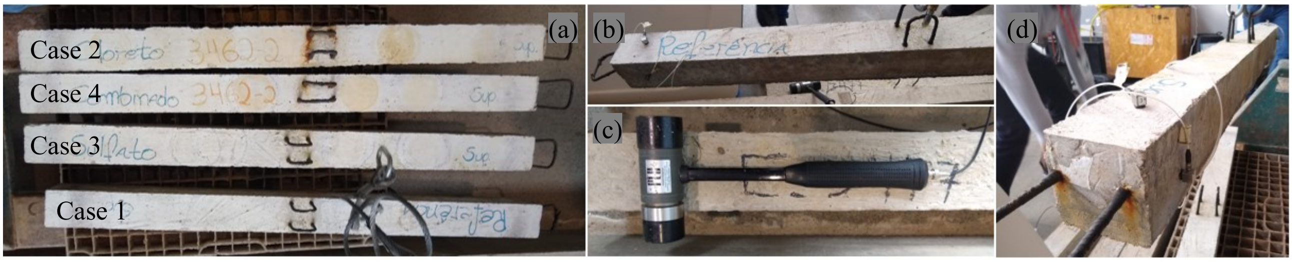

Figure 2. Detail illustration of the (a) four constructed RC beams, the (b) free-free boundary condition, the (c) hammer impact and the (d) accelerometers used on experimental tests.

The RC beams were modelled using Finite Element (FE) software, where the commercially available Ansys Workbench software (version 18.1) was used to construct the numerical models. This software was selected due to its capacity for dynamic analyses and its vast quantity of element types. Aiming to represent computationally the RC beams, their geometry conditions were followed as mentioned in Fig. 1 and their specific RC beams properties were adopted as presented in Tab. 1. The established properties mentioned on Tab. 1 were selected considering the Brazilian reinforced concrete standards from ABNT NBR 6118 (2014) and structural steel standards from ABNT NBR 8800 (2008). Besides that, the concrete density was adopted from values obtained by laboratory tests (Lobo, 2019). The concrete used on the constructed RC beams were periodically tested for its physical and mechanical properties. In spite of the over time changes encountered on the Poisson's ratio of the four proposed RC beams, as described by Pinkoski (2019) (see Fig. 3), that variation was not considered once this property did not interfere on a relevant way to the main EF model dynamic parameter evaluated at this paper (natural frequency).

Table 1. (a) Concrete and (b) Structural Steel physical and mechanicals properties considered for computational model.

Concrete (a)		**Structural Steel (b)**		
Poisson's Ratio	Density (kg/m³)	Young's Moduli (GPa)	Poisson's Ratio	Density (kg/m³)
0.20	2300	200	0.30	7850

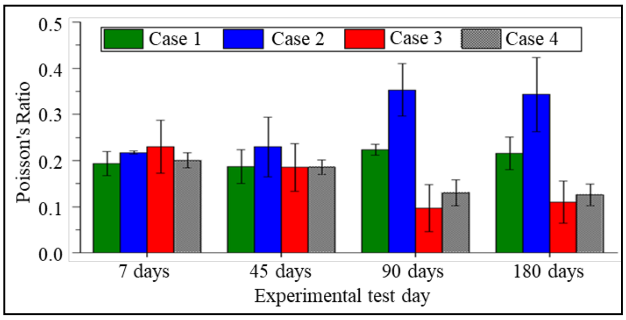

Figure 3. RC beams experimental Poisson's ratios.
Source: Pinkoski (2019).

Using the Modal Analysis Module in Ansys Workbench, the beam computational model was constructed considering the SOLID186 element type to represent the concrete and the BEAM189 element type to represent structural steel. The tridimensional SOLID186 element type has 20 nodes with 3 degrees of freedom (DOF) in each one, being these DOF the *x*, *y* and *z* axes translations. The tridimensional BEAM189 element type has 3 nodes with 6 DOF in each one, where these DOF are the *x*, *y* and *z* axes translations and rotations. The mentioned and used element types are exemplified on Fig. 4a and Fig. 4b.

