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Non-Destructive Testing and Synergistic Investigation of a Historic Tower

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Abstract: This paper focuses on the reliability of a synergistic procedure, involving dynamic tests complemented by documentary and architectural research, for the structural assessment of heritage towers. The reliability of the presented method is exemplified in a relevant case study, the Zuccaro's Tower in Mantua. As is known, historic buildings require addressed procedures in order to consider past building transformation and eventual damage/decay, which can affect the masonry properties and, more generally, the structural behavior. The proposed investigation procedure is based on multidisciplinary information (i.e., merging data) from historical and document studies, direct inspections, surveys of geometry and front surfaces, and dynamic tests in operational conditions. The processed information allowed the development and the calibration of a simplified numerical model, useful in driving the seismic/structural assessment of the tower. The results of past investigations, found during the archival research, contributed to the structural evaluation. This paper describes the main research outcomes and the model tuning with a special focus on the key role of the synergy between the applied methods for the assessment of a historic building.

Keywords: historic tower; structural assessment; historical research; modelling; ambient vibration test; non-destructive testing; masonry

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1. Introduction

Ancient masonry towers are a diffused architectonic heritage in many historic centers. The slenderness and the mass of these structures can trigger specific vulnerabilities due to permanent high loads and/or seismic actions. Furthermore, the lack of a specific use involves rare controls. Even direct visual inspections of the external fronts are often not meaningful in detecting damage due to the height of the structure.

The evaluation of the state of preservation of these structures, which sets a priority for the security of historic centers, requires the planning of addressed investigation activities. The assessment of ancient towers is a demanding multidisciplinary and multidimensional task, involving several stages [1,2] and concluded with a structural simulation merging quantitative and qualitative information.

The investigation can start with the geometric or (if possible) topographic survey of the building and the collection of information through the study of historical documents and direct surveys. These steps, supplemented by direct visual inspections, point out the presumable construction technology and building steps, the time evolution of changes and interventions, and the survey of discontinuities, surface decay, and structural damage [1–4].

The activity could be integrated with on-site non-destructive (ND) and/or minor destructive (MD) tests as well as laboratory tests on sampled materials in order to evaluate the properties of the masonry components or to explore specific problems [5–9]. However, the collected information is generally local and often only qualitative. A list of recent applications of MD and ND investigations on historical towers is reported in [10].

Ambient vibration tests (AVTs, even performed with a low number of sensors [1,2] with an appropriate acquisition layout) could complete the former investigations, contributing to the assessment of historic structures with quantitative experimental data: these data provide information on the structural characteristics and can support the development of reliable linear models. Driven by an overall interpretation of the experimental evidence, the merging of the information from historical/document investigations, surveys, and AVTs allow the estimation of the main unknowns in the calibration of numerical models and allow evaluation of the structural condition with a non-invasive and rather quick methodology (see, e.g., [3,4]). A list of recent applications of AVTs to historical towers is reported in [11].

Nevertheless, considering the lack of a well-defined cultural framework and standard regulations, the reliability of the outlined process is strictly dependent on the quality of the collected data in each investigation step and the skills required in information extraction and fusion. Even supported by wide investigations, structural modelling involves unavoidable simplifications and uncertainties, such as those regarding material characteristics, and boundary conditions.

In the Italian Code [12], the evaluation and the seismic risk mitigation of the historic structures can be carried out through a first-level numerical analysis (LV1), using equivalent static analysis. The required structural knowledge is limited to the geometry and the masonry type. Since LV1 is a simplified procedure, the check of the building condition often involves simplified assumptions of the geometry without considering the contribution of the surrounding structures.

Nevertheless, several research papers highlight the limit of the procedure, and, more specifically, the effects of the excessive simplifications, the "regularizations" of the real geometry, and the high contribution of the adjacent structures to the seismic vulnerability, for example, affecting the slenderness [13].The model updating based on Operational Modal Analysis (OMA)—as pointed out in refs. [4,14,15]—supports the detection of the effect of the adjacent structures, as well as eventual damages. A collection of recent applications of OMA-based model updating is reported in [10,11]. In addition, a recent collection of modelling techniques for masonry buildings—not necessarily based on experimental investigations—is reported in [16].

