Experimental And Numerical Modal Analysis Of A Steel Frame

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Abstract. In engineering, it is extremely important to direct research towards the identification and understanding of dynamic behavior in structures, as dynamic loads are present in all types of structures, including civil construction. Dynamic loads can range from the interaction of wind with the structure to people's footsteps and car movements, causing vibrations. These vibrations can cause damage to the structure and users, depending on their intensity and duration. For this reason, structures are dimensioned and designed to support all the efforts required, incluing vibrations and other dynamic actions. Thus, the present work aims to determine the dynamic properties of a steel frame through a numerical and experimental modal analysis. The steel frame is made of cold-rolled A36 steel. The profiles of the columns and beams are formed by cold rolled profiles forming sections of the type "closed stiffened double U-profile" and "reinforced U-profile", respectively. The beams are joined to the columns by rigid connections. The dimensions of the steel frame are approximately 2.1 m in height by 1 m in width and length. Numerical analysis was performed by the Finite Element Method (FEM) using shell elements. In the experimental analysis, the structure was excited by an impact hammer and the vibration response was measured by piezoelectric accelerometers. The frequency domain Rational Fraction Polynomial Method (RFPM) was used to estimate the natural frequencies, mode shapes and damping factor. There was a good agreement between the experimental results and the numerical values obtained by the proposed procedure.

Keywords: Modal analysis, Finite element, Steel frame, dynamics

1 Introduction

In engineering, it is extremely important to direct research towards the identification and understanding of dynamic behavior in structures, as dynamic loads are present in all types of structures, including civil construction. Dynamic loads can range from the interaction of wind with the structure to people's footsteps and car movements, causing vibrations. These vibrations can cause damage to the structure and users, depending on their intensity and duration. For this reason, structures are dimensioned and designed to support all the efforts required, including vibrations and other dynamic actions [1], [2].

One way to understand the dynamic properties of a structure is to perform a modal analysis. According to Avitabile [3], modal analysis is a process that describes the structure in terms of its natural characteristics: natural frequency, damping factor, and mode of vibration.

Modal analysis involves a complex mathematical formulation to describe dynamic behavior and can be performed

in both the time domain and the frequency domain [4].
In this context, Formenti and Richardson [5] present the advances in modal analysis methods developed since the 1970s. Notable among them are time domain methods: Complex Exponential Method (CEM), Time Domain Polyreference Method (TDPM), and Ibrahim Time Domain Method (ITDM); and frequency domain methods: Circle-Fit, Line-Fit, and RFPM.
Thus, this work aims to determine the dynamic properties of a rigidly "reinforced U-profile" steel frame. To achieve this, both experimental and numerical modal analyses were conducted. The experimental modal analysis method used was the RFPM, while the numerical modal analysis will be performed using the commercial Ansys® software.

2 Rational Fraction Polynomial Method (RFPM)

The RFPM is an experimental modal analysis method used in the frequency domain. This method is based on the work of Ewins [4] and Iglesias [6], and its formulation is as follows:

$$H(\omega) = \sum_{r=1,N} \frac{A_r + i\omega B_r}{(\omega_r^2 - \omega^2 + 2i\omega\omega_r\zeta_r)} \quad . \tag{1}$$

where A_r and B_r are constants, ζ_r is the damping ratio and ω_r is the natural frequency that correspond to mode r.

The difference between the analytical *H(ω)* and the experimental Frequency Response Function (FRF) $H_e(\omega)$ is expressed by the error function, defined as:

$$e_j = \frac{\sum_{k=0}^{2N-1} a_k (i\omega_j)^k}{\sum_{k=0}^{2N-1} b_k (i\omega_j)^k} - H_e(\omega_j) \quad . \tag{2}$$

where the first term of equation (2) is equation (1) expressed in rational fraction form.