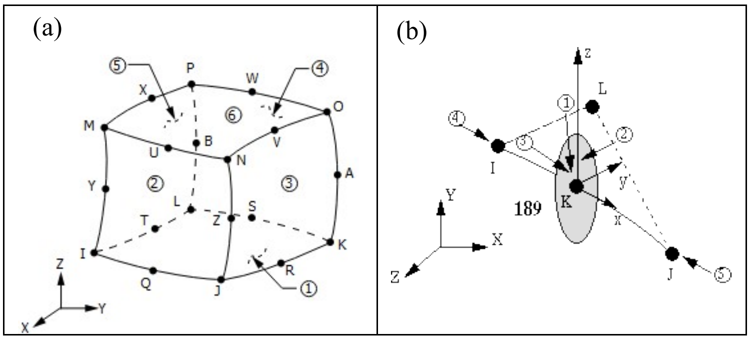

Figure 4. Elements type caractheristics from (a) SOLID186 and (b) BEAM189.
Source: adapted from Mechanical APDL Theory Reference (2019).

By implementing those element types in the RC beams computational models, the proposed mesh for this structure presented a total of 18613 nodes and 4717 elements, where the elements were previously set up to be 30 mm long. For the FE model, a nodal coincidence between the proposed elements was considered, where this coincidence is important to simulate a complete adherence between the concrete and the structural steel.

After computationally implementing the RC beams geometry, the last FE modeling stage is the model updating. According to Sevim et al. (2009), it is common to notice some discrepancies between experimental and analytical natural frequencies that are consequences of some FE method uncertainties. According to those authors, these uncertainties may come from material properties, boundary conditions, element types, meshing sizes and from experimental conditions. To the RC beams analyzed in this paper, the model updating is given by the concrete dynamic Young's Moduli, until the first analytical natural frequency is shown to be equivalent to the experimental natural frequency. This process was used to all four RC beams. The FE model characteristics are illustrated in Fig. 5 including the first and third natural transversal flexural mode shapes.

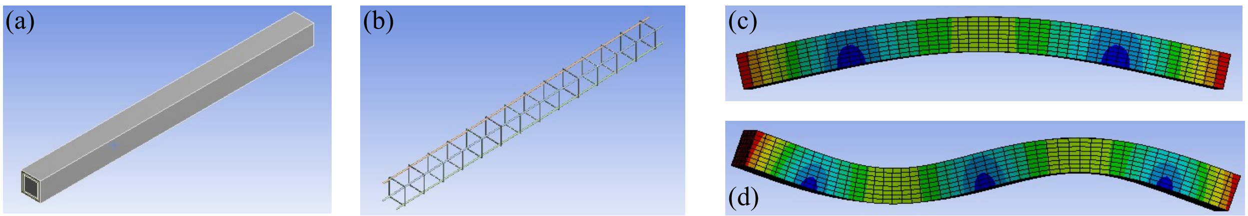

Figure 5. An example of (a) concrete geometry, (b) structural steel positioning, (c) first and (d) third natural mode shapes from a FE RC Beam computational model.

3. RESULTS

The resulted data from the 282 days of periodical tests are discussed here. Included on this paper are an analysis results set on the differences between the first three natural frequencies of the experimental tests, from the calibrated FE model, and an analysis about the behavior and relationship of the concrete dynamic Young's Moduli of the laboratorial tests, from the calibrated FE model, on the four chemical deterioration cases. As previously mentioned, the FE models updating was developed looking for the concrete dynamic Young's Moduli that best approximates the experimental and computational first natural frequency.

In order to fulfill that objective, the biggest error (in module) from each RC beam model and the comparison between the first three natural experimental and computational frequencies and presented on Tab. 2 and Tab. 3, respectively.

Table 2. Maximum error from experimental and analytical at first natural frequencies of RC beams.

Beam type	1st frequency	2nd frequency	3rd frequency
Case 1	0.02%	5.15%	4.19%
Case 2	0.02%	12.83%	12.08%
Case 3	0.01%	16.17%	10.34%
Case 4	0.01%	12.04%	9.65%

As the maximum error to the first natural frequency calibration was almost zero in all the available cases (Tab. 2), it is possible to consider the calibration process satisfactorily performed to all the test period (282 days).

Table 3. Avarege comparison of the experimental and analytical three first natural frequencies after the computational model calibration.

