

EXPERIMENTAL INVESTIGATIONS ON THE BELL TOWER OF PIEVE SAN LORENZO

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Abstract. *This study presents the diagnostic investigations conducted on the bell tower of Pieve of San Lorenzo in Minucciano, in the Lunigiana region of Tuscany, Italy. The bell tower is an integral part of a complex of great historical and architectural significance. The Lunigiana is characterized by high seismic activity, and several significant earthquakes have affected the bell tower over the years. The structure suffered severe damage in 1920, leading to restoration work after the collapse of its top level. More recently, the 2013 earthquake (Mw 5.2) caused new cracks, particularly in the pillars of the bell chamber, highlighting the need for structural assessment. For these reasons, the tower needed diagnostic investigations to assess its current condition and to plan possible interventions.*

The investigations were conducted both in situ and at the Structural and Material Testing Laboratory of the Department of Civil and Environmental Engineering of the University of Florence. Among dynamic tests, environmental vibration and sonic tests were carried out. Ambient vibration tests have been performed with accelerometers placed at various levels of the tower along the inner masonry wall. The vibrations of the structure caused by both environmental factors and the bells motion have been recorded. The signals obtained have been then processed and analysed, to determine the tower's dynamic parameters. Sonic tests have instead been performed on the pillars adjacent to the mullioned windows located near the lower level of the tower's octagonal section.

Beyond assessing the health status of the tower under study, this paper aims to compare the results obtained from the various tests, evaluating their reliability and accuracy, as well as how they complement and integrate with each other for a comprehensive assessment of historical structures.

Keywords: Masonry, environmental vibration, sonic tests, bell tower.

1 INTRODUCTION

The bell towers of historic churches, symbols of faith and architectural testimonies of past eras, are among the most vulnerable elements of cultural heritage. Their height, exposure to atmospheric agents, and the complexity of masonry behaviour make them particularly susceptible to degradation and seismic events, highlighting the need to assess their static and dynamic conditions to ensure their preservation.

In this context, the study focuses on the bell tower of the Pieve of San Lorenzo, located in the municipality of Minucciano, and belonging to the Diocese of Lucca. This structure, an integral component of a complex of parish churches of great historical and architectural significance, consists of a lower quadrangular tower dating back to the 15th century and an upper octagonal section built between the 16th and 17th centuries. Over the centuries, the bell tower has undergone numerous modifications, including additions, restorations, and renovations that have altered its original features.

From a seismic perspective, the area where the bell tower stands is classified as a second-category seismic zone, indicating high hazard levels. The region has experienced significant seismic events throughout history, including the 1481 Fivizzano earthquake (ASMI, Italian Historical Macroseismic Archive) and the 1878 earthquake, as well as the earthquakes of 1545 and 1834 in Pontremoli, those of 1641 and 1837 (Mw 7.1) in Lunigiana, and the most significant event on September 7, 1920 (Mw 6.3). Additionally, lower-intensity tremors were recorded in 1939, 1951, 1995, and during the seismic swarm of 2013.

To analyse the conservation state of the bell tower without compromising its structural integrity, non-destructive testing (NDT) techniques have proved to be fundamental tools. Among these, sonic tests and ambient vibration tests with accelerometers allow for the characterization of the mechanical and dynamic properties of bell towers.

In this paper, the application of these methods to the structure under study is described and analysed, aiming to evaluate their effectiveness in the field of cultural heritage diagnostics.

2 IMPACT OF SEISMIC ACTIVITY ON THE STRUCTURE

The bell tower is located adjacent to the apse on the left side and structurally shares a part of the masonry walls. Its architectural configuration consists of a square base, tapered upwards on one side, above which an octagonal-plan tower rises (Figure 1).

The lower quadrangular tower stands 9.30 meters high, with a wall thickness that varies from bottom to top, decreasing from 1.50 to 1.00 metres. Inside, it features two barrel vaults arranged along two orthogonal directions. The upper octagonal section, built with roughly worked stone blocks and irregular masonry bound with lime mortar, reaches a height of approximately 13.12 m and has a uniform wall thickness of 1.00 m. Inside, the top level houses two domes, while the belfry is characterized by a wooden truss structure, which is now non-functional, and a reinforced concrete roof. Additionally, the belfry contains four bells of different sizes, supported by steel beams embedded within the masonry.



