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Assessment of the Dynamic Behavior of Unreinforced Masonry Bell Towers Using Ambient Vibrations and Numerical Modeling – A Case Study

Jimmy-Lee Mc Lellan[i], Rola Assi[ii] and Jean-Philippe Ouellette[iii]

ABSTRACT

While unreinforced masonry (URM) is prevalent in Quebec's cultural and religious heritage, very few practical and simplified methods are available to predict its behavior when subjected to dynamic loading (such as the sounding of the bells), and seismic events. Through a case study of the Notre-Dame Basilica of Montreal (NDB), the dynamic behavior and stresses within the unreinforced masonry bell towers resulting from vibrational forces were assessed using a reliable numerical model developed and calibrated with ambient vibration measurements (AVM). AVMs were acquired using Tromino® in both towers to obtain their dynamic behavior under 3 conditions: i. ambient solicitations without bell ringing; ii. the influence of bell ringing: a carillon with multiple bells for the East tower and a single bourdon for the West tower; iii. the influence of the bell ringing from both towers simultaneously. To complement the dynamic characterization, the bells were also modeled to assess potential resonance with the towers' structural modes and to isolate their vibration effects from the global response. Subsequently, the recorded signals were processed in the modal operational analysis software ARTeMIS® to obtain the tower's modal properties. Finally, a global finite element model comprising both towers and the narthex façade was constructed using the SAP2000® software and was calibrated using the AVM results. After calibrating the model to the first three fundamental vibration modes, the discrepancies between the initial model and the experimental results obtained through the AVMs were reduced by 52.9%, 23.5%, and 5.3%, respectively, for each mode. The calibration and the modeling process provide a deeper and more realistic understanding of the dynamic behavior of unreinforced masonry bell towers, enabling an accurate assessment of internal stresses and the identification of vulnerable locations in terms of structural integrity.

KEYWORDS

Dynamic analysis, unreinforced masonry (URM), churches, ambient vibration measurements (AVM), modal analysis, finite element model, Notre-Dame Basilica of Montreal (NDB).

[i] Master student, École de technologie supérieure (ÉTS), Montreal, Canada, jimmy-lee.mclellan.1@ens.etsmtl.ca
[ii] Associate Professor, École de technologie supérieure (ÉTS), Montreal, Canada, rola.assi@etsmtl.ca
[iii] Co-research director, Cosigma inc., Montreal, Canada, jean-philippe.ouellette@cosigma.ca

INTRODUCTION

Over 714 churches built before 1945 are located within the province of Quebec. The use of bells by the ecclesiastical clergy remains common in Quebec; however, there is limited practical knowledge regarding their impact on the structural integrity of the unreinforced masonry (URM) structures that support them. It is widely acknowledged that these structures display a significant degree of vulnerability, with unreinforced masonry churches identified as among the most susceptible to damage [1]. While unreinforced masonry demonstrates strong resistance to gravitational loads, it is highly vulnerable to lateral and tensile forces, such as those caused by dynamic loads, seismic events and bell ringing.

Vibrations induced by bell ringing, particularly in the towers and façade, are known to affect the structural stability. However, the full extent of the stresses and their impact on unreinforced masonry remains poorly understood due to the limited number of dedicated experimental studies [2]. The architectural design of churches, including mechanisms that generate direct or indirect tensile (bending) forces, makes them prime candidates for analysis, especially considering the height of their bell towers and the grandeur of their façades. In this context, the Notre-Dame Basilica of Montreal, one of the largest neo-Gothic churches in the world, has been selected for this research project.

As part of a rehabilitation project for its towers, preliminary and detailed inspections of the church's structural condition have been conducted since 2017. In 2020, emergency restoration work was carried out on the north façade between the two bell towers. The West tower was restored between 2023 and 2024, and the works involved, amongst others, the replacement of numerous stones and comprehensive repointing. Similar restoration work was planned for the East tower and began in the spring of 2024.

AVMs were conducted across all floors of both 60-meter-high towers, under typical environmental and operational conditions as well as during bell ringing. These measurements were then analyzed using operational modal analysis to characterize the dynamic responses and stresses of the towers. Additionally, the bells were modeled to assess whether their vibration frequencies could induce resonance with the structure. This analysis allowed for the precise calibration of the finite element model.

AMBIENT VIBRATIONS APPROACH

Case study application

Ambient vibration measurements (AVM) were conducted on the West tower after its renovation and on the East tower at the beginning of its renovation. The AVM approach offers significant advantages for evaluating structures under typical operational conditions, as it simplifies the measurement process and reduces costs compared to traditional forced vibration techniques.

AVMs mainly provide critical insights into a building's dynamic characteristics within the elastic domain, under real-world conditions, and help refine finite element models, which often fail to encompass all structural details [3].

Instrumentation and protocol

In this study, five synchronized Tromino® devices, each equipped with radio antennas and amplifiers, were used to capture AVMs. The velocimetric channels exhibited superior sensitivity compared to the accelerometric channels; therefore, all deployed setups employed the velocimetric channels for enhanced measurement accuracy. These sensors are capable of measuring velocities from microtremors within a range of ±0.05 mm/s [4]. Each acquisition lasted between 20 and 60 minutes, and the signals were recorded within a frequency range of 0 to 128 Hz. Before the initiation of the AVM recording process, a reference point was established on either the roof or the ground floor to standardize all acquisitions. For all experimental setups, one Tromino® was designated as the reference and remained stationary, while the remaining devices, referred to as rovers, were deployed at the four corners of each floor to capture translational and torsional displacements. After each test, the rovers were repositioned to the next floor to capture the response corresponding to the next degree of freedom of interest, ensuring synchronization with the reference point.