On the one hand, there is the simplified approach (LV1) proposed by the Italian Code [12] with the abovementioned limitations; on the other hand, there is the classical validation of refined numerical models based on both MD tests and AVTs for structural assessment [3,4]. However, there is still a research gap regarding simplified methodologies that are more refined that the LV1 approach and can be applied at the territorial level. Such methods should include only ND procedures to be integrated with historical research. Consequently, the present research has two main goals: (i) the development of a merging-data procedure, including architectonic and historical research, direct surveys, and AVT, for numerical modelling and updating; and (ii) the definition of a methodology for the prompt structural assessment at the territorial level of historic towers involving only ND investigations.

In particular, the proposed methodology (Figure 1) starts with three stages of preliminary investigations to obtain adequate knowledge of the structure: (1) historical and documentary research; (2) geometry survey; and (3) damage survey. Subsequently, AVTs are performed to evaluate the dynamic response of the structure. Lately, a simplified numerical model is developed and the uncertain structural parameters are calibrated using a classical model updating technique. In comparison with other research [17–24], the developed procedure focuses specifically on the merging of architectonic information and the FE model updating to develop simplified numerical models that should be integrated in the context of a digital twin for architectural preservation [25,26].

ND investigations	**Data of interest**
Historical research	• Past interventions and modifications; • Past exceptional events (earthquakes, fires, etc.); • Change in the use of the building.
Geometry survey	• Height, plan dimensions, and wall thicknesses; • Structural details and connections; • Position of the openings.
Damage survey	• Crack pattern and active mechanisms; • Presence of extended areas with material decay.
AVT (simplified)	• Natural frequencies; • Mode shapes.

Structural assessment
• Definition of a simplified FE model;
• Selection of uncertain structural parameters (**x**);
• Model updating and identification of a set of optimal structural parameters (**x**);
• Definition of the optimal model for preliminary structural assessment.

Figure 1. Synergistic investigations for historical towers.

This paper illustrates the application of the proposed procedure—including historical and architectural investigation supplemented by dynamic testing—in the structural evaluation of the Zuccaro's tower (or Torre degli Zuccaro, in Italian) in Mantua (Italy) (Figures 2 and 3). This paper reports the details of the building history and the experimental data collected by means of a non-destructive investigation. The reliable information concerning the geometry, the global direct survey, and the large number of identified vibration modes steered toward the development of a simplified numerical model of the tower for the quantitative evaluation of its structural state of preservation. It is important to remark that: (a) although the proposed model is simplified, it represents with good precision, the geometry and leaning of the tower, the constraints of the surrounding buildings, and the experimental evidences of the key vibration modes; (b) the structural parameters adopted in the elastic numerical model—obtained by minimizing the difference between numerical and experimental frequencies—corresponded well to the results of the available material characterization (according to the archive documents concerning the 90's investigations).

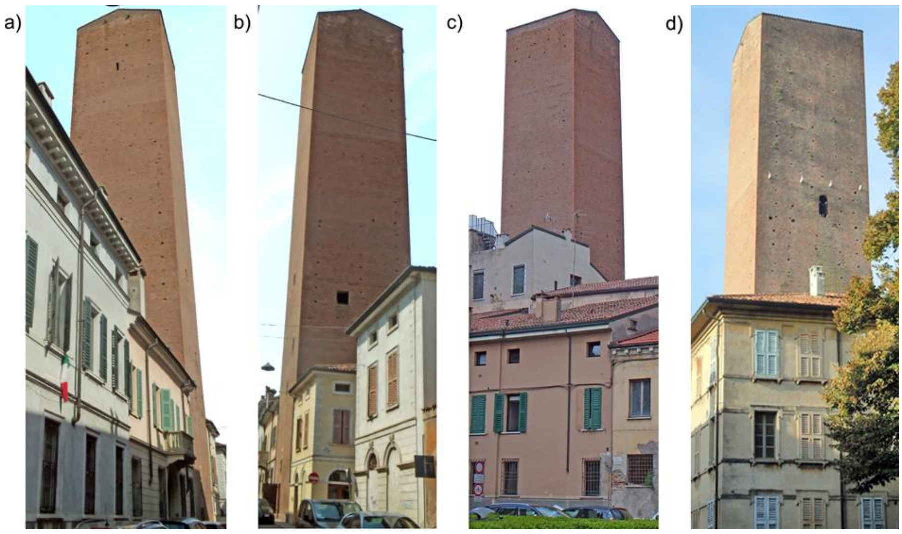


Figure 2. View of the Zuccaro's tower: south-west (**a**), south-east (**b**), south (**c**), and north-east (**d**).

