

## Article

# Field Investigation and Numerical Modeling for the Seismic Assessment of the Castle of Lanjarón, Spain

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**Abstract:** The Castle of Lanjarón is a 16th century stronghold located in Andalucía, Spain. After losing its military function, the castle was abandoned, leading to significant decay. Designated a national heritage site in 1985, recent efforts have sought to preserve its historical and cultural value. This study outlines an inspection and diagnosis campaign carried out on the castle. Non-destructive tests (NDTs) were employed to characterize the properties of the masonry, using both mechanical and wave-based methods. Dynamic identification was performed to determine dynamic and modal properties of the structure, which were used to develop and calibrate a three-dimensional (3D) finite element model (FEM) of the west wall, based on homogenized masonry material. Limit analysis and non-linear static (pushover) analysis under various boundary conditions were conducted to determine the maximum relative load factor in the out-of-plane direction. The results were compared to the expected peak ground acceleration (PGA) of the area, showing that the maximum load capacity of the wall exceeds local seismic demands with a safety factor of 1.39. The study highlights the efficacy of pairing a homogenized macro-modeling approach with wave-based and dynamic identification methods, particularly for resource efficiency. Finally, recommendations for future conservation efforts have been provided.



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**Keywords:** historical structure; limestone masonry; inspection; diagnosis; non-destructive testing; dynamic identification; finite element method; finite element analysis; limit analysis; seismic design

## 1. Introduction

In regions with little recent seismic activity, the effect of a seismic event can often be overlooked. With a low seismic hazard, a return period of, at times, hundreds of years, and the efficacy of modern building materials at mitigating risk, seismic risk can be easy to ignore. However, for assets of cultural heritage and historical buildings that have existed for centuries, these risks are inevitable. The seismic evaluation of heritage structures is of particular interest since they are often unique and irreplaceable, holding significant historical and cultural values as built heritage. These structures are commonly built with traditional building materials such as unreinforced masonry and were not designed to withstand the significant lateral loading often accompanying seismic events. In addition, they can be expensive to maintain and difficult to work on. Many of these buildings are characterized by irregular geometry, massive size and weight, and masonry composing them that is brittle and vulnerable to tensile and shear loading. These characteristics influence the seismic behavior and vulnerability of a historical structure [1].

Numerical methods and modeling have successfully been employed for the analysis of heritage structures by allowing the simulation of an applied loading condition [2–5]. This application is commonly used to simulate the response of a structure to either a static loading condition, such as a lateral load being applied to a wall, or a dynamic loading condition consisting of a time-history seismic input. The methods are also commonly considered in both the linear elastic and non-linear regimes for materials and are therefore entirely dependent on the constitutive and material models used in the modeling approach. Typical methods for the modeling of masonry involve the different treatment of the mortar–brick interface that characterizes the material. Micro-modeling approaches are more computationally demanding and can be used to model individual masonry units and the interfaces between them as discontinuous. Macro-modeling approaches represent the masonry structure as a homogenized approximation of its composition; they are therefore less computationally demanding and require fewer individual properties as inputs [6].

The need for validation and support of the model-based approach is essential; experimentally obtained data can complement a numerical model by obtaining experimental values for the mechanical and physical properties of a specific site [7]. However, for a culturally significant and protected site, it is difficult to properly assess their mechanical and compositional properties using *ex situ* laboratory setups or other means that would involve the partial or total destruction of an element. The importance of performing non-destructive tests (NDTs) to assess these parameters becomes immediately apparent; being able to understand the behavior of the structure and its materials without negatively affecting the existing elements is paramount to assessing its current state responsibly. Physical properties, such as compressive strengths for individual units and mortars, can be obtained using more analog methods such as those outlined in this report (Schmidt hammer, Schmidt pendulum hammer, penetrometer, and scratch testing instrument). More wholistic parameters that represent more global properties can be assessed using wave-based methods and dynamic identification, in addition to being able to highlight potential damage in the site [7]. Furthermore, these methods can be used to calibrate a developed numerical model according to their modal parameters of vibration frequencies, mode shapes, and damping [8].

Lanjarón is in one of the most seismically vulnerable regions in Spain [9]. Despite local interest in the preservation of its historical castle, no innovative academic studies within the framework of seismic evaluation and historical structural conservation have been performed. Such investigations complement the use of finite element methods to simulate the response of a structure to external loading by providing material properties that can be used directly in a finite element (FE) model. Within this context, this novel study aims to outline the results of extensive investigations and experimental campaigns of NDTs that were performed on the site to characterize structural elements and identify potential damage to the castle. The study further developed and calibrated an FE model based on a variety of modal parameters and using the Modal Assurance Criterion (MAC) for validation. Finally, efficient methods for the safety assessment of historical structures were used to determine the safety of the structure, all within a comprehensive case study. Future studies may consider using dynamic non-linear analysis to further evaluate the rest of the structure.

## 2. Materials and Methods

The Castle of Lanjarón (Figure 1a) is a 16th century Christian stronghold that occupies a strategic position upon the southern slopes of the Penibaetic Mountains of the Sierra Nevada adjacent to the town of Lanjarón [10]. It sits atop a steep rocky slope and is, today, composed of two main components that have largely fallen into a state of decay: a keep, on



the southern side of the compound (Figure 1b), and the outer enclosure that surrounds it to the north (Figure 1c). A barbican flanks the entrance to the south, running parallel to the walls of the keep above. In this way, the castle is very characteristic of a stronghold on the Iberian Peninsula from its time: the angled approach created by the barbican and the keep above the entrance echo the influence of the Muslim cultures that occupied the region before [11,12].



**Figure 1.** Visual overview of the Castle of Lanjarón: (a) aerial view of the castle looking west [13]; (b) view of the castle from the south showing the barbican and keep; (c) view of the inner enclosure from the entrance under the keep; (d) view showing the long west wall.

The castle itself occupies a footprint of around 500 m<sup>2</sup> and does not have a regular polygonal shape. Its construction is primarily of stone rubble masonry, except for a lime-based cementitious cistern in the base of the large keep on the southern side of the structure. The keep rises to a height of 12 m above the approach to the south, flanked by the 1.2 m thick barbican. A barrel-vaulted passage perpendicular to this approach provides access to the outer enclosure. The enclosure is fortified on all sides by 1.7 m thick walls that rise to heights of almost 7.5 m at their tallest point along the long western wall (Figure 1d). The terrain inclines steeply from south to north, rising from the keep to the apex of the hill the castle sits upon. At its northernmost point, a massive watchtower looks out towards the town to the north.

The existence of the castle in its current state has its origins in the early 16th century, shortly after the Christian occupation of Granada. Material analysis suggests that it is a Christian masonry construction built during the annexation of the area to the Kingdom



of Castile [10]. The primary function of the castle may have been surveillance and control of the area and its inhabitants during that initial period and beyond, until the eventual expulsion of the Muslim population rendered the castle obsolete. This led to its conclusive abandonment in the centuries following. Archeological surveys further support this conclusion, implying a gradual decline in the structural condition of the castle from its historical state to its current one. The collapsed state of the keep is a product of centuries of abandonment and continuous exposure to the elements [14]. The castle was added to the Spanish Heritage Monument Listing in 1985; it is protected and designated today with the status of Bien de Interés Cultural (BIC), or Cultural Heritage Site.

Concern over the collapse of the structure led to a variety of restorative and preservative efforts during the 1990s and the 2000s, including the consolidation of cracks in the keep. Masonry elements were reconstructed with fallen stones to their original state. Other masonry units were covered with a thick mortar where it had been removed, in addition to deep, medium depth, and superficial repointing being performed. Some deep voids were filled and consolidated; heavily deteriorated stone masonry units were replaced, and fallen masses were relocated or repositioned to avoid further deterioration. The most recent intervention, completed in 2007, also featured the construction of an earlier planned metallic frame in the keep outlining the historical shape of the structure, and the addition of stairs and safety rails [15].

Even with the recency of the intervention in 2007, the conditions of many of the elements continue to decline. Much of the paint on the metal elements has chipped or been removed, exposing these elements to decay. Many of the limestone masonry units have been heavily exposed to alveolization. Two drainage tubes are present to remove water and could easily become blocked. Significant plant growth threatens the state of the masonry slowly. For an asset of significant cultural interest such as the Castle of Lanjarón, the main elements and materials of a site are often fragile and of significant value in their current state. Figure 2 demonstrates some of these damages.

An intensive in situ experimental campaign was carried out to assess and identify properties and conditions of the elements comprising the castle. These works included rock characterization, testing with various NDT equipment (Schmidt hammer, Schmidt pendulum hammer, penetrometer, scratch testing instrument), a comprehensive wave-based campaign based on sonic tests, and concluded with dynamic identification.



Figure 2. Cont.





**Figure 2.** Current conservation state of the Castle of Lanjarón: (a) state of metallic elements in the castle; (b) alveolized limestone ashlar; (c) blocked drainage tube; (d) significant plant growth on structural elements hides a masonry arch.

### 2.1. Rock Test Procedures

The rock tests were performed on visually similar rocks in the area surrounding the castle and used common geological approaches to characterize calcareous rocks in situ through both visual means and an acid test. The acid test characterizes rocks containing calcium carbonate according to the reaction of a weak acid with the compound. The frequency and nature of the gaseous carbon dioxide bubbles formed by the acid-base reaction can serve as an indication of whether limestone or another calcareous rock is present [16]. Units that reacted readily and bubbled aggressively in the presence of acid were labeled as limestone. Units first requiring the surface to be scratched to a powder before reacting were considered dolomitic. Units that did not readily react did not contain significant traces of calcium carbonate. Based on these results, a Schmidt hammer with a rebound energy of 2.207 Nm was also used to evaluate the compressive strengths of the masonry units using 10 consecutive blows according to standard testing procedures, followed using densities and calibration tables provided in the literature [17–19].