A total of N = 30 experimental setups were conducted, comprising 15 setups under ambient conditions and 15 setups influenced by the bell ringing. Comprehensive measurements were obtained across all floors of both the East and West towers. These setups enabled the assessment of the building's dynamic characteristics, including the determination of natural frequencies, damping ratios, and modal shapes. Figure 1 provides a schematic representation of the various sensor configurations used within the structure.

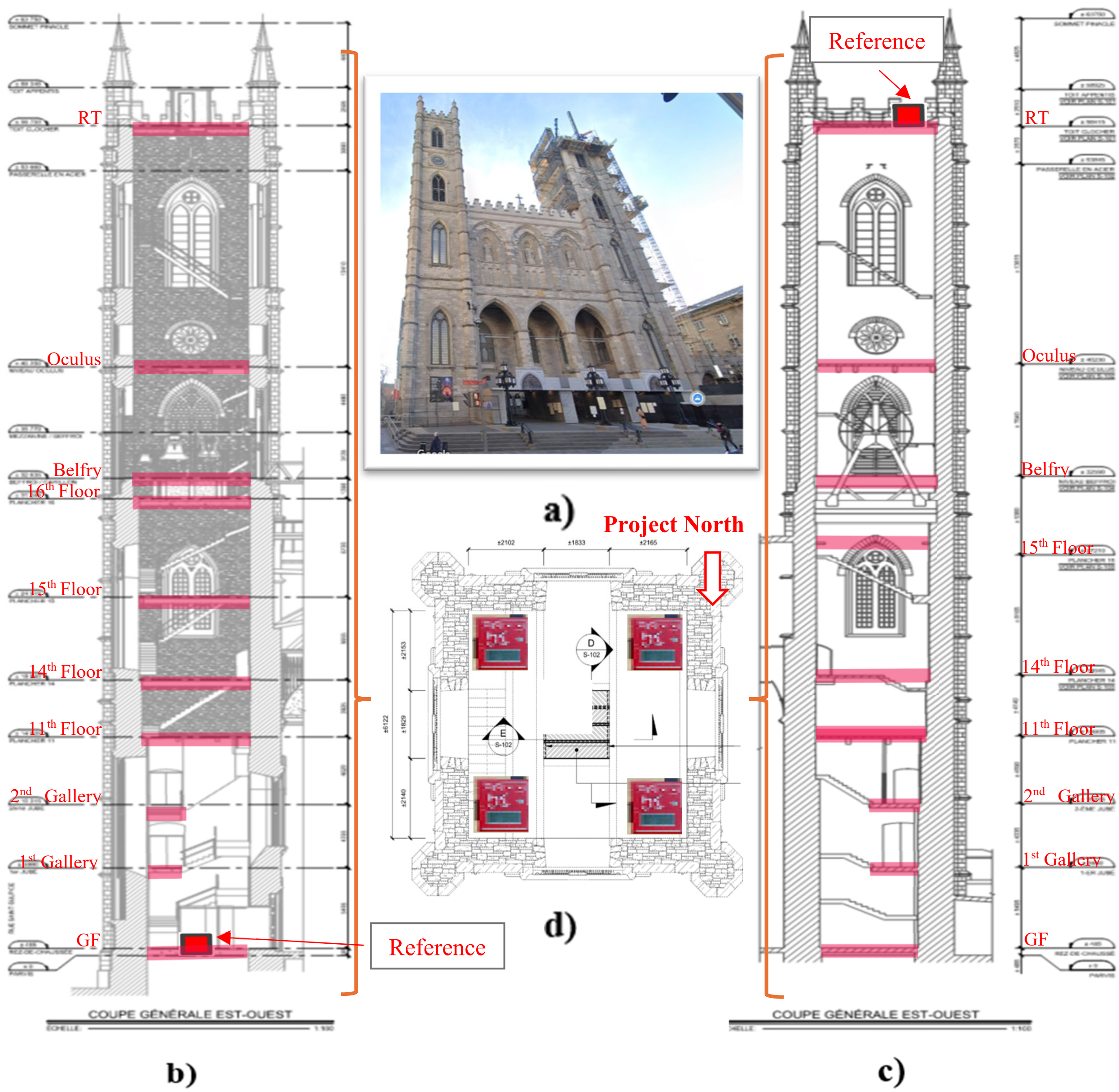

Figure 1: a) NDB front view; Elevations of: b) East tower; c) West tower; d) Plan view: example of configurations (setups) of Tromino®s on each floor

MODAL ANALYSIS BY NUMERICAL MODELING

Data extraction and processing

The recorded signals from the building were first extracted and converted into an ASCII file format using the GRILLA® software [5], making them suitable for import into the modal analysis software ARTeMIS® [6]. The velocities illustrated in Figure 2 demonstrate the variations in amplitude during bell activation sequences compared to the ambient conditions. Channel 1 serves as the reference, while the remaining channels are positioned at the Oculus level. Analysis of all the velocity profiles reveals significant amplitude variations at both the Oculus and belfry levels, with some amplitudes reaching 2 to 3 times higher than those observed at other levels.