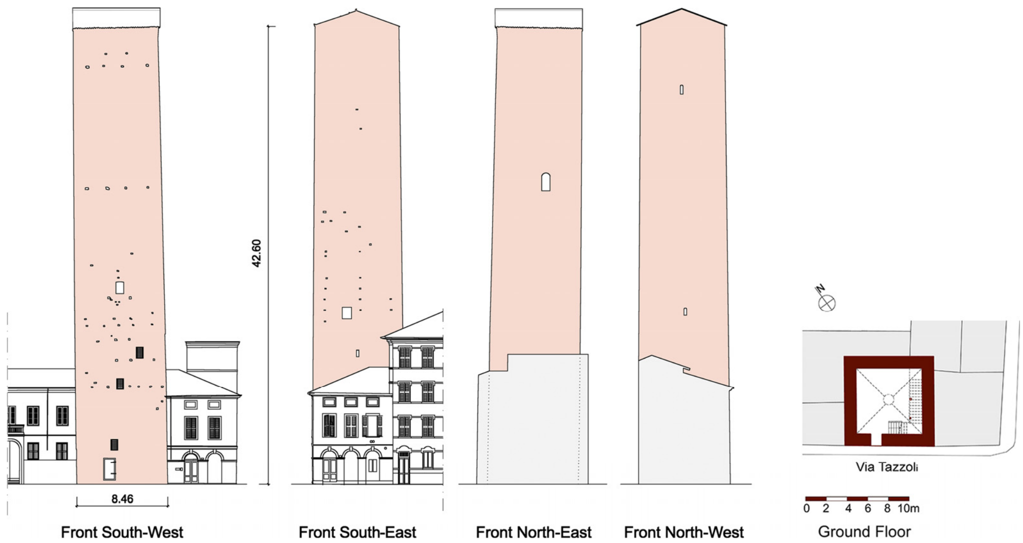


Figure 3. Survey of the elevations (dimensions in m).

The outline of this paper is as follows: Section 2 illustrates the proposed synergistic methodology for the structural assessment of historic towers. Section 3 describes the investigated structure and the performed historical research. In Section 4, the results of the ND investigations are presented. Section 5 illustrates the development and updating of the simplified numerical model, and finally, Section 6 is devoted to discussion and conclusions.

2. The Synergistic Methodology for Structural Assessment of Historic Towers

The proposed synergistic methodology (Figure 1) is developed to support the structural assessment of historic towers at a territorial level, representing an intermediate step between the LV1 evaluation proposed by the Italian Code [12] and a refined structural analysis (e.g., see [27]). Consequently, the on-site investigations are reduced compared to those proposed by the ICOMOS guidelines [28]; in addition, the FE model is simplified and based on 3D beam elements.

The first investigations to be carried out are historical research and visual inspections. Historical research is crucial to reveal possible vulnerabilities and explain existing damages or alterations. In particular, historical research is aimed at identifying past interventions and modifications, and the occurrence of past exceptional events—such as earthquakes or fires—that may have damaged the structure and changes in the use of the building. Historical research is generally performed starting from the available publications on the investigated structure, and then it moves to the archives of the institution owning the building or in charge of its maintenance. The research should focus on sources directly related to the building (direct sources) and, subsequently, on sources related to the urban context (indirect sources). The direct sources are historical drawings, technical reports, and old pictures, while indirect sources are historical cartography, urban plans, and paintings in which the building was represented. An example of the use of indirect sources is given by the identification of architectural alterations (i.e., the transformation and enlargement of the original building): old technical reports are often incomplete or missing, while historical cartography can give qualitative information on the urban modification over the years, regarding the building itself or the adjacent constructions.