### 2.2. Mortar Test Procedures

#### 2.2.1. Schmidt Hammer

To quantitatively assess the compressive strength of the mortar of the castle, a Schmidt hammer (PASI SRL Type M) with an impact energy of 0.169 Nm was used (Figure 3a). Using this apparatus, a spring-loaded mass is propelled to impact the surface of a mortar three times in succession before measuring the rebound number. The resulting value was then converted to compressive strength using the operation instructions [20]. Each location was tested in accordance with common application methods from the EN Eurocodes for the application of a Schmidt hammer [21].





**Figure 3.** (a) Schmidt hammer; (b) Schmidt pendulum hammer; (c) penetrometer; (d) scratch test instrument.

### 2.2.2. Schmidt Pendulum Hammer

An alternative to the Schmidt hammer that uses similar principles is the Schmidt pendulum hammer, which instead uses gravity to swing a pendulum and impact a surface to return a rebound number measurement (Figure 3b). This process was performed using the ‘measuring’ instructions outlined in the device’s guidelines for vertical surfaces [22]. The hammer was placed upon the surface of the mortar with the head facing downwards as vertically as possible using light pressure, and the head was raised into the loaded position. When well positioned, this allows the head to fall and impact the surface, rebounding and returning an analog measurement that can then be used to qualitatively assess the mortar from the operation manual.

### 2.2.3. Penetrometer

With the advantage of not being as restricted by the required available surface area of the Schmidt hammer tests, a penetrometer (Diagnostic Research Company SRL RSM-15) was also used to quantitatively evaluate the compressive strength of mortar (Figure 3c). The device is based on the same principles as the Schmidt hammer, using an impact-driven accelerated mass to drive a small needle into a mortar. It is considered non-destructive. The ‘Type-A—Fast’ procedure from the user manual was followed, in which ten consecutive blows from the device drove the needle perpendicularly into the mortar. The needle was then removed, and the depth of penetration was measured. The resulting depth can be converted into a compressive strength value [23].



#### 2.2.4. Scratch Test

A scratch test instrument (Enertren Pty. Ltd. MortarCheck II) was used for further onsite evaluation of the mortar condition and strength (Figure 3d) [24]. The device was placed upon a flat surface of mortar, and properly oriented by placing its legs on the surrounding wall. The abrasive probe was then lowered onto the surface with the spacer still attached and zeroed accordingly. The device was locked into place, the spacer was removed, and the force of the spring drove the probe into the mortar as the yoke was turned using two full rotations. Care was taken not to apply additional pressure to the probe while supporting the legs, and the resulting reading (in inches) was used to qualitatively evaluate the mortar scratch index.

#### 2.3. Sonic Test Procedures

Tests based on the propagation of a wave through a medium are of particular interest within the context of historical constructions due to their non-invasive characteristics and ability to gain information about what is below the visible surface. Examining material through the behavior of wave movement can reveal several characteristics of the medium, such as the presence of cracks or separation of wythes within a masonry wall. The speed at which a wave propagates is the basis of these methods, alongside some theoretical assumptions commonly used in civil engineering. Additionally, there is a relation between the elastic modulus, density, and speed of the wave. The equations used, presented later, are based primarily on the homogenous and isotropic behavior of a linear elastic material and how a wave propagates through it. As masonry is not homogeneous itself, it becomes important to note the common homogenization approaches used for heterogeneous materials when studying them at a macro-level [25].

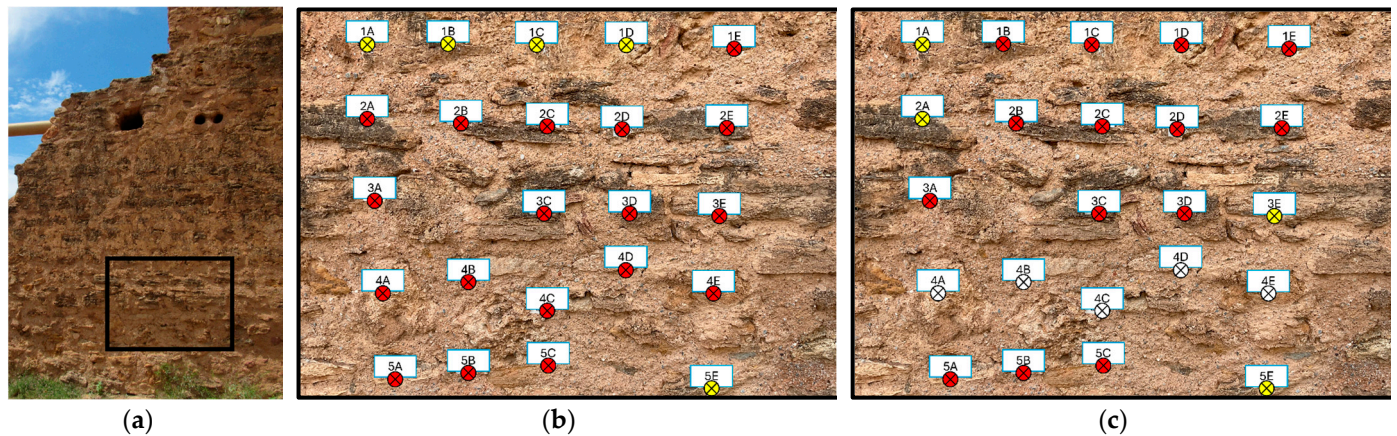
Testing was performed using an impact hammer (PCB Piezotronics Model 086D05) and accelerometer (Model 352B) in combination with a National Instruments (Model NI USB-4431) Data Acquisition (DAQ) system with a maximum input range of  $\pm 10$  V. This setup was used to measure the time between the impact of the hammer and the moment the accelerometer received the signal. The hammer had a sensitivity of 0.23 mV/N and a signal input range of  $\pm 22,240$  N pk. The accelerometer used had a sensitivity of 1 V/g and a signal input range from  $-5$  g to 5 g. The program used in data acquisition was set up to perform continuous sampling with a 100,000 Hz sampling rate, to collect 50,000 samples. A series of six repeated samples were collected from each point to ensure at least three usable tests were gathered.

A  $1\text{ m} \times 1\text{ m}$  area was selected for testing in two different locations, one for direct (through the wall) and one for indirect (across the surface of the wall) testing. An approximately evenly spaced  $5 \times 5$  grid of points was used for indirect testing (Figure 4), while a  $6 \times 6$  grid of points on opposite sides of a wall was selected for direct testing (Figure 5). Points crossing a crack that was repaired in a previous intervention were also investigated using the indirect method. Indirect tests were performed in both a vertical and horizontal direction by striking a point at either extreme of the grid and subsequently progressing through the points along the line and repeating the process.

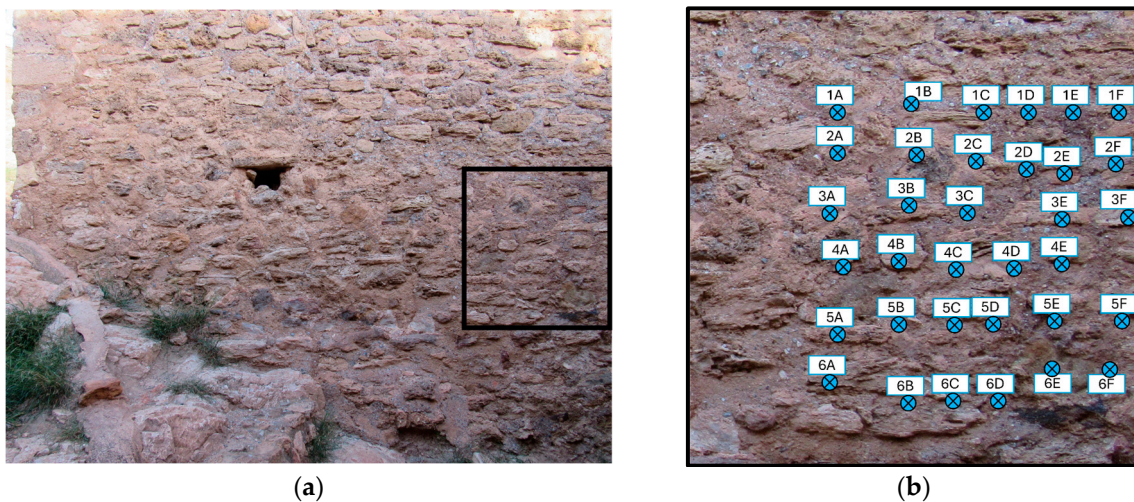
The collected data were then processed, and the modulus of elasticity for the materials was calculated using the following formula:

$$E = V_p^2 \rho \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)} \quad (1)$$

where  $E$  [N/m<sup>2</sup>] is the modulus of elasticity,  $V_p$  [m<sup>2</sup>/s] is the wave propagation speed,  $\rho$  [kg/m<sup>3</sup>] is the material density, and  $\nu$  [–] is the Poisson's ratio [3,26]. Material properties from the literature were used for values of density and Poisson's ratio [6].