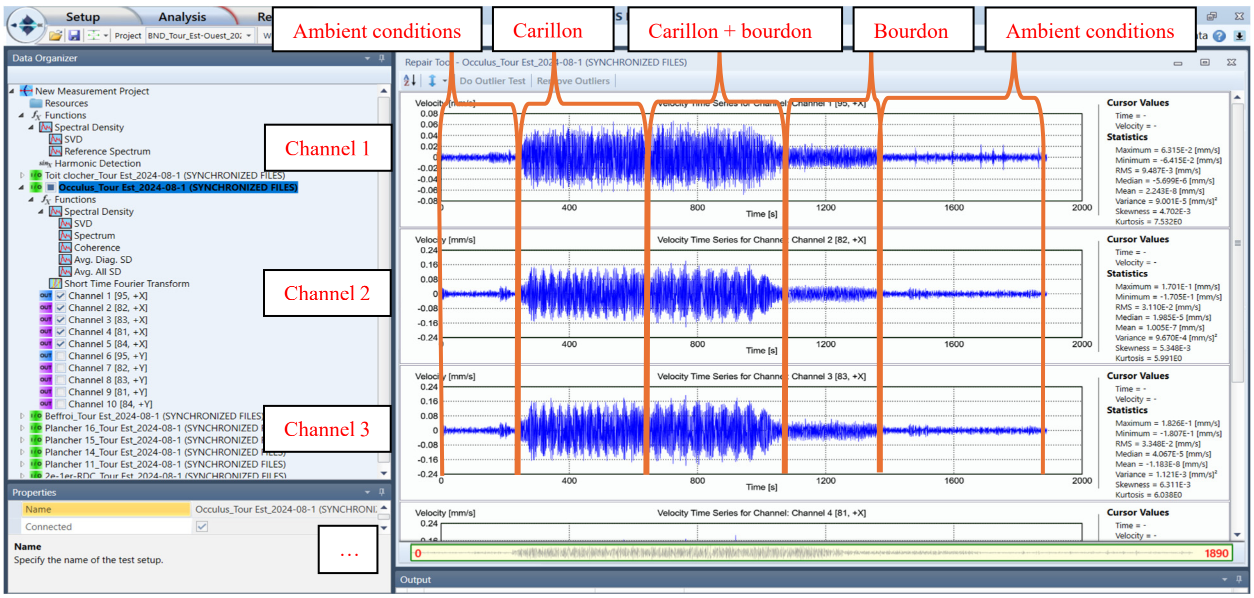

Figure 2: Resulting velocity time histories from ARTeMIS® software

The measurements were then input into ARTeMIS®'s mesh model representing the NDB, as shown in Figure 3. This model allows for the aggregation of all signals to be used in the modal analysis.

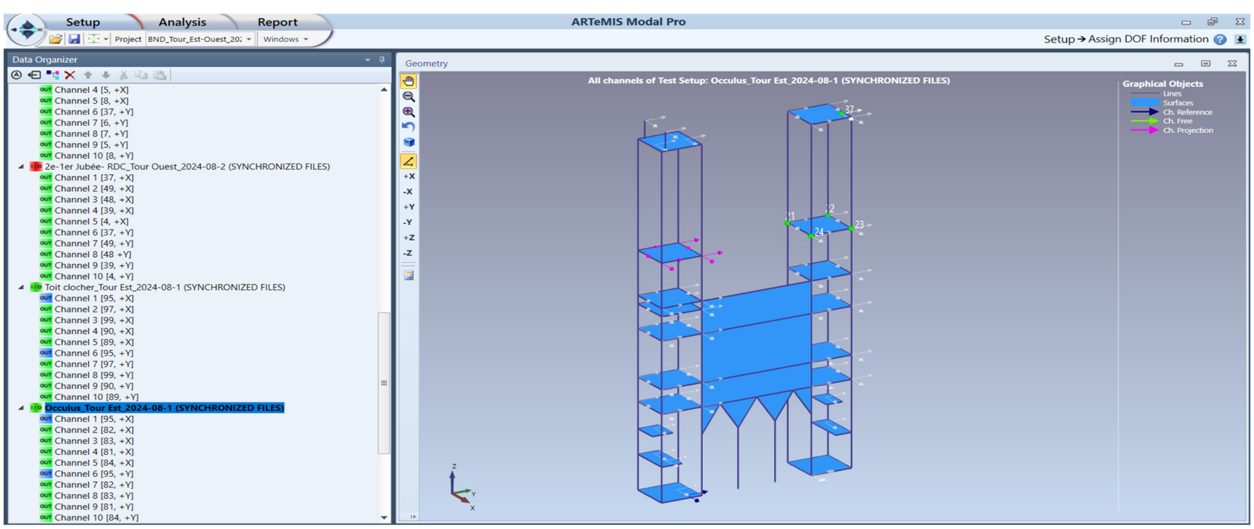

Figure 3: 3D ARTeMIS® model showing the locations of the measurement points and DOF

Modal estimation methods

Among the various modal estimation methods, this analysis primarily employs Enhanced Frequency Domain Decomposition (EFDD) and Stochastic Subspace Identification (SSI), selected for their computational efficiency and ability to cross-validate results [7]. While EFDD and SSI assume linear behavior and may not fully capture the nonlinear effects typical of URM bell towers, their combined use is robust and effective in reducing uncertainties and improving modal identification accuracy. The examples shown in Figures 4 and 5 illustrate the first fundamental modes of the towers, specifically focusing on the West tower. The peak amplitudes observed at frequencies of 1.56 Hz and 2.08 Hz indicate a dominant response. This suggests that these singular values of spectral densities (SVSD) represent the fundamental modes of vibration for the West tower, in the North-South and East-West directions.