Figure 1: The bell tower of Pieve San Lorenzo, Minucciano: view from the apse and from the naves.

The 1920 earthquake caused severe damage to the church and led to the collapse of the uppermost section of the bell tower. The existence of an additional level with respect to the current configuration, is also documented by the 7th–18th century painting “Martirio di Santo Stefano”, preserved inside the church, where the bell tower appears fully plastered and with an extra floor.

Renovation and restoration works were carried out only in 1927. These works involved the reconstruction of the tower's top level using stone masonry and cement mortar, as well as the addition of a reinforced concrete ring beam at the springing of the belfry's arched openings. In 1983, further restoration works were undertaken on the bell tower's roof. These interventions included the construction of a reinforced concrete ring beam, the external cladding of the roof with lead sheets, and the application of the mortar on the outer surface of the wall at the tower's top level. However, the 2013 earthquake (Mw 5.2) caused additional damage to the structure, leading to severe cracks in the octagonal body and damage to the belfries pillars, with the consequent material loss in some areas.

In the following paragraphs, the dynamic tests will be presented in detail. It's worth mentioning that these analyses were conducted following several core drillings (Figure 2) and endoscopic investigations, performed at different levels of the bell tower to verify the composition and quality of masonry. Overall, they provide information on the type of masonry, highlighting a certain masonry compactness, with no discontinuities within the thickness, thus allowing the exclusion of a core-filled masonry. From the cores, samples of both stone and mortar were extracted, tested in the laboratory, and used to obtain some compressive strength data.

A single and double flat jack test was carried out at the second level of the tower in accordance with [1][2]. During the test, a cavity was found behind the test area, extending from about 30 cm to 100 cm deep inside the wall. Therefore, the test was considered invalid.



Figure 2: Drilled core samples.

3 DYNAMIC IDENTIFICATION OF THE STRUCTURE: AMBIENT VIBRATION TESTING

To ensure an accurate representation of the overall dynamic structural behaviour, sixteen accelerometers were deployed across various levels of the bell tower (Figure 3 and 4). Their placement was defined through preliminary numerical analyses conducted on a finite element model to identify the expected frequencies and modal shapes. The instruments used were uniaxial PCB Piezotronics high-sensitivity seismic accelerometers (model 393B12, 393B31 and 393C), fixed to the structure by means of steel supports, ensuring perfect alignment between the instrument base and the metal support surface. Specifically, single-plate supports with a single accelerometer were used to record perpendicular accelerations ("rad"), while L-profile welded supports allowed for the combination of two accelerometers for tangential ("tan") and radial accelerations ("rad").



Figure 3: Measuring station and some positions of accelerometers.