**Figure 4.** (a) Indirect sonic test location (extent shown by black rectangle); (b) test grid with locations for vertical indirect sonic tests; (c) test grid with locations for horizontal indirect sonic tests. Hammer impact points are highlighted in yellow, accelerometer points are red, and points with null readings are white.



**Figure 5.** (a) Direct sonic test location outside the keep (extent shown by black rectangle); (b) test grid with locations for direct sonic tests.

#### 2.4. Dynamic Identification Procedures

Real-time dynamic identification can be used to obtain a variety of dynamic structural properties that describe how a structure oscillates and behaves. The oscillation of a structure is mathematically governed by its mass, stiffness, and capacity to dissipate energy; these parameters are represented generally in dynamic identification through modal parameters. Modal parameters are typically frequencies of vibration, the relative shapes of the modes associated with these frequencies, and damping [8]. The modal parameters can wholistically represent the response of a structure to an input, regardless of any damage or heterogeneity in the structure [4]. They can represent the current state of a structure, together with its material and mechanical properties. Periodically performing dynamic identification on a structure, especially to collect data before and after some damage has occurred, is an effective way to continuously monitor its health non-destructively and without the need for more invasive testing [8].

For this campaign, 8 piezoelectric accelerometers (Model 393B12 PCB Piezotronics) with a sensitivity of 10,000 mV/g, a measurement range of  $\pm 0.5$  g pk, and a frequency range of 0.15 Hz to 1000 Hz were used with a National Instruments DAQ (Model NI 9174 with two NI 9234) for data collection. A sampling rate of 200 Hz for 30 min was used for

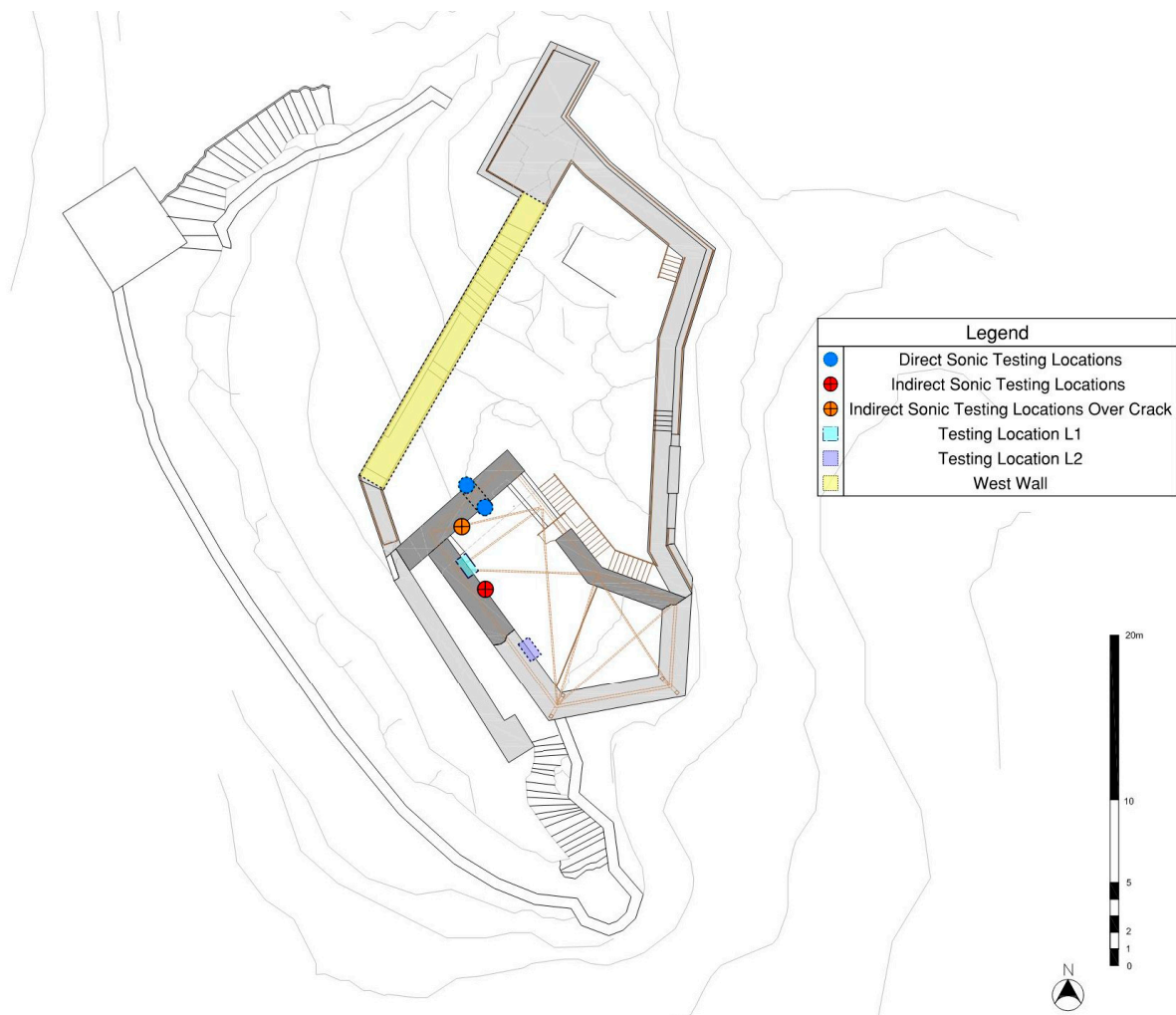


each acquisition; this frequency is commonly applied for masonry heritage structures as it can capture frequency content up to 100 Hz [2,7]. Under ambient conditions for excitation, which is considered an Ambient Vibration Test (AVT), the setups are unlikely to capture any bandwidth above 20 Hz. Using a common reference point and accelerometer, multiple acquisitions of data can be combined. For the analysis of the west wall, two instances of 30 min acquisitions with two distinct setups were used to measure the response of the structure to an unexcited input. This method of output-only dynamic identification measures the ambient vibration of the structure, assumed to follow the input of a Gaussian white noise stochastic process for Operational Modal Analysis (OMA) [8]. Acquisitions were then processed using ARTEMIS 6.0 modal identification software to identify modal parameters according to their spectral densities.

### 3. Results

#### 3.1. Mortar Test Results

A variety of NDT equipment was employed to characterize the mortar onsite, and to identify differences in the types of mortar present from previous interventions. Figure 6 presents the locations of the tests performed and highlights the west wall.



**Figure 6.** Plan view of site with test locations, highlighting the west wall.

Two locations were identified and investigated using these instruments, each on the south-western wall of the keep. The first was a harder and more densely mortared section of the wall, representative of much of the mortar that has been repointed in previous

interventions, called ‘L1’. The second was a more eroded variant of the first mortar with more visible aggregates and was expected to be softer; this was called ‘L2’. The obtained results are summarized in Table 1.

**Table 1.** Summary of results from mortar testing.

| Test                    | Result               | Unit | Location L1 | Location L2 |
|-------------------------|----------------------|------|-------------|-------------|
| Schmidt Hammer          | Compressive strength | MPa  | 10.8        | 2.8         |
| Schmidt Pendulum Hammer | Quality              | -    | Average     | Reasonable  |
| Penetrometer            | Compressive strength | MPa  | 1.8         | 0.6         |
| Scratch Test            | Durability grade     | -    | -           | -           |

### 3.1.1. Schmidt Hammer

The Schmidt hammer calibrated for mortars was used to evaluate the compressive strength of the two test locations. Calibration tables yielded a compressive strength of 10.8 MPa for L1 and 2.8 MPa for L2. These values are notably high for mortars when compared to values from the literature [6]. This could be explained by the fact that the test is performed on a superficial layer of mortar and additionally representative of the mortar strength from recent interventions on the castle.

### 3.1.2. Schmidt Pendulum Hammer

Some notable difficulty was observed in the application of the Schmidt pendulum hammer; the apparatus relied mainly on perpendicular application against the wall, often a significant challenge in historical structures due to the irregularity of the masonry. Too significant an inclination in either direction could artificially increase or decrease the measurement. Nevertheless, nine appropriate measurements for each testing location were obtained, returning rebound values between 25 and 36. These numbers were then used to qualitatively assess the mortar condition, observed as “average” for the more intact location (L1) and “reasonable” for the more eroded one (L2).

### 3.1.3. Penetrometer

Penetrometer measurements are likely to be more representative of the mortar below the surface due to the nature of the tests and returned lower values than the superficial impact hammer tests. The resulting compressive mortar strengths of L1 and L2, respectively, were 1.8 MPa and 0.6 MPa, implying a significant difference in the quality of the mortars.

### 3.1.4. Scratch Test

The scratch test relies heavily on the proper application of the instrument using three moveable legs to orient the device perpendicular to the surface, and a spring to drive the abrasive probe into the mortar. This process is notably more difficult to do on an irregular surface. Care was taken to not apply additional pressure to the probe as the measurement was being taken, and multiple additional test locations were attempted as a result. Despite best efforts, the practical applicability of the test on a very irregularly surfaced historical structure was very difficult to perform and interpret properly. As such, no reliable readings were obtained from this test.

### 3.1.5. Mortar Test Conclusions

The overall state of the mortar appears to be reasonable, supported by the results from the Schmidt pendulum hammer. Despite vastly different interpretations of the compressive strength of the mortar from the Schmidt hammer and penetrometer test, the mortar of location L1 is observed to be around three times stronger than that of its more eroded



counterpart L2. The discrepancy in the results from the differing tests could be explained by the fact that the results from the Schmidt hammer represent only the strength of the newer mortar added in more recent historical interventions on the castle. By the nature of the application, the penetrometer could be more likely to represent the true strength of the historical mortar used in the castle, which is the working assumption of this paper.