However, the error function is linearized using the modified error function presented below:

$$e_j^{'} = e_j \sum_{k=0}^{2N-1} b_k (i\omega_j)^k \tag{3}$$

by setting $b_{2N} = 1$, the following is obtained:

$$e_j^{'} = \sum_{k=0}^{2N-1} a_k (\mathrm{i}\omega_j)^k - H_e(\omega_j) \left[\sum_{k=0}^{2N-1} b_k (\mathrm{i}\omega_j)^k + (\mathrm{i}\omega_j)^{2N} \right] . \quad . \tag{4}$$

Thus, the error vector can be written as follows:

$$\{E\}_{Lx1} = [P]_{L\times 2N} \{a\}_{2N\times 1} - [T]_{L\times 2N} \{b\}_{2N\times 1} - \{W\}_{L\times 1} \tag{5}$$

where P, T and W are matrices in which each row corresponds to the measured frequencies (L).
This error function can be minimized using the quadratic gradient method or the least squares method [6].

3 Methodology

3.1 Experimental Setup

In this experiment, a 1020 steel structure was employed, consisting of a 1-meter beam at the top and two 2.1-meter columns, as illustrated in Figure 1. The physical and geometric properties of this structure are detailed in Table 1. The structure was anchored to the ground using a base plate, simulating fully fixed boundary conditions at the bottom during the measurements. The base plate was secured to the structure with bolts and welds, ensuring its stability. The configuration adopted for the modal analysis of the steel structure is presented in Figure 1. A total of 24 points were measured to capture the vibration modes in the z-direction, with transverse modes not being considered in this study.

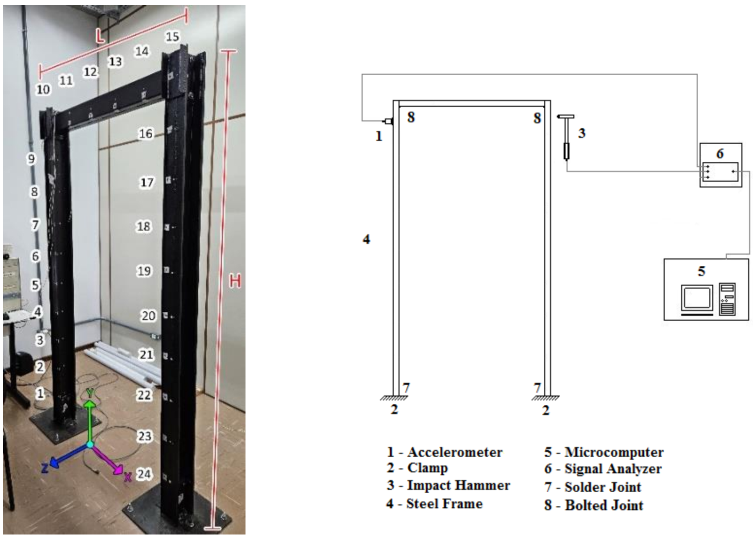
Table 1. Material and geometric properties of the steel frame.

Steel Frame properties	Numerical values
Length (L)	1 m
High (H)	2.1 m
Mass density (ρ)	7850 (kg/m$^{3)}$
Young`s modulus (E)	208 (GPa)
Poisson`s ratio (ν)	0.3 (-)
Mass moment of inertia of the beam (I_v)	1.449 (kg.m^2)
Area moment of inertia of the column's cross-section (I_x)	1.135×10^{-6} (m^4)

Data acquisition was carried out using a dynamic analyzer, with a sampling frequency set at 512 Hz, which is sufficiently high to avoid aliasing issues in the analyzed frequency range of 0 to 200 Hz. The input and response signals were processed using a force window and an exponential decay window, respectively. The measurement resolution was set to 0.25 Hz.

An average of five data acquisitions was performed, ensuring a minimum coherence of 90% for each FRF, with these values used as the reference for point measurements. The structure was excited with an impact hammer from node 1 to 24 in the negative z-direction. The impact hammer was equipped with a load cell with a sensitivity of 0.2506 mV/N, responsible for detecting the magnitude of the excitation force. A nylon tip was used to generate an appropriate spectrum in the frequency range of 0 to 200 Hz.