Beam type	Origin data	1st frequency	2nd frequency	3rd frequency
Case 1	Computational	210.42	210.42	561.63
	Laboratorial	210.42	218.06	581.21
	Percentual Variation	**0.00%**	**3.50%**	**3.37%**
Case 2	Computational	203.60	203.61	543.34
	Laboratorial	203.60	219.89	580.96
	Percentual Variation	**0.00%**	**7.40%**	**6.48%**
Case 3	Computational	214.32	214.32	572.08
	Laboratorial	214.33	241.24	598.80
	Percentual Variation	**0.00%**	**11.16%**	**4.46%**
Case 4	Computational	209.30	209.30	558.62
	Laboratorial	209.30	221.20	574.70
	Percentual Variation	**0.00%**	**5.38%**	**2.80%**

On the average comparison (Tab. 3), presented no variation at the first natural frequency in all cases. Besides that, the percentual variation were about 6.86% for the second natural frequency and 4.28% for the third natural frequency.

The updating process used in this work can be considered effective since the errors on the first frequency (listed on Tab. 2 and Tab. 3) are minimal. However, the errors for second and third frequencies show themselves more significant, which is expected on discretization-based results for higher modes (Lobo et al., 2018). To the second frequency, the error is not explained at the same way, because it was expected to be equal to the first frequency (both the frequency and error) considering the symmetrical inertia condition given by the used cross-section. Here the error's main source is a consequence from experimental tests data variation, which sometimes does not guarantee the symmetrical conditions. The model updating process, that allowed the calibration and comparison in frequency presented above, were achieved by concrete dynamic Young's Moduli update over time, which are presented on Fig. 6, to all proposed cases.

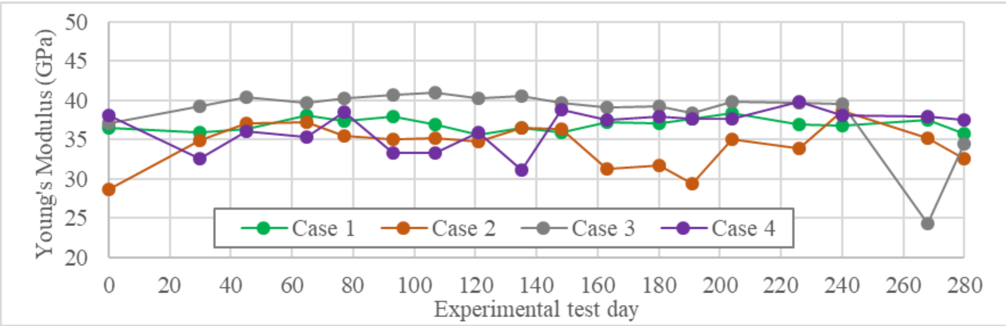

Figure 6. Comparison of computational calibrated concrete dynamic Young's Moduli evolution in time to all cases.

Considering the computational calibrated concrete dynamic Young's Moduli evolution in time presented in Fig. 6, it is not possible to identify any consistent variation tendency to the mentioned property with respect to over time induced chemical deterioration processes. Considering Case 1 as a reference structure, it is possible to note that on Cases 2 and 4, where the beams are submitted to the action of chloride deterioration and to the action of both chloride and sulfate deterioration, respectively, the concrete dynamic Young's Moduli are quite close but with smaller values. On the other side, Case 3, where the beam is submitted to the action of sulfate deterioration, the concrete dynamic Young's Moduli are bigger than the reference. By this scenario it is possible to suggest that, on the proposed chemical deterioration cases, the chloride action is more prevalent than the sulfate one on the concrete dynamic Young's Moduli behavior.

In order to present a comparison between computational and laboratorial data, the next four figures (Fig. 7 to Case 1, Fig. 8 to Case 2, Fig. 9 to Case 3 and Fig. 10 to Case 4) show the concrete dynamic Young's Moduli used to calibrate the FE model (in blue) and the concrete dynamic Young's Moduli measured in laboratory tests (in red). The measured concrete dynamic Young's Moduli were obtained and first presented by Pinkoski (2019) from equivalent concrete cylinders of 300 mm height and 150 mm diameter.