A basic geometry survey is necessary and should be performed during the visual inspections. In more detail, the geometry survey should report: (a) the height and the dimensions in plan at each floor of the tower; (b) the thickness of the masonry walls; (c) the position of the openings; (d) the construction details regarding orthogonal wall connections and the floor–wall connections; and (e) the height and length of the constructions surrounding the tower. The collected geometric data are then summarized in 2D drawings, including plans of each floor and external and internal fronts. During the visual inspections, the damage survey is also performed; each crack and decay phenomenon is reported on the 2D drawings of the inner fronts. A correct interpretation and evaluation of damage is essential for an accurate understating of the condition of the building. In cases where large deformation or severe damage patterns are identified (i.e., the typical small and close cracks characterizing the creep phenomena), an immediate report must be made to the competent authority.

The cantilever-like dynamic behavior of masonry towers can be exploited to perform AVTs with a minimal sensor setup. As demonstrated by different authors [2,15], the minimum number of dynamic measuring channels required to effectively identify and roughly classify the vibration modes of towers is 3–4. This simplified sensor setup allows fast and cost-effective dynamic investigations, applicable at a territorial scale. The only disadvantage is the impossibility of fully characterizing the identified mode shapes. In case it is necessary to obtain a complete representation of the mode shapes for a better understanding of the investigated behavior, the test should be performed using a denser sensor network.

Lately, the data collected from the previous investigations have been gathered together to develop a simplified numerical model. The FE model has the following main characteristics: (i) the tower is modeled as a clamped beam with a variable box section, representing the surveyed geometry at each floor; (ii) the mass is distributed along the actual perimeter of the structure and connected with rigid elements to the central beam elements; and (iii) the effect of nearby buildings is simulated with translational springs. An aspect that should be carefully considered when adopting this modeling strategy is the presence of large openings: the box section works adequately only when the opening has a limited dimension; conversely, different schemes should be considered.

The damages identified during the visual inspections are included in the FE model, differentiating between the elastic properties of the damaged and undamaged regions. This difference cannot be local for simplified models and should be adequately tested.

The optimal values of the uncertain model parameters, i.e., the masonry elastic properties and the stiffness of the boundary conditions, are identified using a simple surrogate-based model updating technique [29]. The method involves (i) selection of the uncertain parameters through engineering judgment and sensitivity analysis; (ii) definition of an initial set of values, including their lower and upper bounds, through systematic manual tuning; (iii) creation of an explicit function (i.e., a polynomial system of equation, neglecting the cross terms) to approximate the structural response in terms of experimental natural frequencies; and (iv) minimization of the function defined and identification of the optimal values with a derivative-free optimization algorithm. If the optimal model does not satisfy specific criteria—such as a minimum correlation with the experimental natural frequencies—the procedure restarts from selecting new modeling assumptions; otherwise, the model can be considered calibrated.

In conclusion, it is worth stating that the AVTs should be performed by technical staff with adequate training, and experienced engineers/architects should perform the interpretation of crack patterns and AVT results.

3. The Zuccaro's Tower

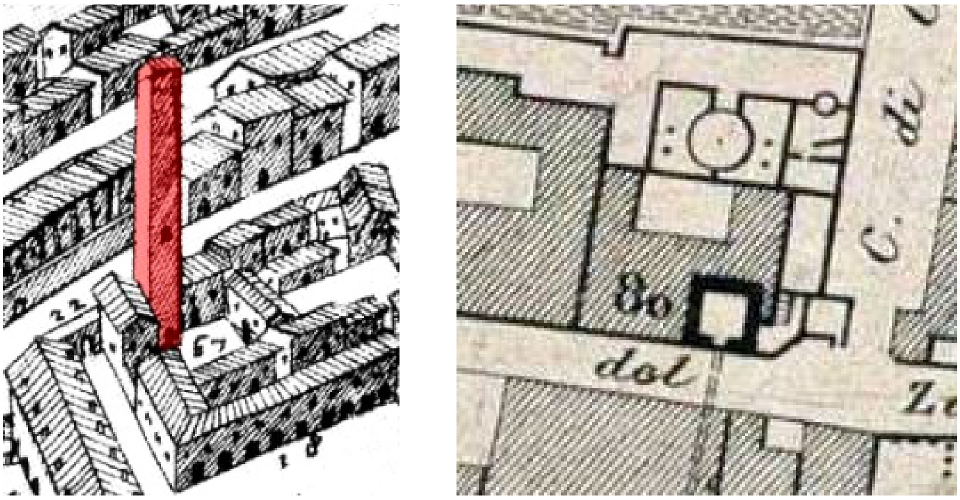
With the close and almost coeval Gabbia Tower (investigated after the 2012 earthquake [2]), the Zuccaro's Tower (Figures 2 and 3) overlooks the historical center of Mantua, listed between the UNESCO World Heritage Site.