### 3.2. Rock Test Results

Three unique types of masonry units were identified. Limestone was the most evident, particularly in heavily alveolized units that had degraded significantly. A layered dolomite or dolomitic limestone was also identified, as well as an amount of a harder marble. Identification was possible based on the reactivity of a weak (acetic) acid on the surface and is further supported by a geological survey from the Spanish Geological and Mining Institute [27]. The compressive strengths of the units tested are reported in Table 2, assuming material densities available in the literature [19]. The rock identified as marble presented the highest compressive strength at 40 MPa, while the dolomitic samples reported the weakest at 34 MPa. The similarity of these values, especially compared to the much weaker compressive strength of mortar, further supports the use of a homogenized approach to modeling masonry.

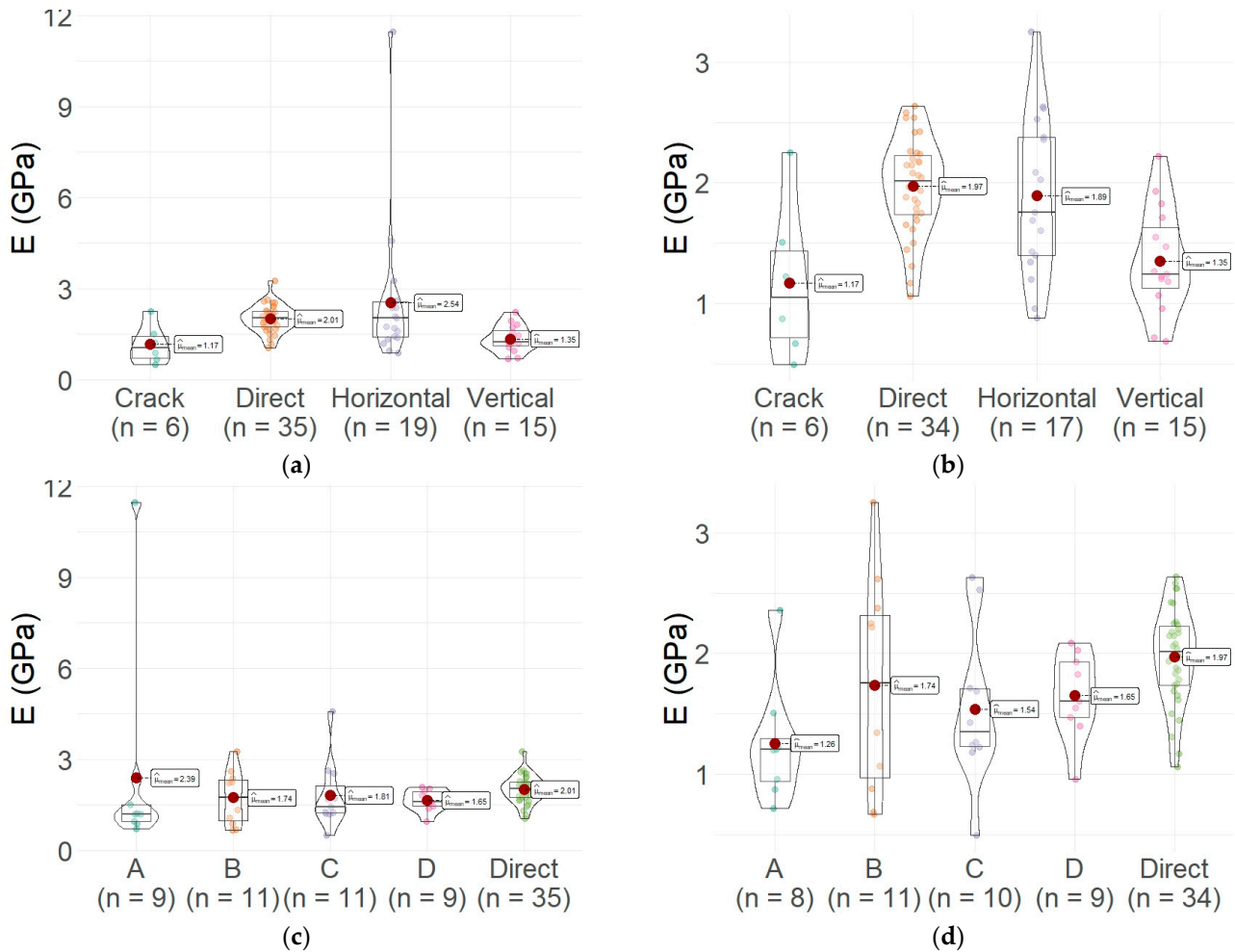
**Table 2.** Summary of results from rock testing.

| Material  | Specific Weight (kN/m <sup>3</sup> ) | Compressive Strength (MPa) |
|-----------|--------------------------------------|----------------------------|
| Dolomite  | 24                                   | 34                         |
| Limestone | 24                                   | 37                         |
| Marble    | 26                                   | 40                         |

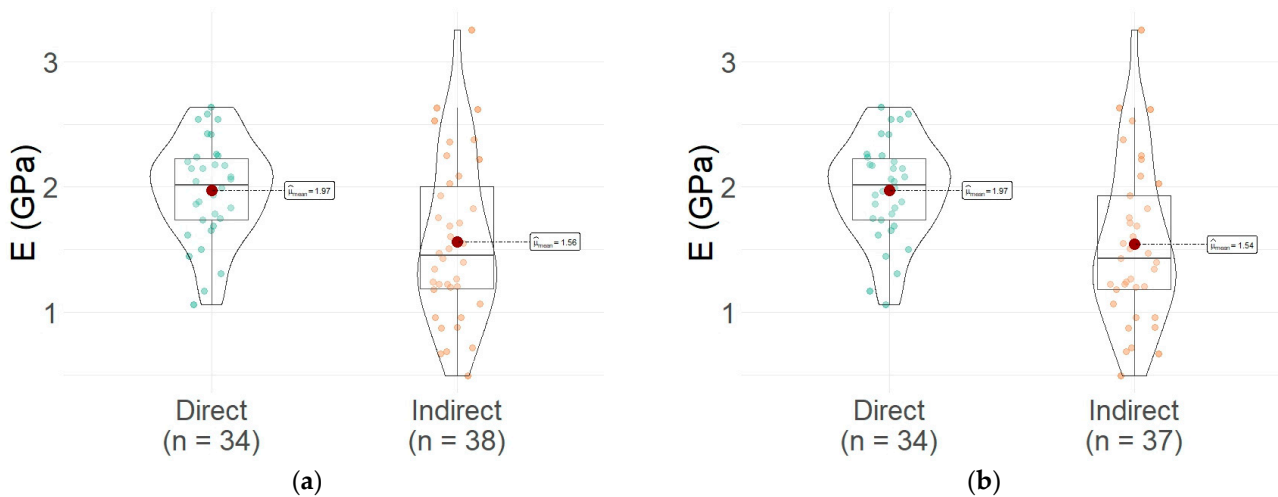
### 3.3. Sonic Test Results

Sonic test results were divided into direct and indirect methods, totaling 336 results for the direct test and 353 for the indirect test over 79 different testing locations. The data were organized twice using both test type (direct, horizontal indirect, vertical indirect, or over the crack indirect) and approximate distance traveled by the wave to group and remove outliers. The distance category is further subdivided into categories A (20–25 cm), B (40–50 cm), C (60–75 cm), and D (80–100 cm). For the purpose of removing outliers, these grouped datasets were divided into four quartiles with equal populations, and the interquartile range between the first and third quartiles was calculated. Values beyond 1.5 times this range from the median in either direction were defined as outliers and removed, and the results were reported [28]. The results are shown in Figure 7 using boxplots.

Reconstructing the dataset and comparing the resulting elastic moduli after filtering by each method suggests a value between 1.5 and 2.0 GPa. The former is more in line with the results from the indirect testing and the latter with those from the direct testing methods. Boxplots present the reconstructed datasets after each type of filtering; note that the results from the direct test are unchanged since the filtering criteria do not affect them, and that filtering by distance was able to remove one additional data point (Figure 8). The higher value of 2.0 GPa was used as a starting point for initial calibration of the numerical model in Section 4. The results are summarized in Table 3.



**Figure 7.** Results from sonic testing: (a) unfiltered elastic moduli by test type; (b) filtered elastic moduli by test type; (c) unfiltered elastic moduli by test distance; (d) filtered elastic moduli by test distance.



**Figure 8.** Results from sonic testing: (a) reconstructed dataset after test type filtering; (b) reconstructed dataset after distance filtering.