The vibration response to the impact was measured at node 9 with a piezoelectric accelerometer positioned perpendicularly to the structure, so that only accelerations normal to the surface were recorded.



(a) (b)

Figure 1: Steel frame: (a) Experimental setup and (b) Schematic view.

3.2 Experimental Procedure

An impact hammer was used to excite the structure. Modal parameters, such as eigenvalues and eigenvectors, are complex functions due to the presence of damping. These parameters, along with damping factors, were determined using the EasyMod Toolbox, developed by Kouroussis et al. [7], [8]. The method, based on the RFPM parameter estimation theory [5], [9], was implemented using the same variable standard employed in EasyMod.

3.3 Numerical Procedure

The numerical modeling was performed in Ansys®, which uses the FEM. The finite element used was the SHELL181 type, with four nodes and six degrees of freedom at each node. The natural frequencies and vibration modes of the structure were obtained through modal analysis, and the FRF of the accelerance was obtained through harmonic analysis using the modal superposition method. The damping factor (ζ) found in the experimental procedure was incorporated into the numerical simulation. The frequency range analyzed was from 0 Hz to 200 Hz.

Table 2 shows the comparison of the results and relative errors with the experimental data for the first four frequencies with three element sizes: 14 mm, 15 mm, and 16 mm. The first, second, and third meshes contain 17552, 15933, and 13458 nodes, respectively. From these results, we conclude that the mesh with 15933 nodes and 14,663 elements is sufficient for the numerical simulation. The mesh with 17552 nodes was not suitable due to the high computational cost. The boundary conditions applied in the numerical simulation were a fixed base with the ground and semi-rigid contact between all structural elements.

Table 2: Natural Frequencies versus Element size

	Element size (mm)					
	14	**Relative error (%)**	**15**	**Relative error (%)**	**16**	**Relative error (%)**
1ª Frequency (Hz)	14.316	0.03	14.3	0.14	14.306	0.1
2ª Frequency (Hz)	22.371	0.27	22.308	0.01	22.273	0.17
3ª Frequency (Hz)	130.12	0.10	129.98	0.01	150.49	15.77
4ª Frequency (Hz)	150.86	0.19	150.57	0.00	130.05	13.63
Nodes	17552		15933		13458	
Elements	16184		14663		12244	

4 Results and Discussion

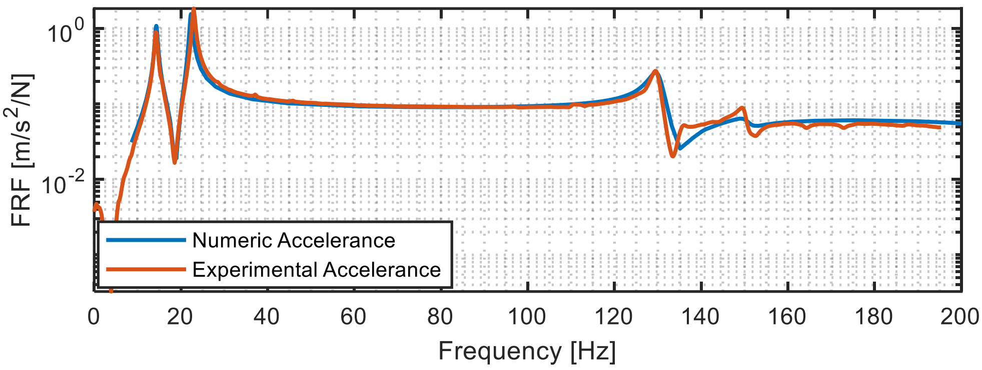
4.1 Frequencies and Damping Ratios

Table 2 presents the natural frequencies and damping ratios for the first four modes. Only the modes in the ZY plane were measured, while vibrations in other planes were disregarded. It is noted that the natural frequencies obtained by the FEM show significant proximity (maximum error of 3.04%) to those from the RFPM.