3.1. The Tower and Its Transformation

The height of the investigated tower is 43.0 m, about 12 m lower than the tallest historic masonry tower of Mantua, i.e., the Gabbia Tower [2].

The plan of the Tower [30,31] is approximately square (Figure 4) with 1.0 m thick load-bearing walls, built in solid-brick masonry. The structure has nine orders of timber floors and a masonry cross vault—with traces of painting probably datable to the

16th–17th centuries—covers the ground floor. The above eight wooden levels show an alternate orthogonal layout of the supporting beams in order to improve the walls' links.



(**a**) (**b**)

Figure 4. The tower in ancient maps: (**a**) G. Bertazzolo, Mantua Descriptio Urbis, 1628 [32]; (**b**) G. Raineri, Map of the Royal city of Mantua, 1831 [33].

The fronts show a remarkable lack of verticality, with a visible tapering of the plan and being generally out of plumb toward the North corner.

According to the collected historical information, the tower belonged to the Zuccaro family, and its building dated back to the Middle Ages, approximately in the 12th century [30,31]. Most of the transformations occurred in the past are not documented, and archival records that explain the diffused irregularities found during the on-site inspections are missing. On the other hand, direct inspections and the stratigraphic analysis of the wall surfaces reveal that several changes occurred over the centuries but are not clearly datable (Figures 5–7). The north-east front (Figure 2d) shows a sequence of stone corbels, which probably supported a wooden structure that is no longer present. Indoors, the north-west and south-east sides show several masonry corbels placed at a lower level than the beams; this highlights the past rebuilding of the roof structure (Figure 7).

(**a**) (**b**)

Figure 5. The ground floor of the tower. View of the north-east (**a**) and north-west (**b**) fronts with visible infilled openings.

(**a**) (**b**) (**c**)

Figure 6. Several infilled openings are clearly recognizable in the masonry textures of the first floor, on the south-east (**a**), north-east (**b**), and south-west (**c**) fronts.

(**a**) (**b**) (**c**)

Figure 7. The present roof is slightly higher than the former one, as visible in the indoor front through the presence of the corbels supporting the beams (**a**,**b**) and on the external front (**c**).

The construction date is unknown but, according to an inscription, the tower already existed in 1143. Its position—at the limit of the city fortification of the time—and the reduced number of small openings suggest its original military function. The tower had several owners of the main family of the city (di Adelardo, Bonacolsi, Gonzaga, Zuccaro). In 1551–1552, Giulio Ceruti bought the tower [31] that appeared in poor condition: the building was neglected for a long time and was unsafe as it was struck by lightning in 1540. Repair interventions of the structure began in 1553, but they probably did not involve the roof. According to the documents found in the local State Archive, at that time, the tower was used as a salt warehouse. A new roof was built only in 1717, during a general retrofitting of the structure for reuse as a gunpowder magazine [31]. It is likely that the present entrance in the south-east front was open as well. The first view of the tower is in Descriptio Urbis Mantua [32], a general survey of Mantua by G. Bertazzolo (1596). In the second version, which is dated to 1628 (Figure 4a), the tower appears more detailed and conforms to the present geometry, with a wide opening in the north-east front. Furthermore, low buildings surround the tower on two sides; at the north-east side, a wide door opens onto a courtyard. Despite this passage is infilled now, its geometry is still visible by analyzing the changes of the masonry texture and the surface discontinuities (Figure 5).

The subsequent map of G. Rainieri (1831) [33] represents the tower nowadays, surrounded on three sides by buildings (Figure 4b). A staircase appears on the south-east side at the ground floor, probably accessing a door on the first floor, now infilled (Figure 6). The falling of roof tiles was documented since 1820, but only in 1979 was the roofing rebuilt.