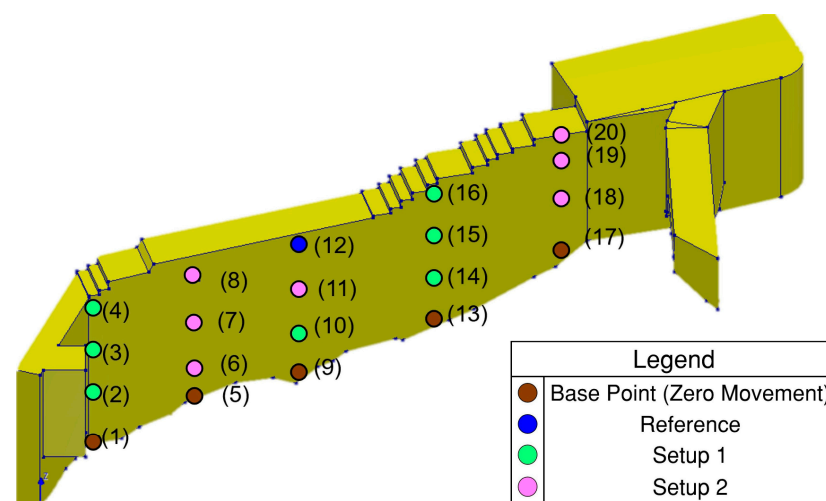
**Table 3.** Mean values from sonic testing before and after outlier filtering based on test type and distance.

| Test ID       | Filtering | Mean of Elastic Modulus Before Filtering (GPa) | Mean of Elastic Modulus After Filtering (GPa) |
|---------------|-----------|------------------------------------------------|-----------------------------------------------|
| Crack         | Type      | 1.17                                           | 1.17                                          |
| Horizontal    | Type      | 2.54                                           | 1.89                                          |
| Vertical      | Type      | 1.35                                           | 1.35                                          |
| A (20–25 cm)  | Distance  | 2.39                                           | 1.10                                          |
| B (40–50 cm)  | Distance  | 1.74                                           | 1.74                                          |
| C (60–75 cm)  | Distance  | 1.81                                           | 1.54                                          |
| D (80–100 cm) | Distance  | 1.65                                           | 1.65                                          |
| Indirect *    | Type      | -                                              | 1.56                                          |
| Indirect *    | Distance  | -                                              | 1.54                                          |
| Direct        | Type      | 2.01                                           | 1.97                                          |

\* Reconstructed datasets after separation and filtering.

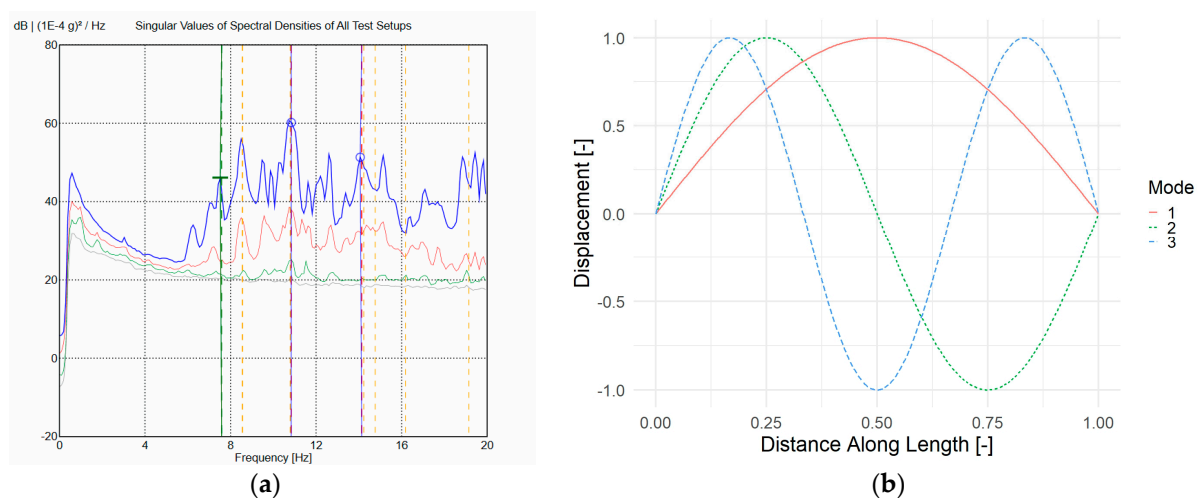
### 3.4. Dynamic Identification Results

Data for the dynamic identification of the west wall were collected with two, 30 min acquisitions. Using multiple acquisitions allowed for multiple recordings of data to be combined, thereby improving the amount of data collected on the wall. Using the same accelerometer and location for multiple acquisitions is essential for this. Fifteen accelerometer locations were selected; with eight accelerometers, this allowed for seven to be moved between setups, while one was kept in place. Five vertical lines across the wall were selected, allowing for three accelerometers to be placed vertically to record the shape and curvature of the wall at each line. An additional point below the lowest accelerometer was also added as a base, with no assumed movement or assigned measurement. The highest point on the midspan, shown as point 12 in the diagram, was selected as the reference point; for each acquisition, three accelerometers were placed along a line to the left of the reference, three accelerometers were placed in a line to the right, and one was placed below the reference point (Figure 9). It is important to note that accelerometers were only placed in the out-of-plane direction, as the stiffness of the wall in the in-plane direction was considered too high to present any descriptive mode shapes.

**Figure 9.** FE model of west wall with nodes for MAC calculations.

The acquired data were processed using ARTEMIS 6.0 modal software for dynamic identification. The data were filtered to consider the frequency content from 0 Hz to 100 Hz

for decimation, with a resolution of 1024. Raw signals were converted from a time domain to a frequency domain for processing in this way. Identification of the modes was performed using both a manual peak-selection process and the support of the Enhanced Frequency Domain Decomposition (EFDD) and Stochastic Subspace Identification-Unweighted Principal Components (SSI-UPC) to validate the selected shapes (Figure 10a). Three modes were identified for the west wall setup, consistent with those from a fixed string vibration system due to the length of the wall. The frequencies for the first, second, and third mode were, respectively, 7.58 Hz, 10.84 Hz, and 14.13 Hz. Spatially, the first mode was composed of a single antinode at the midspan where the maximum displacement of the mode occurred, with two nodes at the ends of the wall. The second mode introduced an additional node and antinode, with the maximum displacement occurring at one-quarter of the length of the wall to effectively half the wavelength of the mode. The third mode introduced a final node and antinode pair, returning the maximum displacement to the midspan and introducing another half-wavelength to the system. These shapes are visible in Figure 10b.



**Figure 10.** Overview of results from dynamic identification: (a) spectral density for west wall setups with estimators; (b) first three mode shapes for the fundamental frequencies of a fixed string.

### 3.5. Validation

Validation for the experimentally obtained properties was explored by their ability to calibrate a numerical model in Section 4. Of particular interest were the values obtained by sonic testing and dynamic identification, which were used to calibrate the elastic modulus and outline the mode shapes used in the modeling and calibration process. The ability of the model to replicate these characteristics will be used to validate experimentally obtained values.

## 4. Finite Element Modeling Approach

### 4.1. Modeling Approach

Various methods are commonly applied for the evaluation of masonry structures using numerical models. Masonry as a building material comprises a combination of bricks and mortar, with each element being equally important to the composition. Individual units generally make up the majority of masonry, while the mortar fills in the gaps between the units. Thus, the models used to represent masonry vary in their treatment of the interface and interaction between these elements. Detailed micro-modeling represents all elements individually using continuum elements to model the masonry units and mortar and introducing a discontinuous element as an interface between these units. This allows for the detailed application of mechanical and constitutional models to all of the

material types individually. A more simplified approach can be adopted in simplified micro-modeling, in which the masonry units are geometrically expanded to contact one another using continuum elements, and the mortar and unit–mortar interface are modeled together using discontinuous elements [6]. For the application of both these methods, knowledge about the exact geometry and composition of not only a structure, but also the individual masonry elements that comprise it, is required. This is often extremely difficult to apply in a historical context, especially where very irregular and random arrangement and size of units is expected as in the case of the castle. These approaches are also very computationally expensive, requiring much greater computational time and storage capabilities [29]. Macro-modeling was therefore used for the purpose of this study, in which the masonry units and mortar interface are all treated wholistically as a homogenous anisotropic material. This homogenized approach is well supported by the application of non-destructive tests where little additional information about the nature of a site can be gathered, further increasing the efficiency of the methods. Sonic testing and modal analysis, therefore, are complimentary approaches in ascertaining homogenous material properties [6].

A 3D finite element model was prepared using DIANA Finite Element Analysis (DIANA FEA) 10.8 for numerical simulations [30]. The model considered the extent of the masonry elements comprising the castle in three dimensions, based on an existing photogrammetric survey of the site (available in Supplementary Materials). The exterior walls were modeled from the foot of the wall outside the castle to the height of the masonry, disregarding railings or the infill of the soil inside the enclosure. The walls were then extruded inwards. The faces of the underside of the model were considered with translationally fixed boundary conditions. For this study, a separate model considering the large western wall of the enclosure was also prepared. The locations of the accelerometers from dynamic identification were also imprinted on the model to directly compare the models (Figure 9) and calculate the MAC later.

#### 4.2. Mesh Definition

To model the 3D structure, 20-node solid elements (Type CHX60) were used. A maximum element size of 0.34 m was used to ensure that the model would consider five elements across the thickness of the wall to account for possible curvature in a non-linear model.