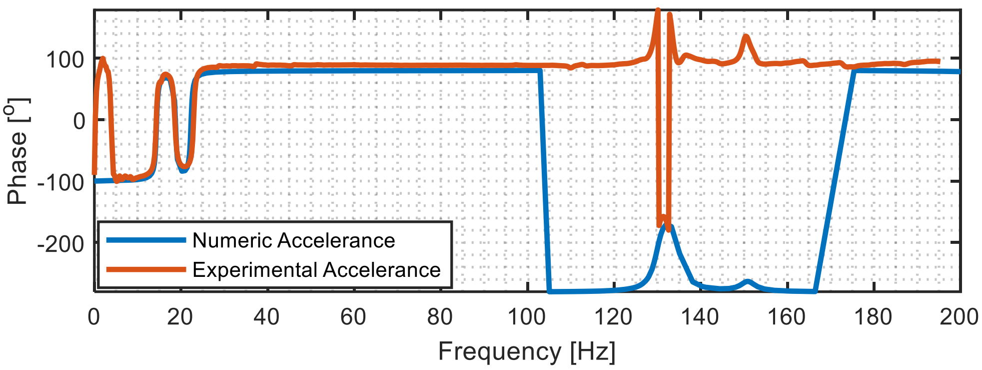
Table 3: Frequencies and Damping ratio

	RFP Method		FEM	Relative Error [%]
	Frequency [Hz]	Damping ratio [%]	Frequency [Hz]	
First mode	14.32	1.9	14.32	0.0
Second mode	23.01	1.3	22.31	3.04
Third mode	130.1	1.4	129.99	0.08
Fourth mode	150.38	2.0	150.57	0.12

Figure 2 shows the FRF for acceleration (a) and phase (b) at point 10. In Figure 2(a), a similar FRF behavior is observed between the experimental and numerical results. In Figure 2(b), a phase shift is noted at approximately 102 Hz. This phase shift is associated with the natural frequency of the structure's transverse mode.



(a)



(b)

Figure 2: Experimental and numerical FRF accelerance at point 10: (a) FRF and (b) Phase.

4.2 Mode Shapes

The RFPM enables accurate identification of vibration modes. Figure 3 illustrates the comparison between the results obtained by the RFPM and the FEM for the first four modal shapes. In Figures 3(a), 3(b), 3(c), and 3(d), the black dashed line represents the structure at rest, while the solid blue line represents the structure in motion in its respective vibration mode.