During the 1960s, several reports from the Superintendence of the Architectural Heritage informed about the alarming structural condition of the tower, as well as requests for prompt controls and urgent interventions. In 1979, a fire produced severe damages, particularly on the north-east wall and to the timber structures. The damages were repaired in 1982 involving local masonry rebuilding, the reconstruction of the timber roof, staircases and floors, and the opening of new windows.

3.2. The 1990s Investigation

Following the collapse of the Civic Tower of Pavia in 1989, extensive inspections were carried on several Italian Towers, including the Zuccaro's Tower. The on-site survey documented alarming damages on the south corners and an evident vertical crack in the south-west inner elevation (from about 10 m). Then, a wide investigation was planned.

ISMES [34] carried out several tests: inspection of the foundations, the crack pattern survey, coring, tests on sampled masonry components (mortars and bricks), and single and double flat-jacks. Furthermore, the opening of some cracks, the verticality, and the horizontal movements of the walls at two different heights were monitored from 1994 to 1997.

The coring carried out at two levels (2.0 m and 18.6 m) on the south-west wall confirmed that the wall section is made of solid bricks (Figure 8); however, the subsequent inspections by borescope point out that 0.15 m cladding coats the external side. Despite the lack of any evidence in the available historic documentation, the load-bearing walls of ancient towers were frequently built in solid bricks with an inaccurate texture, and a regular masonry leaf of stretcher bricks coated the external (and sometimes the internal) sides. The cladding and the masonry core were built together, connected with courses of header bricks regularly placed along the height of the wall; the distance between the header courses can vary and can be partially hidden by the use of broken bricks. The other courses were partially connected by headers; hence, in many parts, the cladding and the masonry core are bonded only by mortar. Therefore, the cladding—built simultaneously with the masonry section with a regular surface texture and thin mortar joints—had the function of hiding the rough surface underneath.

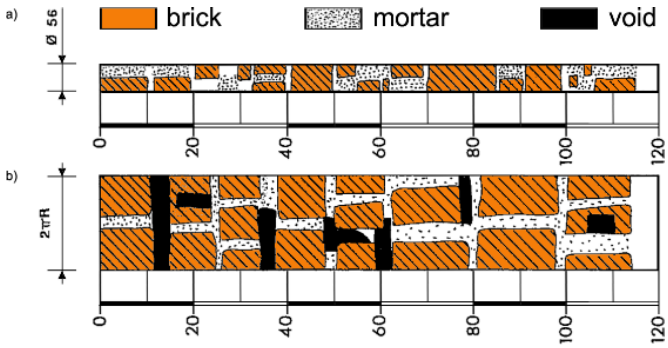


Figure 8. Core (**a**) and reconstruction of the visual inspection by borescope (**b**) carried out at on the external south-west front at 2 m from the ground.

The mortar samples collected in various sites in the tower, are lime based and with calcareous aggregates, sometimes not perfectly washed before the use.

Figure 9 shows the results of three double flat-jack tests carried out at the base (Figure 9a,b) and at +17.10 m (Figure 9c). At the base, the masonry appears stiffer (Figure 9a,b) than that observed at the upper level, as expected with higher loading. According to Table 1, the stress distribution measured by single flat-jack tests appears irregular, with the highest stresses measured around the east corner; this stress concentration could be conceivably explained by the surveyed irregularity of the masonry (e.g., see Figure 5).

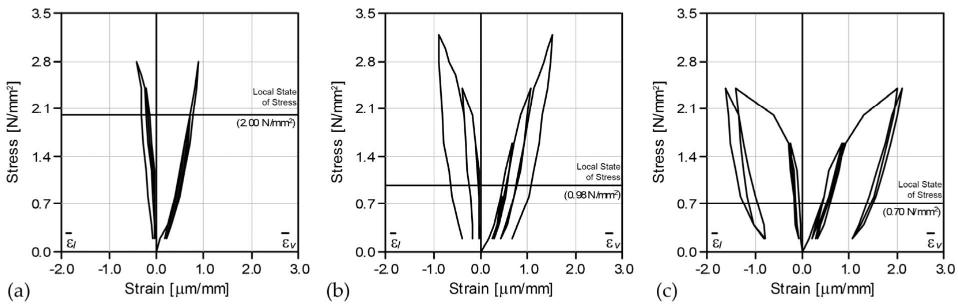


Figure 9. Outcomes of the double flatjack tests carried out on: (**a**) southeast front, internal side at +1.50 m; (**b**) northwest front, internal side at +1.80 m; (**c**) northeast front, external side at +17.10 m [34].