#### 4.3. Material Properties

The basic material properties used for the FEA are outlined in both a linear elastic and non-linear regime. Linear properties were obtained experimentally and from the literature, namely the Italian code NTC 2018 [31]. Non-linear properties were considered in the model using a total strain-based crack model with rotating crack orientation. The compression regime considered a parabolic compression curve with no residual compressive strength, reduction due to lateral cracking, or stress confinement in the model. The tensile regime considered an exponential stress curve with no residual tensile strength or Poisson's ratio reduction. Compressive strength and fracture energy were evaluated using formulae from the literature, shown in Equations (2) and (3), respectively [6]. Values for tensile strength and fracture energy come from the literature [6]. Table 4 outlines these properties for both regimes.

$$E = 500 f_c \quad (2)$$

$$G_c = \frac{32 f_c}{10 + f_c} \quad (3)$$



**Table 4.** Linear elastic and non-linear properties used for masonry.

| Property                    | Symbol  | Unit              | Value |
|-----------------------------|---------|-------------------|-------|
| Density                     | $\rho$  | kg/m <sup>3</sup> | 1900  |
| Elastic Modulus             | E       | GPa               | 2.0   |
| Poisson's Ratio             | $\nu$   | -                 | 0.2   |
| Compressive Strength        | $f_c$   | MPa               | 3.0   |
| Compressive Fracture Energy | $G_c$   | N/mm              | 7.40  |
| Tensile Strength            | $f_t$   | MPa               | 0.1   |
| Tensile Fracture Energy     | $G_f^I$ | N/mm              | 0.02  |

#### 4.4. Model Calibration

Calibration was necessary to tune the initially assumed material properties of the model to better represent those obtained experimentally from dynamic identification. An eigenvalue analysis of the FE model was used to identify the frequencies and mode shapes of the structure in the model for direct comparison with those obtained experimentally. The frequencies determined in this method are directly impacted by both the mass and the stiffness of the structure; therefore, mass was fixed, while stiffness was adjusted for calibration. The relevant frequencies from DIANA are obtained from an eigenvalue analysis, in which the solutions to the free vibration equation (Equation (4)) are represented by the eigenvalue  $\lambda$  (Equation (5)) [32].  $K$  represents the stiffness matrix,  $M$  the mass matrix,  $\phi$  the corresponding mode shape vector,  $\omega$  the angular frequency, and  $f$  the frequency:

$$K\phi = \omega^2 M\phi \quad (4)$$

$$\lambda \equiv \omega^2 = 4\pi^2 f^2 \quad (5)$$

For the calibration, the initial elastic modulus was reduced from 2.0 GPa to 1.5 GPa, which reduced the frequency error for similar mode shapes to under 2.0% for the first two frequencies, and under 9% for the third. The average absolute error was therefore 3.8% with a maximum error of 9.0%. Modes were further validated from the eigenvector analysis by selecting the first three that moved above 20% of the mass of the wall each, from an initial analysis that considered up to 70% of the mass moved from 20 eigenfrequencies computed. A visual comparison between the numerical and experimental modes demonstrates their similarity (Figure 11).

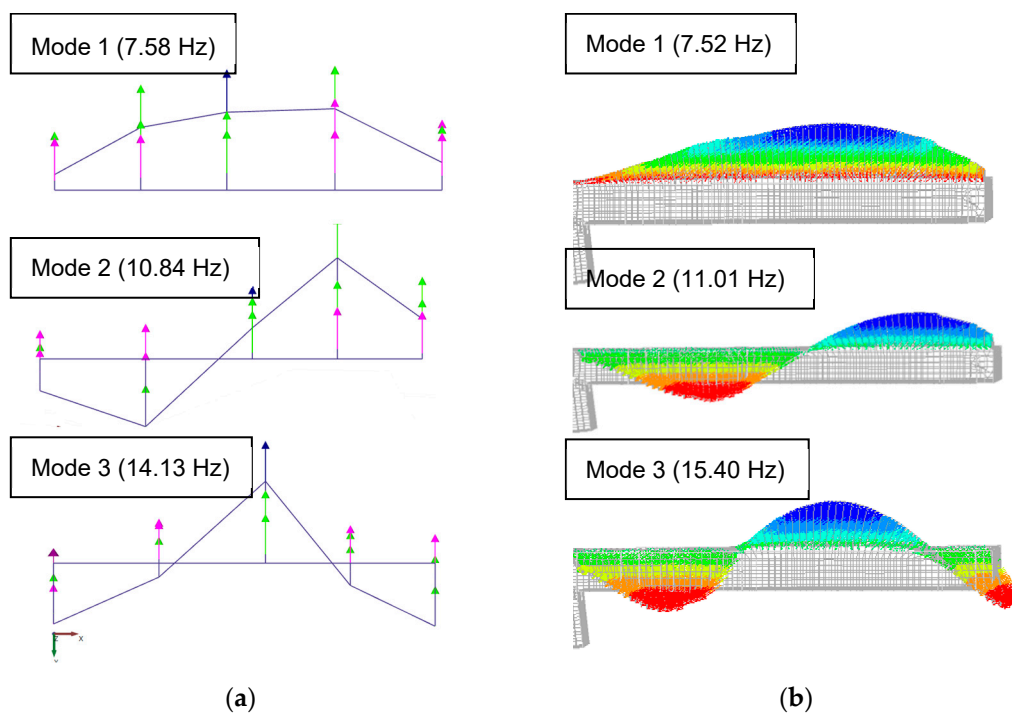
#### 4.5. Modal Assurance Criterion

Once the elastic modulus and frequencies had been determined both experimentally and numerically, it was possible to calculate the Modal Assurance Criteria (MAC) to compare the FEA model with the experimental one. The MAC is the most commonly used method in the literature to calculate the correlation between two vectors. In this case, these vectors represent the mode shapes obtained experimentally and numerically. Mathematically, the MAC calculates the scalar product of two normalized vectors [33]. The MAC compares the values of any number,  $n$ , entries in two  $1 \times n$  matrices, and could therefore directly be used to compare the displacement vectors between the numerical and experimental results for  $n$  nodes. Of note is the fact that the formulation of the MAC in this case is based on the vectors used representing only one-dimensional space, since the accelerometers are all placed in an out-of-plane direction. The displacement vector outputs from the numerical model for each of the relevant nodes were therefore expressed only in the out-of-plane direction. One displacement value for each node, at each point that an accelerometer was placed, was then entered into a vector based on the node's ID so that relevant pairs of numerical and experimental displacement vectors could be compared.

The output of the MAC is a single number ranging from 0 to 1 depending on how similarly the methods express the explored mode shape; 1 represents complete correlation, while 0 represents complete disagreement of the values and therefore of the mode shapes. The formulation is written without the need for normalization:

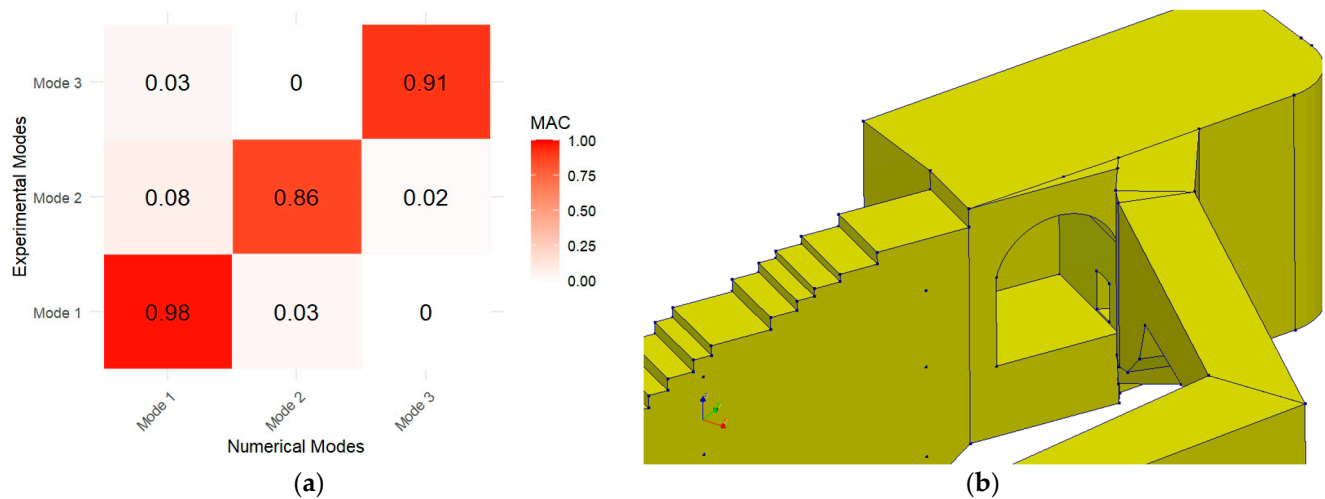
$$\text{MAC}(x_e, x_n) = \frac{x_e^T x_n}{\|x_e\|_2 \|x_n\|_2} \quad (6)$$

where  $x_e$  is a vector of experimental displacement vectors, and  $x_n$  is a vector of numerical displacement vectors [34]. These vectors were obtained from DIANA directly at the nodes specified using the mesh by imprinting and considering only the vectors in the out-of-plane directions.



**Figure 11.** Mode shapes for west wall: (a) initial three experimentally determined mode shapes; (b) initial three numerically determined mode shapes.

Initial MAC calculations for the system revealed an overall good agreement between the mode shapes obtained, with all values above 0.8. The calculated MAC value was 0.96 for the first mode, 0.84 for the second mode, and 0.94 for the third mode. A graphical comparison between the vectors obtained through both methods revealed that the points of maximum displacement, although similar, occurred in different points between the models. The experimental behavior of the structure did not match the numerical one at the points closest to the north tower. This suggested the need to reduce the stiffness of the tower due to the presence of an arched room which represented a significant void in the structure (Figure 12). The ultimately successful approach was the addition of an approximation of the void room in the structure, which improved the MAC values for the first mode to 0.98 and the second mode to 0.86 while slightly reducing that of the third mode to 0.91. This also slightly increased the frequency error in the first mode to 1.1% while reducing that of the second mode to 0.3% and that of the third mode to 8.8% (Table 5).