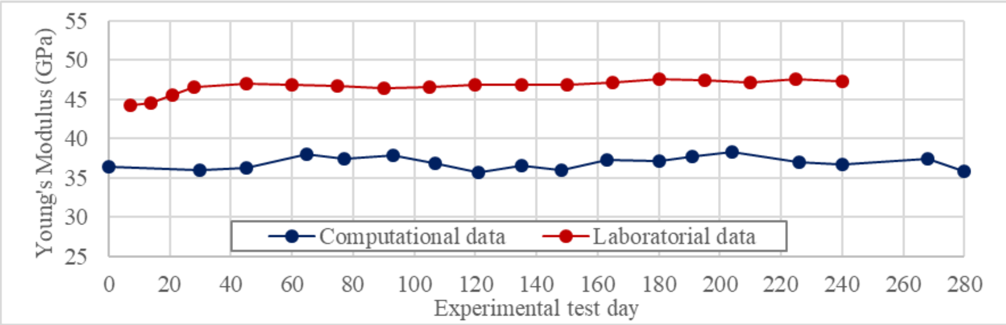

Figure 7. Comparison of the laboratorial and computational calibrated concrete dynamic Young's Moduli evolution in time to Case 1.

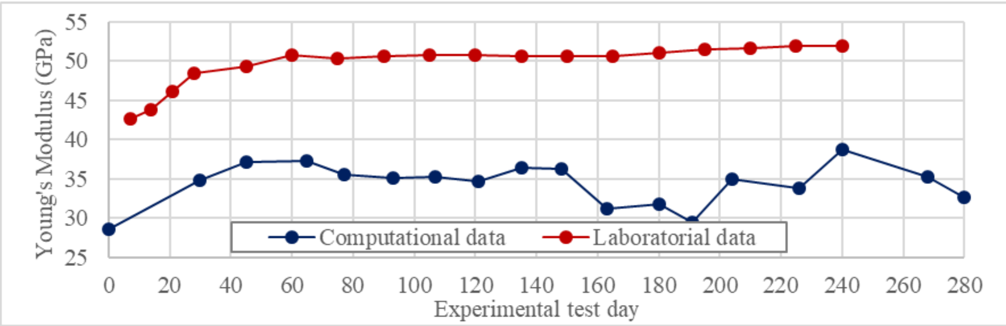

Figure 8. Comparison of the laboratorial and computational calibrated concrete dynamic Young's Moduli evolution in time to Case 2.

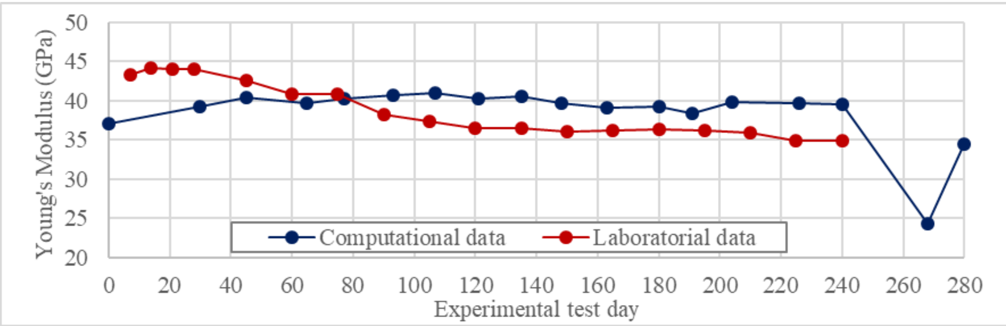

Figure 9. Comparison of the laboratorial and computational calibrated concrete dynamic Young's Moduli evolution in time to Case 3.

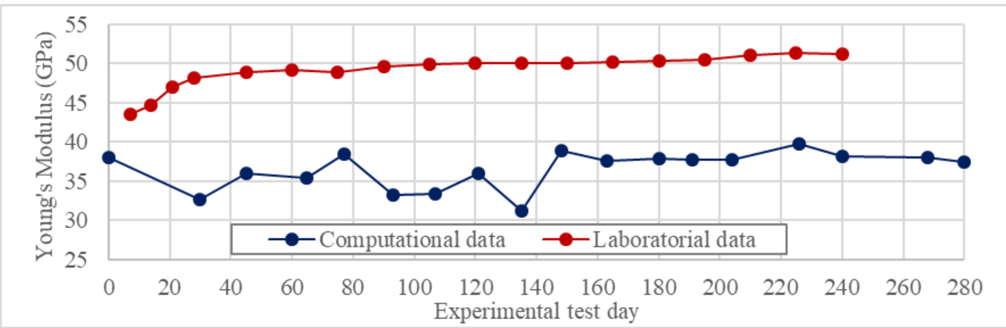

Figure 10. Comparison of the laboratorial and computational calibrated concrete dynamic Young's Moduli evolution in time to Case 4.