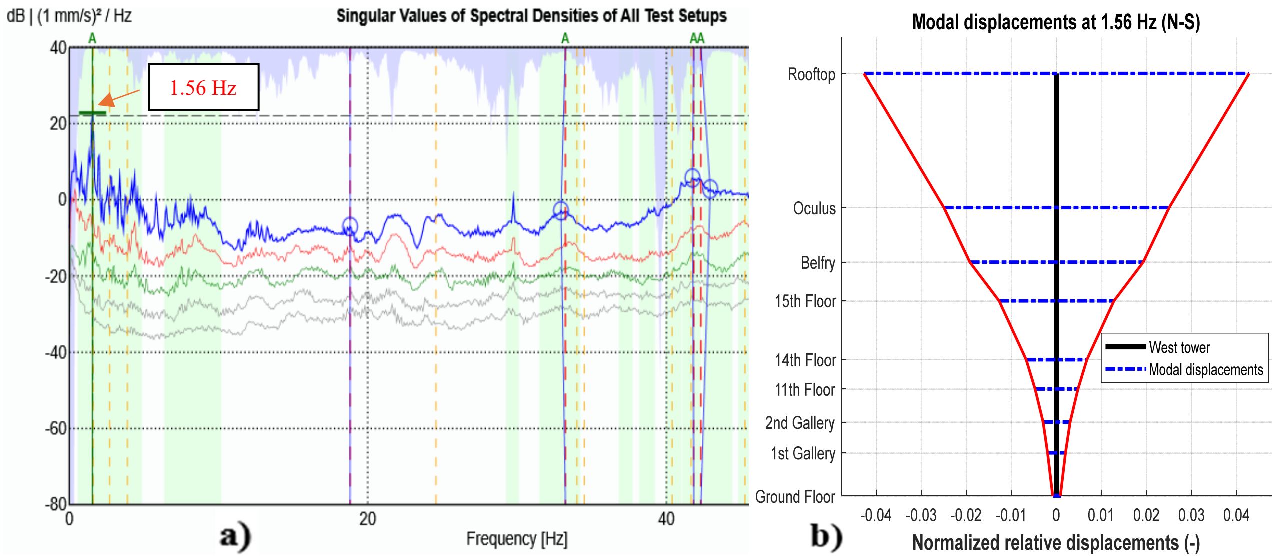

Figure 4: a) Singular Values of Spectral Densities (SVSD) from all test setups using EFDD and b) associated modal shapes in the North–South direction

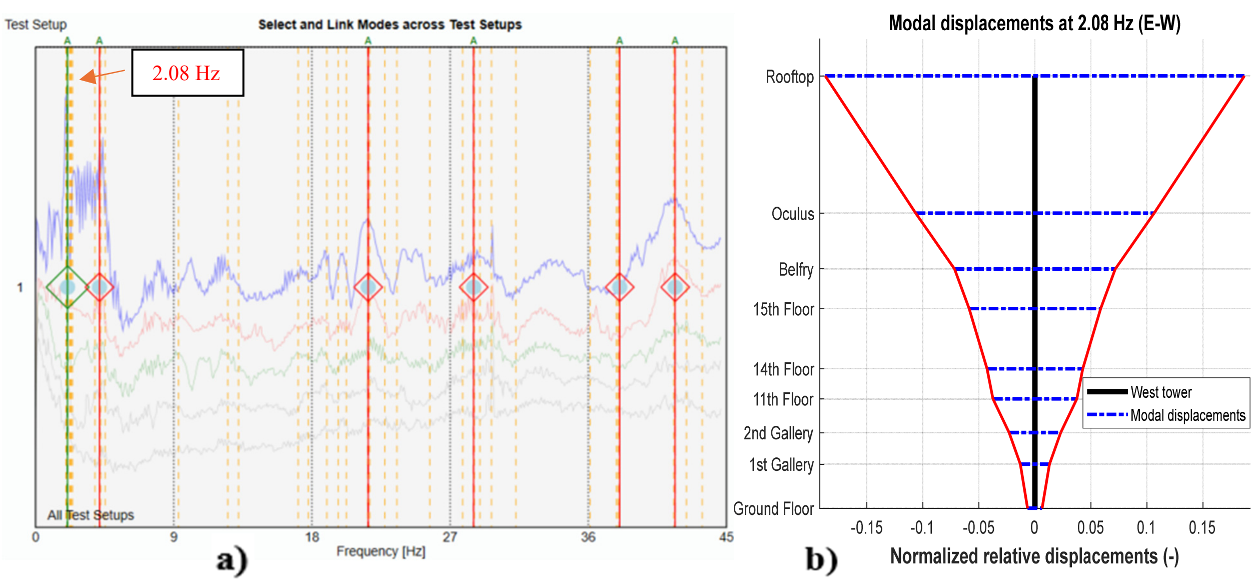

Figure 5: a) Singular Value of Spectral Densities (SVSD) from all test setups using SSI and b) associated modal shapes in the East-West direction

Cross-validation of the modal analysis results was demonstrated using the Modal Assurance Criterion (MAC) with a conservative rejection level of 0.85, as shown in Figures 6 and 7. This threshold was selected to ensure reliable modal correlation while limiting spurious modes, following the recommendations of Brincker and Ventura (2015), who consider values greater than or equal to 0.85 as indicative of strong modal consistency.