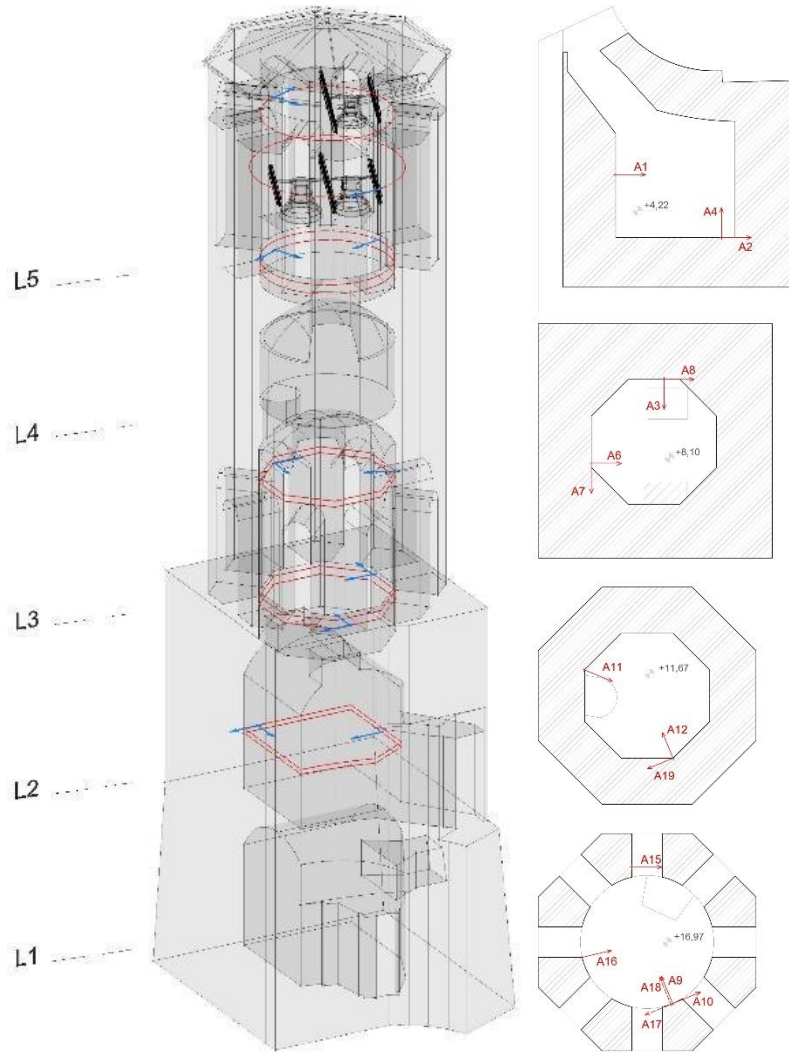


Figure 4: Three-dimensional representation of the bell tower and cross sections at level L2, L3, L5, with the accelerometers' location.

During the experimental campaign, ten continuous recordings were performed, capturing the dynamic responses of the bell tower to environmental excitation. In some tests, an impulsive action was also applied by synchronously or selectively ringing the three bells in the belfry. The length of the time series recorded was decided so as to comply with the empirical suggestions given in the literature, hypothesizing a first frequency at 2 Hz and a damping equal to 2% [3]. The signals were simultaneously recorded at a sampling frequency equal to 4800 Hz, for all the recordings.

Since the investigation is based on an output-only approach, it is important to highlight that the recorded vibrations were of low amplitude due to the minimal vehicular traffic in the area and with no significant wind events. As a result, the data analysis required careful signal processing, including the removal of instrumental and environmental noise and subsequent resampling. The obtained signals were then analysed in the frequency domain.

For the sake of example, Figure 5 shows the signal recorded during the Minu_3 test by the accelerometer A16, together with the corrected and resampled data, with a frequency of 120 Hz (Resample 120 Hz, Filt 0.5-50 Hz), plotted with red line.

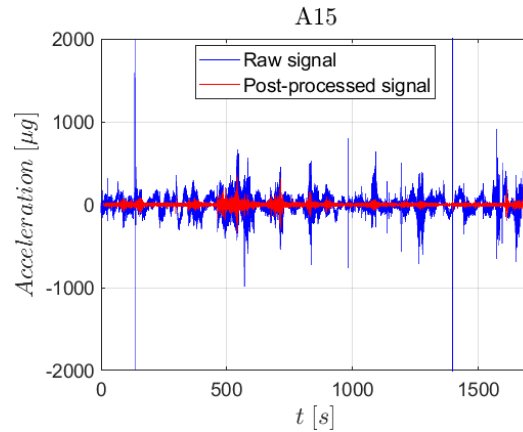


Figure 5: Example of a signal recorded during the environmental test “Minu_03”. The raw signal (blue) and the post-processed signal (red).

3.1 Data analysis and dynamic identification of the numerical model

The Frequency Domain Decomposition (FDD) method [4] was used for the data analysis. Specifically, the average spectral tensor was evaluated for signals from Minu-03 and Minu-10 recordings. Following the modified periodogram method, time windows composed of 2^{12} and 2^{13} data points have been set, overlapping by approximately 30% or 70%. Singular values of the PSD matrix have been evaluated for cases: Minu-03, 2^{13} , 70% (case “a”), Minu-03, 2^{12} , 70% (case “b” shown by Figure 6), and for Minu-10, 2^{12} , 70% (case “c”), Minu-10, 2^{13} , 30% (case “d”).