**Figure 12.** (a) MAC comparison between numerical and experimental mode shapes for model considering the addition of a void in the north tower; (b) model representation of the void in the north tower.

**Table 5.** Comparison of experimentally obtained frequencies with calibrated and uncalibrated numerically obtained frequencies (percent difference in parentheses).

| Mode | Experimental Frequency (Hz) | Uncalibrated Numerical Frequency (Hz) | Initial Calibrated Numerical Frequency (Hz) | Updated Calibrated Numerical Frequency (Hz) |
|------|-----------------------------|---------------------------------------|---------------------------------------------|---------------------------------------------|
| 1    | 7.58                        | 8.68<br>(14.58%)                      | 7.52<br>(−0.76%)                            | 7.49<br>(−1.11%)                            |
| 2    | 10.84                       | 12.71<br>(17.29%)                     | 11.01<br>(1.60%)                            | 10.87<br>(0.34%)                            |
| 3    | 14.13                       | 17.79<br>(25.87%)                     | 15.40<br>(8.99%)                            | 15.37<br>(8.80%)                            |

## 5. Analysis

The structural performance of the castle was evaluated using two methods of applying an out-of-plane force to overturn the structure. The first of these methods was a simple limit analysis to outline expectations; the second used a non-linear static analysis, also known as a pushover analysis in an FEA environment. This method of evaluating the seismic behavior of an irregular historical masonry structure has been successfully used and is well documented [1]. All methods were compared to the expected peak ground acceleration for the region according to the local design codes [9]. The results of these analyses represent a factor of gravitational acceleration, equal to  $9.81 \text{ m/s}^2$ , applied to the self-weight of the structure uniformly across its body.

### 5.1. Limit Analysis

The brief analytical limit analysis was conducted for a 1 m long cross-section of the west wall, considering a thickness in the out-of-plane direction of 1.7 m and the average of the minimum and maximum height of the wall of 6.4 m. The analysis considered a force proportional to the self-weight being applied at exactly the mid-height, using a factor ( $\alpha$ ) to represent this force. The factor is defined as the ratio of the overturning of the force to the resisting moment of a structure's self-weight at the point of equilibrium. The analysis



was performed according to a triangular, elastic distribution of stress at the base of the wall using the following equation:

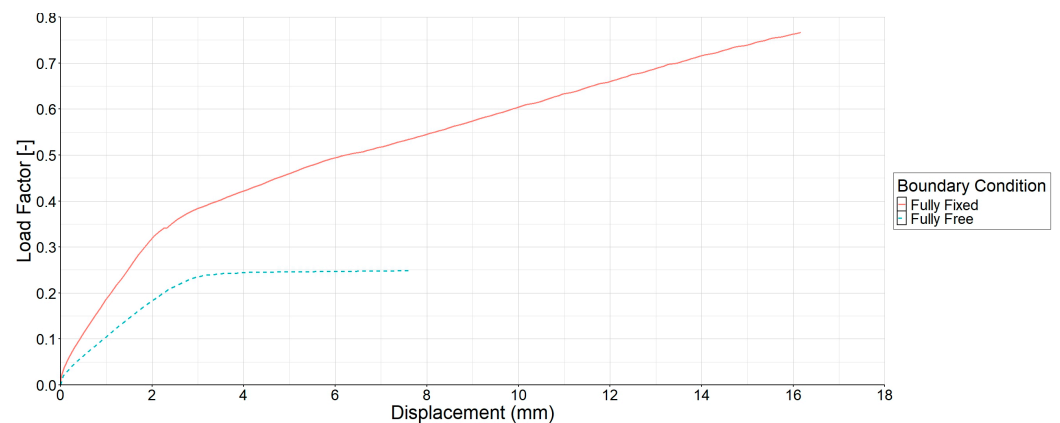
$$\alpha = \frac{W * \left( \frac{tk}{2} - t \right)}{W * \frac{h}{2}} = \frac{\frac{tk}{2} - \frac{2*W}{3*fc*L}}{\frac{h}{2}} \quad (7)$$

where  $tk$  [m] is the thickness of the wall,  $h$  [m] is the height of the wall,  $W$  [kN] is the self-weight of the wall set in the geometric centroid,  $fc$  [kN/m<sup>2</sup>] is the compressive strength of the masonry,  $L$  [m] is the length of wall considered, and  $t$  [m] is the distance of the hinge from the side of the wall. The value of  $\alpha$  using this method was 0.25 g, or 25% of the self-weight applied laterally to the structure.

## 5.2. Non-Linear Static Analysis

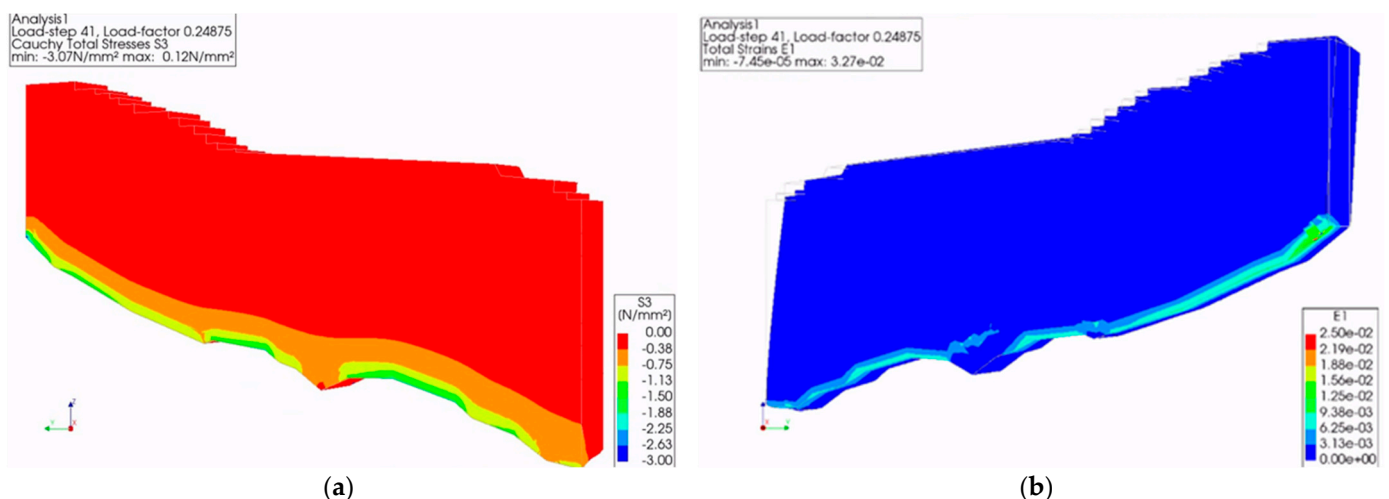
The FE model of the west wall was further reduced to represent only the wall itself and not the adjoining structures of the north tower and the southern wall. In this way, it was possible to explore extreme cases of the end conditions of the adjoining masonry. The lowest of these conditions represents a worst-case scenario, in the form of a weak connection to the adjoining masonry. If this minimum condition is considered safe, then any greater state of connection will be as well. The other extreme considered the connecting surfaces as fully translationally fixed with the intent of modeling a failure pattern indicative of arching behavior. The real state of the wall should be somewhere between these two extremes. In each case, the load in the out-of-plane direction was sequentially increased using the secant method for the convergence of the boundary value problem. An energy method with a convergence criterion of  $10^{-3}$  was used, alongside arc-length and line search methods to assist in convergence, following methods outlined in the literature [6]. Each model was first loaded vertically under the self-weight of the wall using five equal load steps before the sequential non-linear static, or pushover iterations were performed.

Ultimately, the walls were able to exhibit non-linear behavior, as evident by the approximately bilinear regimes outlined by the resulting capacity curves for both conditions (Figure 13). The capacity curve modeled by the pushover analysis presents the differences in the two utilized boundary condition models visually. The fully fixed condition immediately exhibits a steeper rise in load factor for a given displacement, attributed to the increased stiffness and capacity provided by the stronger boundary conditions. The slight dip in the load factor that appears just after 2 mm could be indicative of the formation of cracks and the dissipation of energy as the slope changes and the model enters the flatter non-linear regime. The fully free model exhibits a less significant rise in the initial slope, requiring a smaller load factor to achieve a similar amount of displacement. As the non-linear behavior begins at a load factor slightly above 0.2, the lack of energy dissipation capacity due to the absence of boundary constraints results in a much less significant increase in displacement for a given load factor as the masonry continues to deform. The nature of this deformation will be discussed later. The completely free end conditions exhibited a maximum load factor of 0.25 g being applied laterally to the wall, while the fully fixed conditions continued to a value of almost 0.80 g.



**Figure 13.** Capacity curve comparing fully fixed and fully free end conditions for pushover analysis of the west wall.