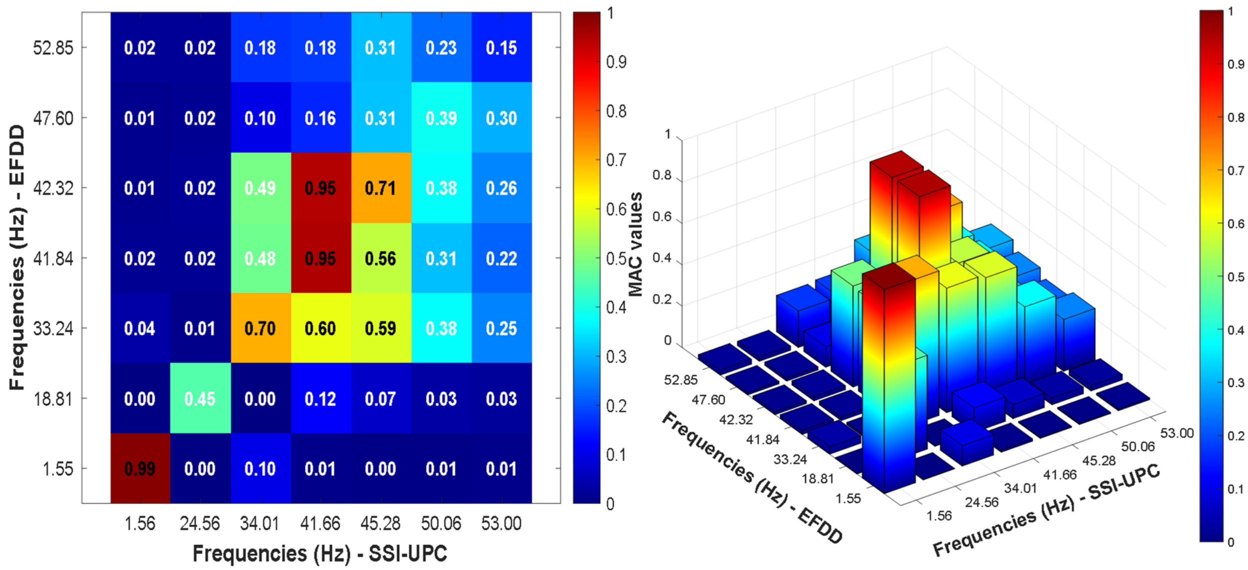

Figure 6: Modal Assurance Criterion (MAC) - West tower N-S direction

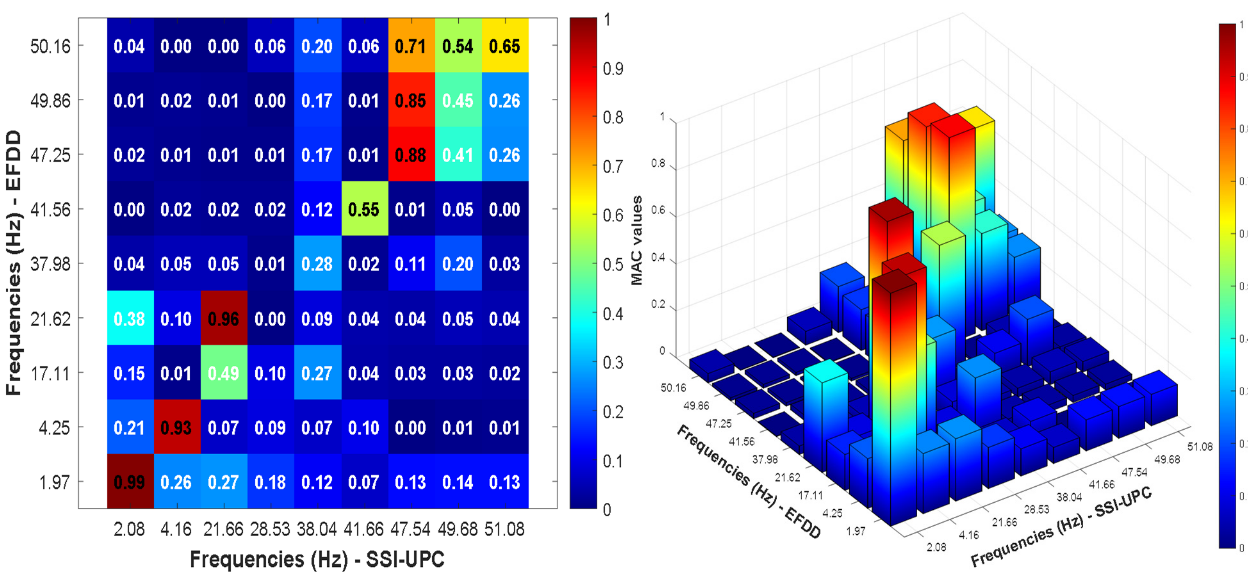

Figure 7: Modal Assurance Criterion (MAC) - West tower E-W direction

The first three fundamental modes of vibration and their corresponding dynamic parameters are presented in Table 1. The slight discrepancies between EFDD and SSI results (0.6–5.3% in frequency and 1.7–3.6% in damping) are expected and align with previous findings, such as those reported by Brincker and Ventura. The two methods process signals differently: EFDD, which is frequency-domain based, tends to smooth spectral variations, whereas SSI, which operates in the time domain, is more sensitive to damping effects.

This sensitivity can lead to either overestimation or underestimation of damping values depending on the signal characteristics and noise levels.

Table 1: Modal frequencies and damping ratios obtained from modal analysis estimated by EFDD and SSI methods

	EFDD Method		SSI-UPC Method		MAC
	f (Hz)	ζ (%)	f (Hz)	ζ (%)	West vs East
Mode shapes of the West tower					
1st Mode (N-S) - Translation	1.55	2.41	1.56	2.50	0.99
2nd Mode (E-W) Translation + torsion	1.97	1.77	2.08	1.80	0.98
3rd Mode - Torsion	4.25	3.18	4.16	3.12	0.99
Mode shapes of the East tower					
1st Mode (N-S) - Translation	1.50	2.79	1.49	2.80	0.99
2nd Mode (E-W) Translation + torsion	1.97	2.66	2.04	2.68	0.98
3rd Mode - Torsion	4.38	3.08	4.49	3.14	0.99

FINITE ELEMENT MODEL

Calibration and adjustment

A finite element model calibration was carried out to match the model closely to the experimental results obtained from the AVMs. The frame was modeled in the SAP2000® [8] software and consisted of various structural components (SC), including the masonry walls of the towers, the narthex façade, and the lateral walls. Additionally, the model incorporated masonry buttresses for the towers, as well as reinforced concrete (RC) beams and slabs for the floors of the towers. The concrete floor slabs, each with a thickness of 200 mm and a compressive strength of 27.6 MPa (4000 psi), were modeled as rigid diaphragms to ensure lateral load distribution and structural coupling. The masonry walls were modeled according to architectural specifications, featuring varying thicknesses across the structure. Both materials were assigned a unit mass of 2400 kg/m³ and were represented by 4-node shell elements with an optimal mesh size of 150 mm × 150 mm. Lateral walls were modeled to include their restraining effect on the towers. The hinged supports were used to reflect the flexibility of the base-foundation system and were validated by the close agreement between the calibrated model's modal frequencies and those identified experimentally.