Table 1. Localization and outcomes of the single flat-jack tests [34].

Test	Front	Position	Height (m)	σ (N/mm^2)
1	South-west	Indoor	2.40	0.74
2	North-west	Indoor	1.35	0.70
3	North-east	Indoor	1.20	0.17
4	North-east	Indoor	1.20	1.52
5	South-west	Indoor	1.20	1.83
6	South-west	Outdoor	2.20	0.78
7	North-east	Indoor	16.70	0.49
8	North-west	Indoor	16.60	0.44
9	South-west	Outdoor	17.40	0.53
10	South-east	Outdoor	17.20	0.74
11	North-east	Outdoor	17.10	0.70
12	North-west	Outdoor	17.10	0.75
13	South-east	Indoor	1.50	2.00
14	North-west	Indoor	1.80	0.98

The control of the crack opening showed a slight increase in the relative displacements, despite the prevalence of the thermal effects.

After the experimental investigation, the tower was repaired, the wooden roof and floors were rebuilt, and the more relevant cracks injected with expanding mortars.

4. On-Site Investigations

The investigations herein presented began with accurate document research in the Mantua's Archive, which highlighted the transformation that occurred over time. This documented information was directly compared with the building during the on-site inspections and processed through a stratigraphic survey of the fronts.

AVTs concluded the experimental activity.

4.1. Visual Survey and Inspections

The archive documentation reports a complicated course of transformation and re-use of the tower, periods of neglection, damage, and a series of interventions. The recognition, the surveying, and the filing of these modifications, after in-depth study of the building, is a key factor in the structural assessment. Modification of the construction technology or poor connection between walls can give rise to a weak structural layout and vulnerability.

As no further attention was directed to the tower since the works in the 90s, a careful inspection and survey of all the internal elevations was recently performed. The survey provided information concerning the building geometry and highlighted local irregularities and potential vulnerabilities of the tower. The reconstruction of the building transformations in time helps in the interpretation of the surveyed damage. Since the first on-site survey, the poor state of preservation was evident, with a diffuse surface decay conceivable caused by the lack of maintenance and the natural aging of the materials. Furthermore, the accurate visual inspection and the stratigraphic survey of the internal sides of the walls highlighted the diffused masonry discontinuities and of the cracks, particularly concentrated on the north-east elevation (Figures 10 and 11a) from about 21.0 m. The wall appears superficially covered by a blackish patina—a conceivable effect of the 1979 fire—and locally rebuilt by modern bricks. The south-west and south-east inner fronts (Figures 10 and 11), from the ground level up to about 7.22 m (Figures 5 and 6) exhibit marked changes of the masonry textures and evident cracks. The structural layout of the south-west side, particularly at the first floor, appears very complex, with damage and fragmentary masonry for the several infilled passages toward the close buildings.