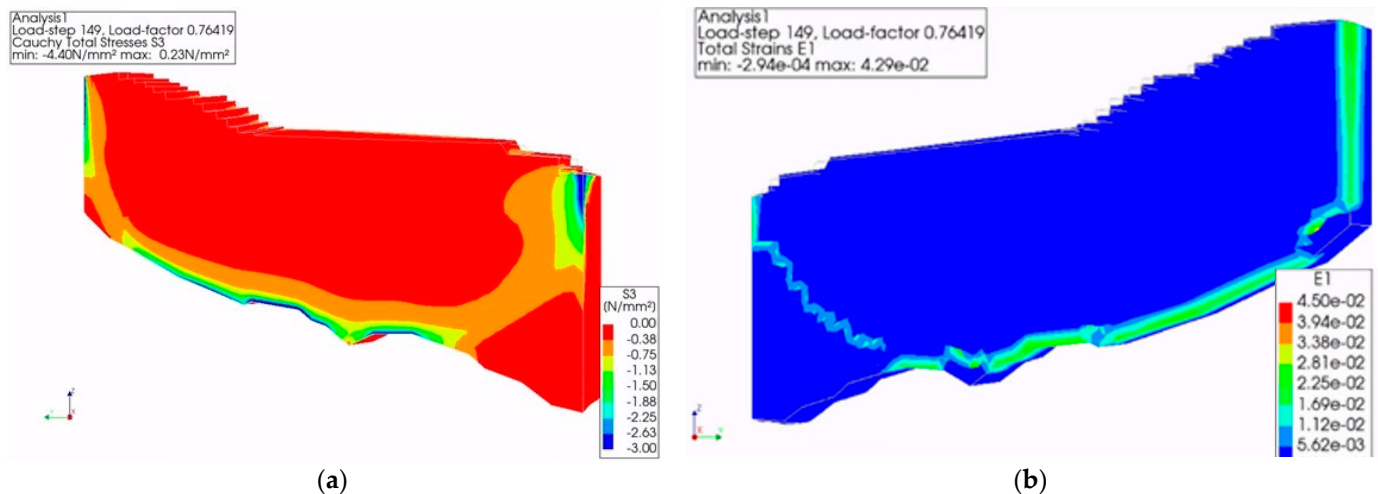
For the version of the model considering fully free end conditions, a total of 37 converged steps were able to outline the deformed shape of the wall as it was sequentially loaded through the linear elastic regime, into non-linear behavior. A maximum displacement of 7.8 mm was reached at the highest portion of the wall, and the structure was able to withstand 0.25 g in the out-of-plane direction. As the structure reached failure, compressive stresses in the outer toe of the wall increased, approaching the crushing strength of the masonry, as outlined by the minimum principal stress in the model (Figure 14a). At the same time, a crack formed on the opposite side of the wall, consistent with the expected behavior of an overturning cantilevered wall (Figure 14b). Cracks began to appear first at the upper base of the wall, indicated by the maximum principal strain in the model. Some nuances were observed as a result of the complexity of the lower portion of the wall used in modeling; the maximum stress did not follow the base across its length and instead jumped over the mid-portion of the base that extends downward.



**Figure 14.** Results from the pushover analysis of the west wall with free end condition: (a) minimum principal stress (view from the outside of the wall); (b) maximum principal strain (view from the in-side of the wall).

The second variation of the analysis, which used fully fixed end conditions, was stopped after 144 successfully converged load steps. The model reached a capacity of 0.76 g, over three times higher than the previous model. The deformed structure demonstrated an arching behavior at the height of the wall as the midspan bulged outwards in the direction the load was applied. From the point of maximum displacement, the displacement at

a given point decreased radially until the fixed end conditions were met. Compressive principal stresses indicated crushing at the base of the wall and along the outer edges of the exterior of the wall (Figure 15a). Maximum principal strain outlined the formation of a curved crack on the interior of the wall as it begins to fail, consistent with the arching behavior (Figure 15b). The crack follows the base of the wall at the upper side of the wall, which is both shorter and more elevated than the taller, lower section in which the crack propagates directly through the masonry. The fixed end conditions allow the wall to behave more like a beam and less like a cantilever, allowing it to resolve shear forces and relevant stresses due to horizontal loading at the end conditions.



**Figure 15.** Results from the pushover analysis of the west wall with fully fixed end condition: (a) minimum principal stress (view from the outside of the wall); (b) maximum principal strain (view from the inside of the wall).

### 5.3. Safety Evaluation

The actual conditions of the wall are likely to exhibit behavior with characteristics of both failure patterns. The support conditions suggested by the model calibration are of significantly good quality, though unlikely to be completely translationally fixed. For the purpose of evaluating the safety of the structure under seismic loading, however, the analysis in this case is sufficient. Direct comparisons can be made between the expected peak ground acceleration (PGA) from local design codes and the values resulting from the analyses. Spanish seismic codes use probabilistic seismic hazard analysis to estimate the expected PGA for a given location, integrating data from historical and instrumental records of seismicity, which typically reference magnitude in terms of the moment magnitude scale (Mw). Spanish seismic codes outline Lanjarón as one of the more seismically susceptible regions in Spain, with an expected PGA value of 0.18 g [9]. At minimum, the wall was able to achieve a value of 0.25 g before collapse. This strength can be considered sufficient to withstand local peak ground accelerations according to the code. A safety factor can be obtained from the ratio of the minimum peak value achieved to the PGA of the area, defined as  $0.25 \text{ g} / 0.18 \text{ g} = 1.39$ . However, the positioning of the castle atop a steep rocky slope and the local aquifers of the surrounding region could destabilize the ground around the structure as suggested in [35].

## 6. Conclusions

In this work, an extensive array of non-destructive tests for sites of cultural significance were presented and performed for the Castle of Lanjarón, Spain. The works explored the application of a Schmidt hammer, Schmidt pendulum hammer, penetrometer, scratch test



device, and a series of direct and indirect sonic tests for wave-based methods to characterize the materials of the structure and their mechanical properties. Dynamic identification was employed to register ambient vibrations and obtain relevant information for the numerical models. Three primary modes were obtained from the dynamic identification process under 15 Hz, being 7.58 Hz, 10.84 Hz, and 14.13 Hz. The fundamental frequencies of vibration were selected manually and further validated using EFDD and SSI-UPC methods. Operational Modal Analysis was used to calibrate and validate a developed homogenized 3D FE model for the west wall of the castle. This was used to assess the seismic behavior of the site using non-linear static analysis in addition to limit analysis. The behavior of the structure was evaluated for safety in accordance with current design codes.

The efficacy of wave-based methods and OMA were highlighted in the study; sonic test results were able to effectively predict the range of the elastic modulus (1.5–2.0 GPa) that resulted from an eigenvector analysis of the model, which considered an initial value of 2.0 GPa but was ultimately successful in calibration with a value of 1.5 GPa. The values obtained from indirect sonic testing were within 3% of the calibrated elastic modulus for the wall, while those from direct testing were around 33% higher. The Modal Assurance Criteria were used to validate the movement of the west wall in the FE model and the experimentally gathered data from OMA to above 85% agreement for all modes presented. The calibrated frequencies for the FE model were 7.52 Hz, 11.01 Hz, and 15.4 Hz, which displayed an average absolute error of 3.8% and maximum absolute error of 9.0%.

Limit analysis and FEA allowed for the evaluation of the seismic behavior of the wall in the out-of-plane direction, the most vulnerable for masonry structures. Common failure patterns were explored, ultimately concluding that the structure could withstand the local peak gravitational accelerations of 0.18 g as defined by the Spanish seismic design codes due to seismic activity. A safety factor of 1.39 was obtained from the ratio of the minimum peak value achieved from non-linear static analysis, 0.25 g, to the PGA of the area, 0.18 g.

The novelty of this case study presents findings that should be observed and considered for all future works on the Castle of Lanjarón. The approach presents a framework that can be adopted for the evaluation of regular and irregular historical masonry structures alike. It is especially relevant for sites that are otherwise difficult to perform common NDTs on. While more traditional NDTs (Schmidt hammer, Schmidt pendulum hammer, penetrometer, and scratch testing instrument) were less successful in their application on the irregularly surfaced masonry, the wave-based tests and modal analysis were extremely successful in their application. These methods were able to wholistically approximate the mechanical behavior of the structure in terms of a homogeneous modulus of elasticity and modal parameters including mode shapes and frequencies of vibration. These wholistic values complemented the use of the macro-modeling approach for masonry and the homogenization assumptions that it entails. These methods can strongly be recommended for the preliminary seismic assessment of a site where resource availability and the applicability of analog methods is difficult.

Further analyses beyond the scope of the paper for the Castle of Lanjarón are recommended. Another non-linear pushover analysis using more accurate boundary conditions for the wall with more computational time could provide a more accurate assessment of the ability of the west wall to resist lateral forces. A non-linear dynamic analysis for the wall or a more general model of the castle could better support the seismic safety conclusions of this paper [5]. Further investigations making use of macro-modeling techniques could utilize deep learning or AI-based imaging models to develop their FE models but may be similarly challenged due to the irregularity of the masonry in the castle [36].

**Supplementary Materials:** The following supporting information can be downloaded at: <https://www.mdpi.com/article/10.3390/app15031518/s1>, Appendix S1: Photogrammetric survey and damage mapping of the Castle of Lanjarón.

**Author Contributions:** H.L.: conceptualization, methodology, validation, formal analysis, investigation, data curation, writing—original draft preparation, and visualization. R.R.: conceptualization, methodology, investigation, data curation, formal analysis, writing—review and editing, and visualization. P.P.: conceptualization, methodology, resources, writing—review and editing, supervision, and project administration. P.B.L.: conceptualization, methodology, resources, writing—review and editing, supervision, project administration, and funding acquisition. All authors have read and agreed to the published version of the manuscript.

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