The finite element model used shell elements that incorporate both in-plane (membrane) and out-of-plane (bending) stiffness. Mass was automatically included based on the material densities. The entire analysis assumed a linear-elastic response, without accounting for plasticity or material damage in order to remain consistent with the calibration of the FEM based on AVMs, which reflect structural behavior under low-amplitude and elastic conditions.

The seismic response was calculated based on cracked section properties, with the effective stiffness Ie assumed equal to 0.80 Ig for URM shear walls [9], and 0.20 Ig for slab element [10], where Ig denotes the gross stiffness. To effectively calibrate the model, the thickness of the masonry walls was accurately represented with dimensions that varied from 500 mm to 1880 mm. The initial elastic modulus of 5.0 GPa in the FEM, was increased to 11.5 GPa to better represent the heterogeneous composition of the unreinforced masonry walls, which included natural stone facing and a rubble core. Since the wall behaves as a composite system of stones and mortar, the chosen value reflects the overall structural response rather than the intrinsic properties of stone alone. Although the stone itself is inherently stiff and could exhibit a much higher modulus, petrographic analysis revealed internal weaknesses such as microcracks and micritization, which limited the potential increase in effective stiffness for the composite wall system. Similarly, the shear modulus was adjusted from 1.9 GPa to 4.425 GPa, based on Equation 1.

$$(1)\quad G = \frac{E}{2\,(1+\nu)} \quad \text{with } \nu = 0.30.$$

These values are consistent with the suggested ranges from the User's Guide – NBC 2015: Part 4 of Division B [9] and were validated through AVMs, which confirmed close agreement between measured and simulated modal frequencies. According to the literature on Operational Modal Analysis (OMA), specifically in the modal update section of Brincker and Ventura's (2015) work, a discrepancy of approximately 12% between experimental test results and the responses derived from finite element models (FEM) is generally considered acceptable for practical applications, especially for modal parameters such as natural frequencies. The mode shapes obtained from the calibrated FEM are presented in Figure 8.

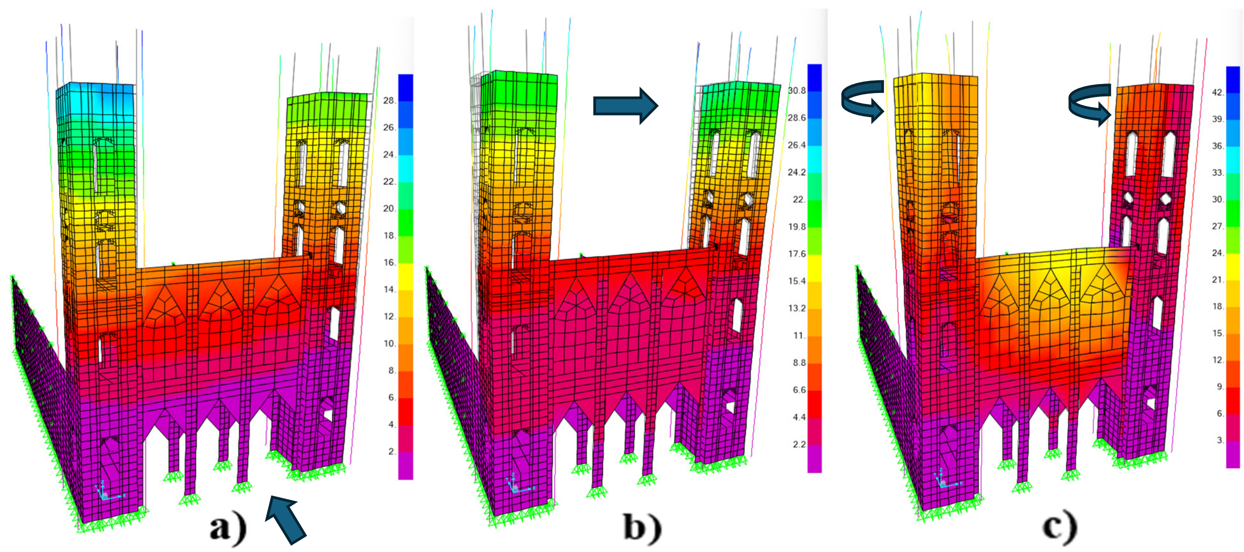

Figure 8: Mode shapes of NDB a) 1st Mode (N-S); b) 2nd Mode (E-W); c) 3rd Mode (X-Y)

Table 2 provides a comparative analysis of the frequencies derived from the initial SAP2000® model and the calibrated SAP2000® model, as well as the modal participating mass ratios, in conjunction with the results obtained from the modal analysis.

Table 2: Modal frequencies obtained from the SAP2000® numerical models and the experimental SSI method

Mode shapes of the Bell towers	SAP2000® model		Experimental (SSI-UPC)	Difference between SAP2000® model and SSI method		Modal mass participation ratio
	Initial	Calibrated				
	f (Hz)	f (Hz)	f (Hz)	Initial (%)	Calibrated (%)	
1st Mode (N-S) - Translation	1.02	1.56	1.56	52.9	0	32.6%
2nd Mode (E-W) - Translation + torsion	1.58	2.09	2.08	24.0	-0.5	36.2%
3rd Mode - Torsion	4.48	4.06	4.16	-7.7	2.4	2.6%

As shown in Table 2, the frequency discrepancies for the first three modes (0%, -0.5% and 2.4%) are well within the 12% threshold, validating the quality of the finite element model calibration. The experimental frequencies from the West tower were selected for FEM calibration, as this tower was completely restored and thus best represents the final structural condition. It also provides a reliable indicator of the global behavior of both towers. Importantly, the MAC values between the corresponding mode shapes of the East and West towers ranged from 0.98 to 0.99, indicating a very high degree of modal coupling. While the towers are geometrically similar, small differences in mass distribution, material conditions, or boundary constraints may introduce slight structural asymmetries, potentially influencing the modal response.