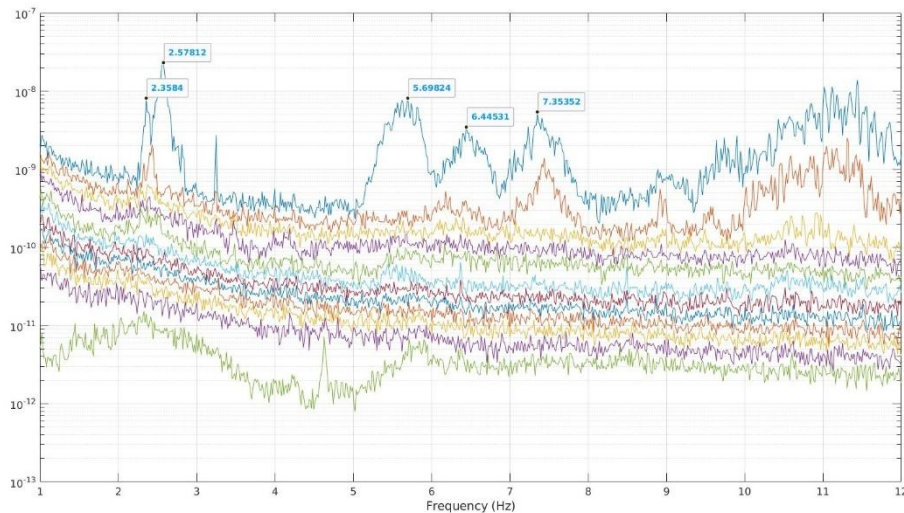


Figure 6: Singular values of the PSD matrix from Minu-03 signals (2^{13} , 70%).

In order to correlate the different sets of vibrational modes obtained, the Modal Assurance Criterion (MAC) was applied [5]. In particular, Figure 7 shows the MAC indices of the modes found by changing the time windows (on the left and the middle) and between the modes found from the signals Minu-03 and Minu-10 (on the right): the results confirm the correspondence between the modes found.

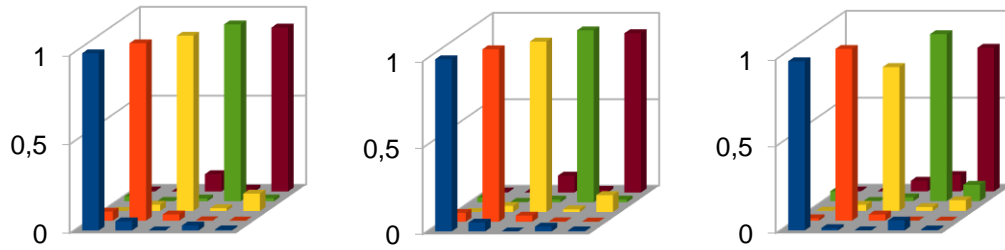


Figure 7: MAC values computed between the vectors from “case a” and “case b”, “case c” and “case d”, “case a” and “case d”.

The frequencies obtained seem to be aligned with the results in the literature concerning historic masonry bell towers [6][7]. Typically, the first fundamental frequency falls within the 0.5 Hz to 3 Hz range, while the second and third frequencies can vary between 1.5 Hz and 6 Hz [8]. Higher frequencies are associated with local vibrations of specific structural elements.

| | Case a | Case b | Case c | Case d | Average |
|----|--------|--------|--------|--------|---------|
| f1 | 2.358 | 2.374 | 2.373 | 2.358 | 2.366 |
| f2 | 2.578 | 2.578 | 2.549 | 2.549 | 2.563 |
| f3 | 5.698 | 5.683 | 5.742 | 5.742 | 5.716 |
| f4 | 6.445 | 6.445 | 6.475 | 6.475 | 6.460 |
| f5 | 6.680 | 6.592 | 7.412 | 7.397 | 7.020 |

Table 1: Main frequencies (Hz) of the bell tower evaluated experimentally.

The collected data enabled the calibration of the three-dimensional finite element model of the tower, which was used to perform some nonlinear dynamic simulations [9]. The numerical model was created from the 3D geometric model in Figure 4 using Mady, a FEM code specifically developed for the analysis of masonry structures [10][11]. The model, made of 4-node isoparametric brick elements, makes use of a constitutive equation developed assuming a constraint on the stress that must belong to a stress range, defined accounting for a limited material’s tensile, compressive and/or shear strength [12][13].

The numerical simulations allowed the identification of the most significant uncertain parameters of the model; then, boundary conditions imposed by the church body on the tower and the material’s Young’s modulus E were calibrated to minimize the discrepancy between the modal parameters of the model and those determined experimentally. The density d and Poisson’s ratio ν were instead kept constant during the calibration.

| Mechanical parameters | E (GPa) | d (kg/m ³) | ν |
|-----------------------|-----------|--------------------------|-------|
| Quadrangle tower | 2.40 | 2000 | 0.15 |
| Octagonal part | 1.25 | 1900 | 0.15 |
| Roof | 30 | 2550 | 0.2 |

Table 2: Mechanical parameters of the calibrated model.