As presented first in Fig. 6 for the computational data, either the laboratorial data (in red) presented on the Fig. 7 to Fig. 10, it is not possible to identify any consistent variation of the concrete dynamic Young's Moduli tendency with respect to over time induced chemical deterioration processes. For Case 1 (Fig. 7) the computational data is about 10 GPa lower than the laboratorial values, where this laboratorial data is growing on the first 28 days and remains constant after that. For Case 2 (Fig. 8) the computational data is about 15 GPa lower than the laboratorial data, where this laboratorial data ascends on the first 28 days and keeps almost constant after that. For Case 3 (Fig. 9) the computational data is approximately equal to the laboratorial one, where this laboratorial data is almost constant on the first 28 days and suggests a small reduction after that. Finally, for Case 4 (Fig. 10) the computational data is about 13 GPa lower than the experimental, which grows on the first 28 days, keeping a behavior close to constant after that point.

After comparing the computational and the laboratorial data, it is possible to notice that the relationship between these data sets are similar on Cases 2 and 4 and completely different from Case 3, in accordance with the results discussed on Fig. 6. Besides that, the non-indication of a tendency of variation to the concrete dynamic Young's Moduli on the evaluated cases suggests that the over time induced chemical deterioration processes (either in computational or laboratorial data) do not act to constantly reduce de RC beams rigidity.

In a complementary way, on Tab. 4 an average abstract from the Fig. 7 to Fig. 10 data is presented. This table shows a concrete dynamic Young's Moduli average, the percentage difference between computational and laboratorial data and the standard deviation.

Table 4. General comparison of the laboratorial and computational calibrated concrete dynamic Young's Moduli.

Beam type	**Origin data**	**Average**	**Average range**	**Standard deviation**
Case 1	Computational	36.94	-20.8%	0.80
	Laboratorial	46.64		0.95
Case 2	Computational	34.40	-30.7%	2.71
	Laboratorial	49.66		2.70
Case 3	Computational	38.56	-0.7%	3.87
	Laboratorial	38.83		3.50
Case 4	Computational	36.54	-25.7%	2.43
	Laboratorial	49.16		2.14

On Tab. 4 is possible to note the percentage variation from the computational to the laboratorial data that are, in module, about 20.8% in Case 1, 30.7% in Case 2, 0.7% in Case 3 and 25.7% in Case 4. Additionally, the concrete dynamic Young's Moduli and the standard deviation shows themselves similar to the presented and discussed between Fig. 7 to Fig. 10 data.

4. CONCLUSIONS

Considering the calibration process to the RC beams prosed on this paper, evaluated according to natural frequencies, it was possible to conclude that the computational models can successfully represent the experimental results. Since the errors on first frequency are minimal, the second frequency presented discrepancies from the experimental tests and the third one has an expected variation. These calibrations were performed adjusting only the concrete dynamic Young's Moduli, once the Poisson's ratio did not significantly interfere on the obtained modal analysis results.

The calibrated concrete dynamic Young's Moduli evaluation, in the four presented cases, did not indicated a variation tendency to the mentioned property with respect to over time induced chemical deterioration processes. These results, to this work next steps, reveal a difficulty related to the use of modal parameters variations (such as natural frequency) to predict a reduction of the physical properties (rigidity), by chemical deterioration, as a step of a SHM system implementation.

On the four proposed RC beams, it was possible to notice that on the proposed chemical deterioration cases, the chloride action is more prevalent than the sulfate action on the concrete dynamic Young's Moduli behavior. Its appointed once the beam submitted to the action of chloride deterioration and beam submitted to the action of both chloride and sulfate deterioration presented frequencies and, consequently, concrete dynamic Young's Moduli smaller than the reference beam did (without any kind of chemical deterioration), while the beam only submitted to the action of sulfate deterioration showed properties with higher values when compared with the reference beam.

On this research continuity, the main next steps are about considering the computational model of an electricity distribution pole including its more complex geometry and boundary conditions. On this way, the results presented on this paper will allow for a correct calibration from its structure. After that, to the electricity distribution pole structure, other steps on this research line intend to evaluate the influence on modal parameters of a parametric variation from conditions such as the buried level, kind of soil, induced damages and others.

5. ACKNOWLEDGEMENTS

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