Earthquake solicitations and corresponding spectral analysis

To gain insight into the structural responses of NDB to earthquake forces, spectral analysis was conducted using the National Building Code 2020 (NBC) [10] uniform hazard spectrum for Montreal, corresponding to Site Class C, with a 2% probability of exceedance in 50 years. Spectral values were obtained from the Earthquakes Canada website, and the resulting design spectrum at the site is plotted in Figure 9. The site has been classified as site Class C, a designation based on the absence of specific geotechnical studies at the NDB site. Instead, data from the Geological Survey of Canada's geological survey map is utilized, highlighting the presence of fluvio-glacial deposits primarily composed of sand and gravel. This composition suggests that the soil in this region is robust and can be characterized as dense to very dense soil, designated as site class C in the NBC.

To assess the impact of the design seismic event, the seismic force (E) is applied in two distinct orientations: the North-South direction and the East-West direction. Damping ratios were taken directly from the AVM results, with values of 2.41%, 1.77%, and 3.18% for the first, second, and third modes, respectively, to reflect the actual energy dissipation observed in the structure. The load combinations considered in this analysis for comparability are defined as follows: 1.0 D + 1.0 E in the North-South direction, and similarly, 1.0 D + 1.0 E in the East-West direction.

Figure 9: The 2020 Uniform hazard spectrum for site class C in Montreal with a return period of 2% in 50 years

Table 3 presents the maximum values of displacements and stresses observed specifically in the bell towers. The calibrated finite element model (FEM) reveals generally higher stress levels across the structure. The increased rigidity of the structure results in increased stresses when subjected to external loads. The tensile and compressive stress results should be interpreted as preliminary indicators of the internal force distribution in the masonry walls. Although these values are derived from a linear elastic model and do not account for cracking or nonlinear behavior, they serve to identify critical areas and regions of potential fragility. Thus, they are used as qualitative indicators to better understand the stress distribution rather than precisely predicting failure. Importantly, the observed stress patterns predominantly follow the vertical and longitudinal axes of the tower walls, which confirms that these stresses are primarily in-plane and reflect the expected load paths in unreinforced masonry systems.

As the damping ratios for all fundamental modes remain below 5%, the increase in spectral accelerations diminishes the significance of the minimal displacement variation between the initial and calibrated models. This indicates that, despite increased seismic demand, displacement responses remain of limited impact. Therefore, the stress distribution offers a more insightful representation of the structural demand.

Table 3: The displacements and stresses obtained by the FEM before and after calibration

Bell towers	SAP2000® model			
	Initial		Calibrated	
	Displacement max (mm)	Stress max (MPa)	Displacement max (mm)	Stress max (MPa)
Earthquake (N-S)	100	-1.0 / 3.5	106	-2.1 / 8.0
Earthquake (E-W)	61	-1.1 / 2.4	68	-1.6 / 8.0

To facilitate a comprehensive graphical interpretation of the stress and displacement distributions within the structure, some of the results derived from the calibrated finite element model are presented in Figure 10. This figure illustrates the results for the earthquake scenario in the North–South (N–S) direction, while those for the East–West (E–W) direction are provided in Table 3.

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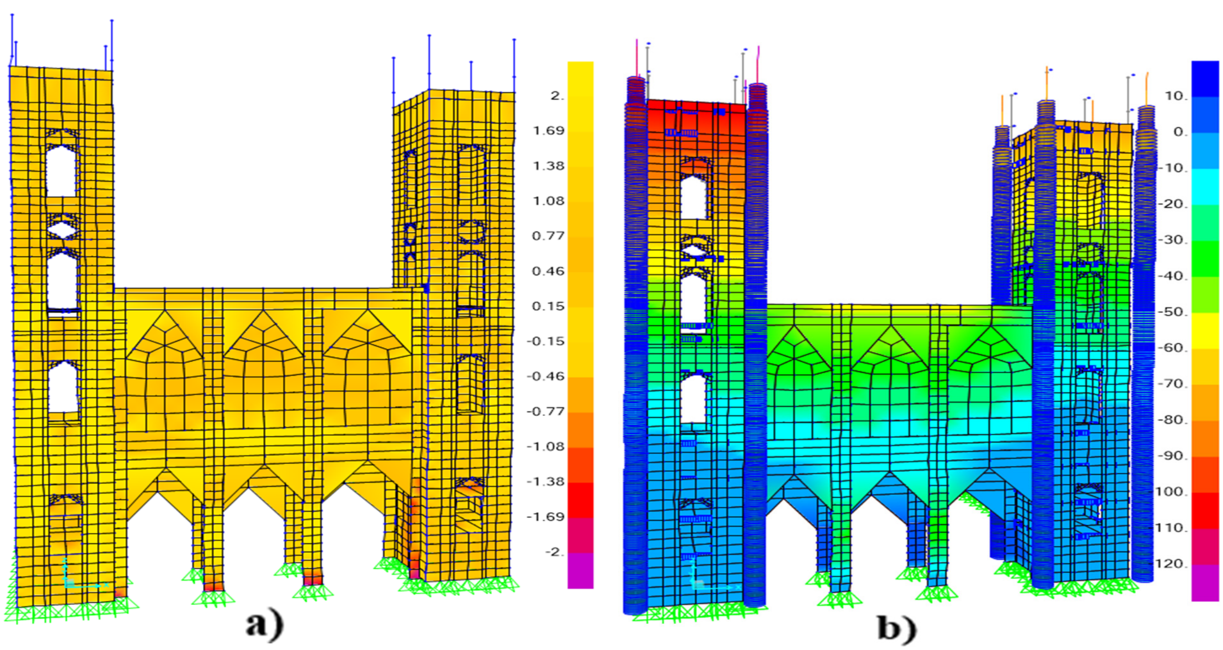