4 DETAILED STUDY ON SOME STRUCTURE PART: SONIC TESTS ON PILLARS

Sonic tests were conducted on two consecutive pillars, which are separated by one of the monofora openings and located at the base of the octagonal part of the tower (Figure 8). Over each pillar, the mechanical impulse was generated at four emission points (A, B, C, D), and the transmitted sonic waves received and recorded at six reception points, located on the opposite (A, B, C, D) or adjacent face (E, F). In particular, to generate the stress wave an impulse force hammer (PCB Piezotronics Model 086D20) was used, while uniaxial accelerometers were used as receivers. The signals were recorded at a sampling frequency of 38400 Hz. The automatic identification of wave travel time is not always reliable due to signal disturbances, such as interference or background noise. For this reason, the recorded data have been manually verified.

Figure 9 shows the distances between the impact hammer and the accelerometers.



Figure 8: Instruments location over the pillars.

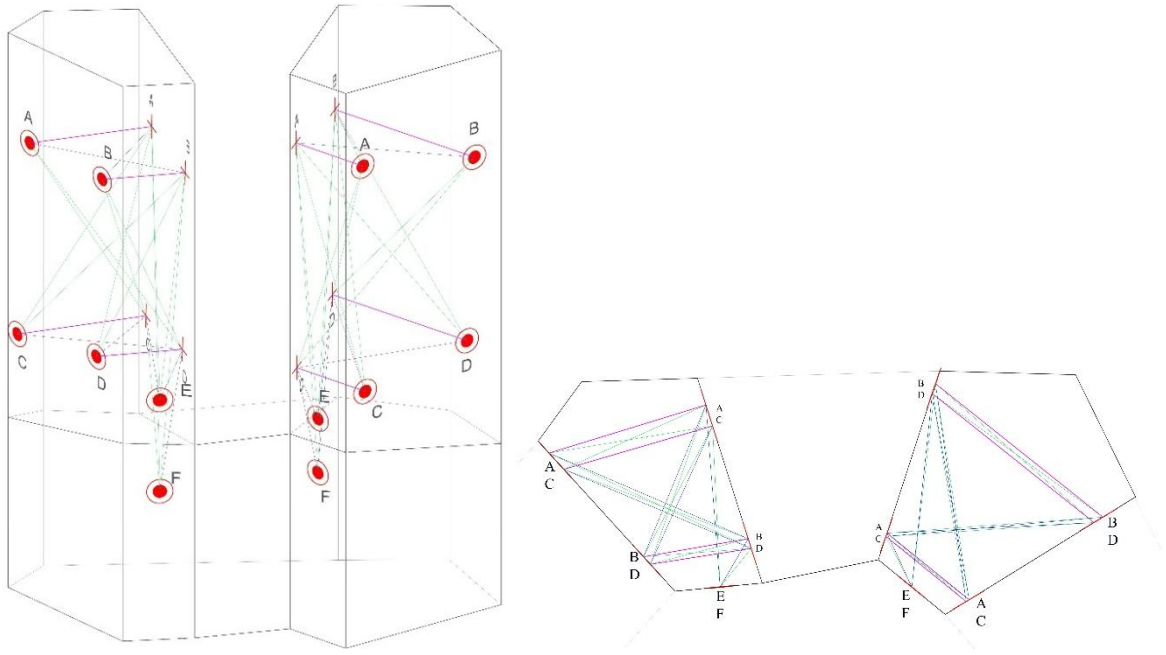
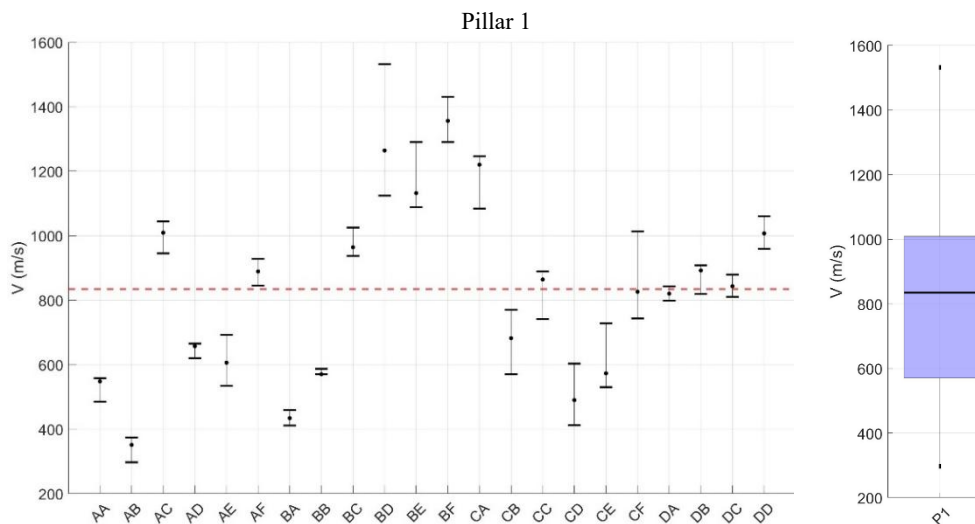