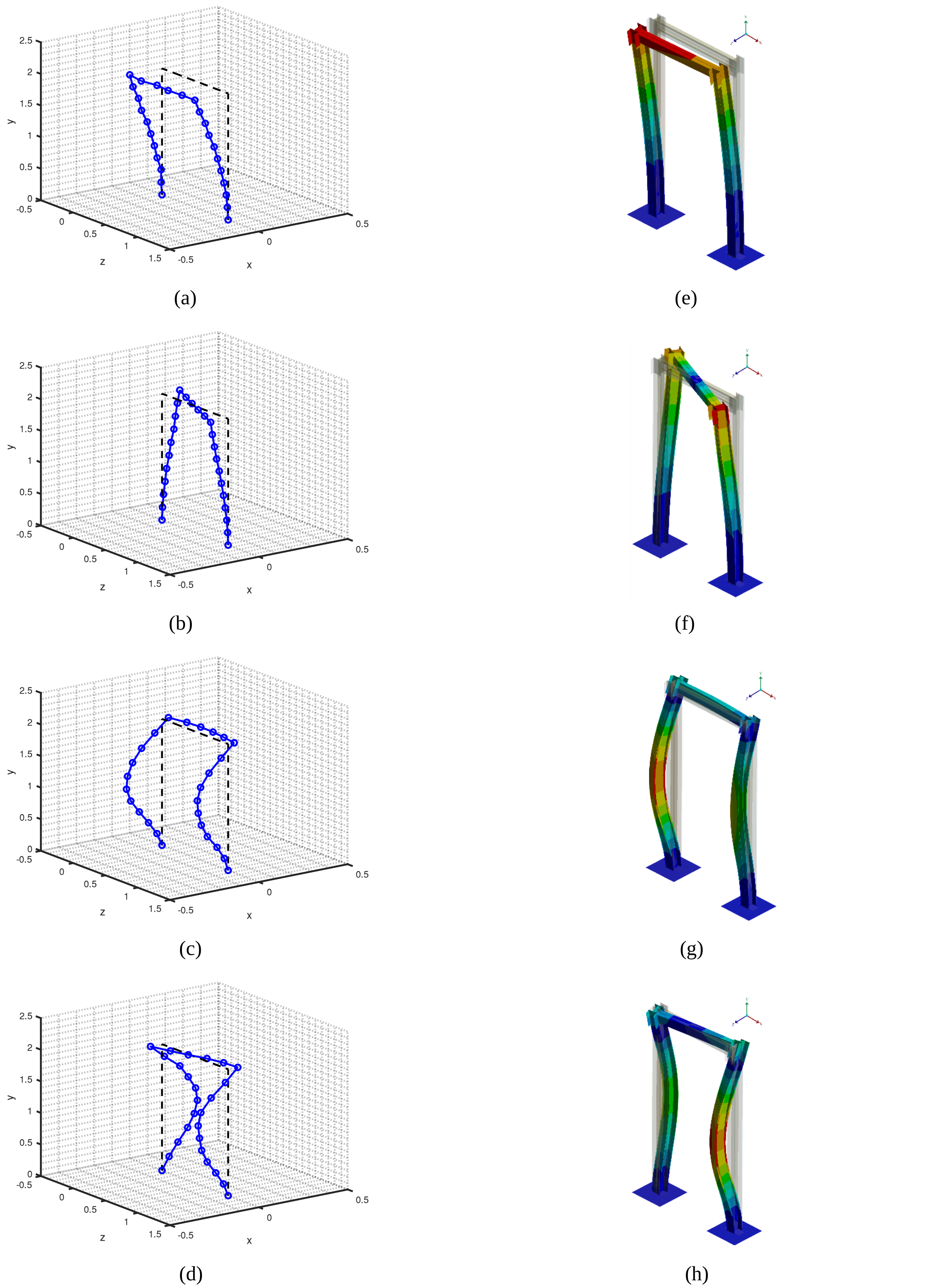


Figure 3: Experimental and numerical mode shapes corresponding to the four first modes. Experimental mode shapes: (a) First mode shape, 14.32 Hz; (b) Second mode shape, 23.01 Hz; (c) Third mode shape, 130.1 Hz; (d) fourth mode shape, 150.38 Hz. Numerical mode shapes: (e) First mode shape, 14.32 Hz; (f) Second mode shape, 22.31 Hz; (g) Third mode shape, 129.99 Hz; (h) Fourth mode shape, 150.57 Hz.

5 Conclusions

This work conducted both experimental and numerical modal analysis of a rigidly “reinforced U-profile” steel frame. For the experimental modal analysis, the RFPM implemented in the EasyMod toolbox was used. The numerical simulation of the frame was performed using the FEM in the Ansys® software.

The damping factors of the structure were determined experimentally using the RFPM and subsequently incorporated into the numerical model.

The comparisons of natural frequencies obtained experimentally and through the numerical method showed larger errors for the second mode of vibration (3.08%). However, the results for the vibration modes and the accelerance FRFs demonstrated good coherence. This is evidenced by the fact that the four vibration modes were similar in both analyses. Additionally, the FRFs exhibited prominent peaks and frequencies with similar behaviors, despite the relative errors in the natural frequency.

Despite the greater discrepancy in the second natural frequency, the procedure used in the numerical model proved to be effective in determining the natural frequencies, representing the vibration modes, and the FRF. In the future, a Tuned Mass Damper (TMD) will be attached to the steel frame to control vibration at the determined natural frequencies.

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