Figure 10: Response spectrum results in the N-S direction: a) Axial stress; b) Displacement

The greatest displacements are observed at the apex of the two towers. Concurrently, the zones with the highest concentrations of axial stresses are located near the openings and at the junctions where the façade of the narthex connects with the two towers, identifying critical areas that should be carefully considered in any conservation or strengthening strategy. The calibration of the FEM reveals that axial stresses can increase considerably, potentially reaching up to twice the values observed in compression and even higher in tension, particularly within these vulnerable zones. In contrast, the corresponding displacements exhibit minimal variation.

Bell frequencies

The bells were modeled using brass material with a mesh size of 0.2 mm x 0.2 mm, a density of 8700 kg/m³, an elasticity modulus of 103 GPa, and a minimum yield stress of 345 MPa. The natural frequency of 84 Hz obtained from the calibrated numerical model for the bourdon aligns with the theoretical frequencies of the range of Fa^2 (87 Hz) and its harmonics (infinite multiples of the fundamental frequency). The examination of the fundamental modes of the bells provided a clear delineation of the building's dynamic parameters, thereby resolving any ambiguities associated with the vibrations generated by the bells during operation. The results of the modal analysis indicate that there is no resonance between the bells and the fundamental vibration modes of the church structure.

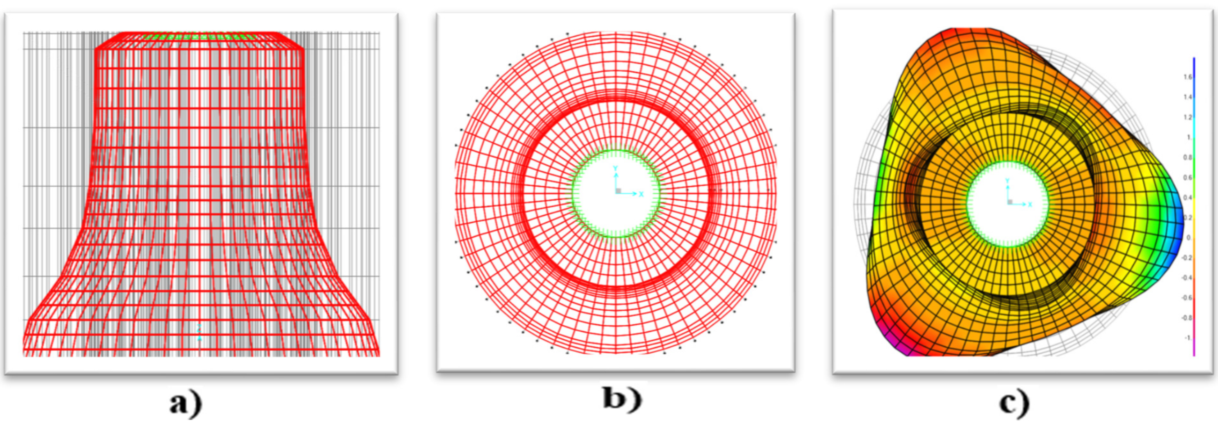

Figure 11: Bourdon: a) Elevation view; b) Plan view and c) natural frequency at 84 Hz

CONCLUSIONS AND FUTURE WORK

The study investigates the dynamic response of the bell towers of the Notre-Dame Basilica in Montreal through ambient vibration measurements (AVM) conducted under various conditions and earthquake response spectrum analysis. The AVM analysis successfully differentiated bell vibration noise from modal deformations by comparing measurements taken under normal ambient conditions with those recorded during bell ringing events. This approach led to the identification of the dynamic responses of the bell towers by effectively decoupling the vibrations generated by the bells from the modal deformations associated with the fundamental periods of the structures. This differentiation was achieved through the finite element model of the bells, combined with operational modal analysis techniques (EFDD) and Stochastic Subspace Identification (SSI). The analyses revealed a high Modal Assurance Criterion (MAC) correlation, with values reaching 99% for the first two modes and 93% for the third mode. The calibration of the FEM with the experimental results obtained from AVMs showed discrepancies of 0%, -0.5%, and 2.4% for the first three vibrational modes, all within the acceptable range established in literature.

A numerical finite element model was constructed and then calibrated using results from AVMs and operational modal analysis (OMA). The seismic analysis of the calibrated model revealed significantly higher global axial stress levels, particularly in critical areas such as structural connections and openings, compared to the initial model. Compression stress values increased by 145% to 210%, while tension stress values increased by 230% to 335%. As previously established, damping ratios below the 5% reference threshold induce an increase in spectral accelerations. However, despite this elevated seismic demand, the displacement response at the apex of the towers exhibited marginal variation in both orthogonal directions relative to the initial model. This phenomenon is attributable to the towers' enhanced rigidity, which mitigates the impact of the increased spectral accelerations on displacement. These findings underscore the substantial differences introduced by the FEM calibration, highlighting the importance of this methodology in assessing the dynamic and seismic behavior of URM bell towers.

Future work will focus on refining the assessment of stresses and shear forces induced in the masonry by both earthquakes and bell usage. This will include simulating the time-history function of bell ringing and evaluating its impact on the structural integrity of the towers. The results of these analyses will be compared with the resistance criteria outlined in the CSA S304 standard. By enhancing the understanding of the dynamic demands under seismic activities and bell usage, the study aims to inform the development of targeted mitigation and rehabilitation strategies based on more accurate and detailed analyses.

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