Figure 9: Distance between the impact hammer and the accelerometers: direct (magenta), indirect short (blue) and indirect long (green) measures.

Figure 10 shows the lower, upper, and median values for the speed recorded in the five impacts, and the lower and upper extremes determined using the interquartile range (IQR) method. Pillar 1 shows greater variability in velocity values, with a wider distribution and lower velocities. The boxplot confirms the greater homogeneity of Pillar 2 compared to Pillar 1, with a more contained interquartile range (IQR) and a more compact distribution around the median. DF for Pillar 1, and CC and CE for Pillar 2 (see Figure 9) exhibited velocity values ranging from 1232 m/s to 5013 m/s, and significantly higher with respect to the average. The reason could be related to the fact that the measurements were taken along the outer surface of the wall, where the stones appear to be more regularly shaped. Hence, these values have not been considered in the analysis.

Additionally, Pillar 1 has an average velocity of 818 m/s, with a standard deviation of 276 m/s, while Pillar 2 has an average velocity of 1192 m/s, with a standard deviation of 176 m/s.



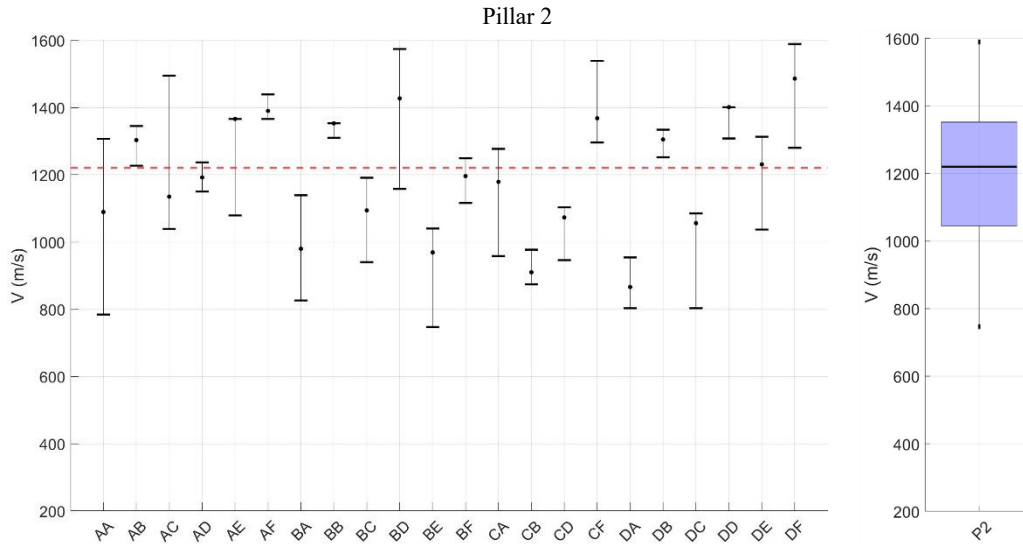


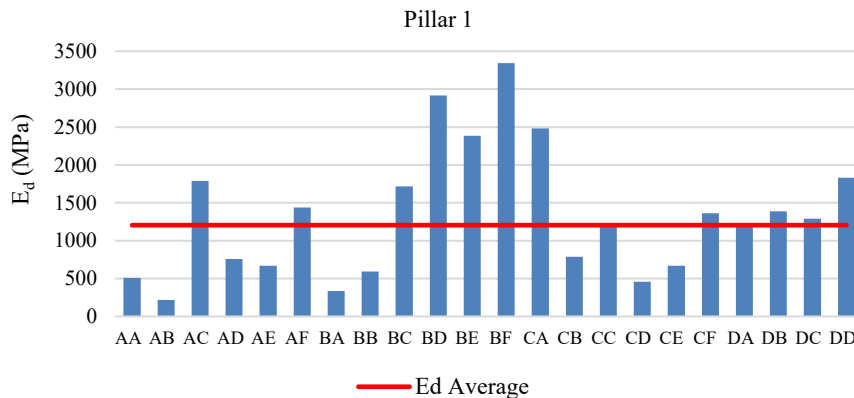
Figure 10: Recorded speeds and IQR analysis of Pillar 1 and Pillar 2.

Furthermore, the results confirm the absence of significant discontinuities within the pillars or significant internal voids, in accordance with the core drilling tests results.

Based on the obtained results, the dynamic Young’s modulus can be estimated using Equation (1) [14]. However, this calculation is affected by uncertainties related to the assumed material parameters, particularly Poisson’s ratio and density, which are necessary for determining the modulus.

$$E_d = \rho \cdot V_p^2 \cdot \frac{(1 + \nu)(1 - \nu)}{(1 - \nu)} \tag{1}$$

In the absence of specific information on the analysed masonry, the following reference values were adopted: Poisson’s ratio $\nu = 0.15$ and masonry density $\rho = 1900 \text{ kg/m}^3$. Excluding anomalous values, the estimated dynamic Young’s modulus from Pillar 1 is 1205 MPa and 2559 MPa from Pillar 2 (Figure 11).



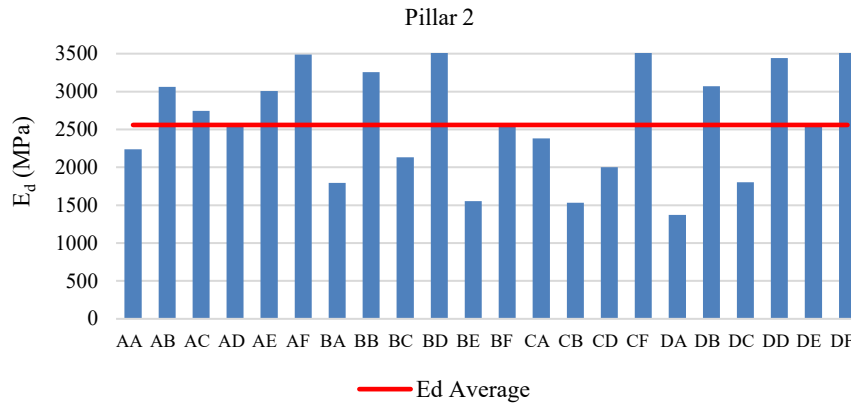


Figure 11: Young's Modulus of Pillar 1 and Pillar 2.

Although the E_d provides useful insights into the material properties, it cannot be directly used to assess the structural response of the bell tower but need to be converted into a static Young's modulus (E_s) [15], which better represents the material's behaviour under gravitational loads and static actions, such as self-weight. This conversion is essential because materials may respond differently to dynamic and static stresses. The ratio k between the dynamic (E_d) and static (E_s) Young's modulus depends on several factors [16], including:

- (i) masonry type: stone, brick, or mixed masonry may exhibit different E_d/E_s ratios;
- (ii) state of preservation: the presence of cracks, deterioration, or restoration interventions affects mechanical properties;
- (iii) material characteristics: density and porosity can alter mechanical behaviour.

There is no single universal conversion coefficient, but scientific literature provides reference ranges based on experimental studies on stone and brick masonry. The ratio k generally varies between 1.1 and 2.5 [15].

Using a conversion coefficient of $k = 1.1$, the corresponding static Young's modulus values turn out to be 1085 MPa for Pillar 1 and 2305 MPa for Pillar 2.

A further analysis was carried out with the data obtained from the sonic tests. Specifically, the signals recorded were analysed in the frequency domain in order to have an insight on the natural frequencies of the pillars. First of all, the signals were filtered in a frequency band between 2 Hz and 200 Hz. Then, the power spectral densities of the signals were evaluated for all the sensors. Moreover, the signals obtained from the sensor in the position A, B, C and D were combined among themselves with an approach like the ANPSD (e.g. [17]) to have a clearer interpretation of the results. For Pillar 1, the first three frequencies evaluated were $f_1 = 47.24$ Hz, $f_2 = 54.84$ Hz and $f_3 = 75.36$ Hz, while for Pillar 2 the results were $f_1 = 60.52$ Hz, $f_2 = 66.61$ Hz and $f_3 = 92.05$ Hz.

For each pillar, the obtained results have been compared with the numerical frequencies of finite element models, using for the mechanical parameters values of the sonic tests, summarized in Table 3. The geometric modelling was carried out using AutoCAD, while the mesh was generated with Gmsh [18], and the model analysis conducted with Mady

| Mechanical parameters | E_d (GPa) | d (kg/m³) | ν |
|------------------------------|-------------------------------|--|-------------------------|
| Pillar 1 | 1.205 | 1900 | 0.15 |
| Pillar 2 | 2.559 | 1900 | 0.15 |

Table 3: Mechanical parameters of the FE model of pillars.

The results of the modal analysis, which provides values of the frequencies $f_1 = 53.76$ Hz, $f_2 = 62.63$ Hz and $f_3 = 86.57$ Hz for Pillar 1 and $f_1 = 80.99$ Hz, $f_2 = 86.93$ Hz and $f_3 = 124.18$ Hz for Pillar 2, evidences an acceptable agreement with those obtained by the frequency domain analysis of the signals, thus confirming the reliability of sonic results.

Lastly, the results of the sonic tests were compared with those obtained from ambient vibration tests and those obtained from compression tests performed on mortar and stone specimens (Table 4). With this aim, the static elastic modulus E_s was derived from the dynamic elastic modulus E_d obtained through ambient vibration tests, analogously to what described in Section 4. Moreover, to evaluate the static elastic modulus of masonry on the base of compression tests both the Italian Building Code [19] and Eurocode 6 [20] were used. In both code, the procedure involves determining the characteristic compressive strength of the masonry, which is then multiplied by a coefficient k , whose value depends on the type of masonry. According to these criteria, the resulting elastic modulus ranges between 2 and 2.4 GPa.

| Test | E_d (GPa) | E_s (GPa) |
|---------------------------|-------------------------------|-------------------------------|
| Ambient vibration testing | 2.400 – 1.250 | 2.162 – 1.126 |
| Compression testing | | 2 – 2.400 |
| Sonic tests | 1.205 – 2.559 | 1.085 -2.305 |

Table 4: Mechanical parameters obtained from different tests.

The numerical simulations conducted on the two pillars confirmed the consistency of the global model representing the tower. Besides, the comparison between the static elastic modulus obtained from the compression tests and the values adopted in the numerical simulations shows a good concordance, reinforcing the reliability of the estimated mechanical properties of masonry. On the other hand, according to the literature [21], the results show that the interpretation of the sonic tests should be used in conjunction with in-situ and/or destructive tests to obtain quantitative, reliable results. In addition, considering the inherent variability of historical masonry materials, it remains essential to integrate different types of tests for evaluating the mechanical properties and ensuring a reliable assessment of the structural behaviour.

5 CONCLUSIONS

The experimental investigations conducted on the bell tower of Pieve San Lorenzo include non-destructive investigations, namely sonic tests and environmental tests, as well as semi-destructive investigations, with flat-jacks and core sampling.

Sonic tests are highly sensitive to the local characteristics of the masonry, and their results cannot be considered fully representative of the entire structure. In addition, sonic investigations are not yet well-standardized, and their effectiveness in providing reliable

indications on the properties of materials, especially in the case of disorganized stone walls, as often in the case of monuments, has yet to be demonstrated. However, compared to flat-jack tests and core drilling, they have the advantage of being less invasive and faster to perform, avoiding damage to the historical monument, and for these reasons, they are increasingly used in heritage conservation.

In this work, comparisons are made on the estimation of the Young's modulus form obtained from the various tests. The comparison of these different methodologies revealed a good correlation in the results, which in principle validates the results obtained from sonic measurements. Nevertheless, it appears that the interpretation of the sonic tests benefits from being supported by other in-situ and/or destructive tests, to obtain quantitative, reliable results.

For what concerns the health status of the bell tower, the testing campaign carried out has shown that despite some variations in masonry quality, the tower doesn't present significant structural discontinuities and maintains overall compactness.

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