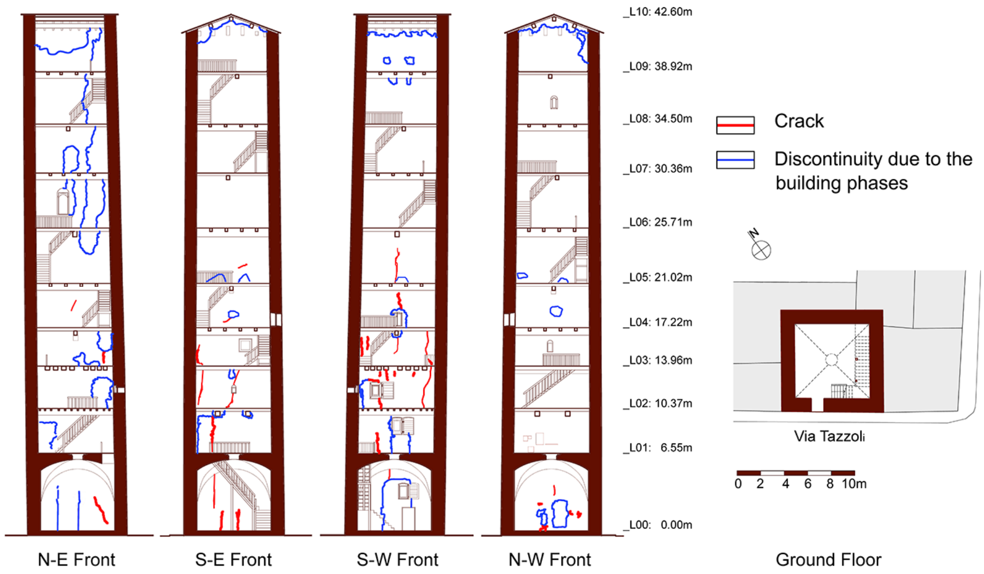


Figure 10. Crack pattern and discontinuities survey of the inner fronts of the tower.

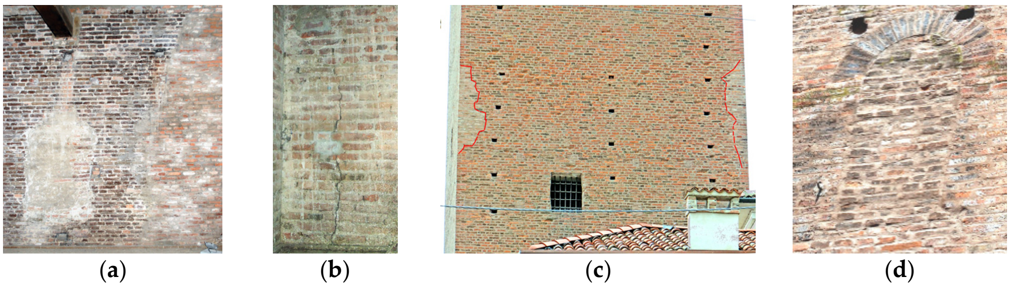


Figure 11. (**a**) Changes in the masonry texture documented on the north-east inner wall at the seventh level; (**b**) Detail of cracks on the inner south-west wall at the third level; (**c**) Local masonry reconstruction on the north-east elevation, and (**d**) an example of former openings, now infilled.

On the ground floor, the cross vault appears damaged with deep and thick cracks. At present, the inspections and the survey of the outer side of the tower is a difficult operation, owing to the adjacent buildings and the narrow streets. Nevertheless, the south-west wall exhibits visible and diffused thick cracks on the outer side, as well as infilled openings and local masonry rebuilding (Figure 11b). The south and east corners show visible traces of local rebuilding (Figure 11a).

4.2. Ambient Vibration Tests and Results

AVTs or continuous dynamic monitoring should be conveniently included in the diagnostic program [2–4,35,36] to complement the available information concerning the tower (document study; geometric, topographic and direct survey; crack pattern mapping; mechanical characterization of the materials) and to merge the collected data to assess the overall state of preservation. These experimental methods are based on the measurement of the dynamic responses induced by ambient excitation and the output-only identification of the modal parameters (i.e., natural frequencies, mode shapes, and damping ratios). AVTs exhibit several advantages, including the fully non-destructive way of testing, the sustainability at moderate costs, and the possibility of using the identified modal parameters to validate FE models [3,4] as well as to detect any alteration of the structural performance [2,36], through long-term monitoring (even temporarily discontinued and reactivated at a later time with different equipment). It is indeed true that the resonant frequencies are also especially sensitive to sources other than structural changes—such as the temperature—but a relatively short period can be enough to understand and remove those "mask" effects.

Two series of AVTs were carried out, on 23–24 October 2016 and on 11–12 December 2017.

During the first test, the opposite corners of the upper floor were instrumented and 12 h of data were continuously collected (from 20:00 of 23 October 2016 to 08:00 of the subsequent day). The testing equipment included two seismometer couples (electro-dynamic velocity transducers) and one recorder.

As is shown in Figure 12a,b, during the second test, the dynamic response was measured at the opposite corners of seven floors. The tests were carried out in 2 days using a multi-channel acquisition system with high-sensitivity accelerometers (10 V/g sensitivity and ±0.50 g peak acceleration). In both AVTs, data were recorded at 200 Hz and stored in separate files of 3600 s. Air temperature and relative humidity were not measured as those data were available from the neighboring weather station.

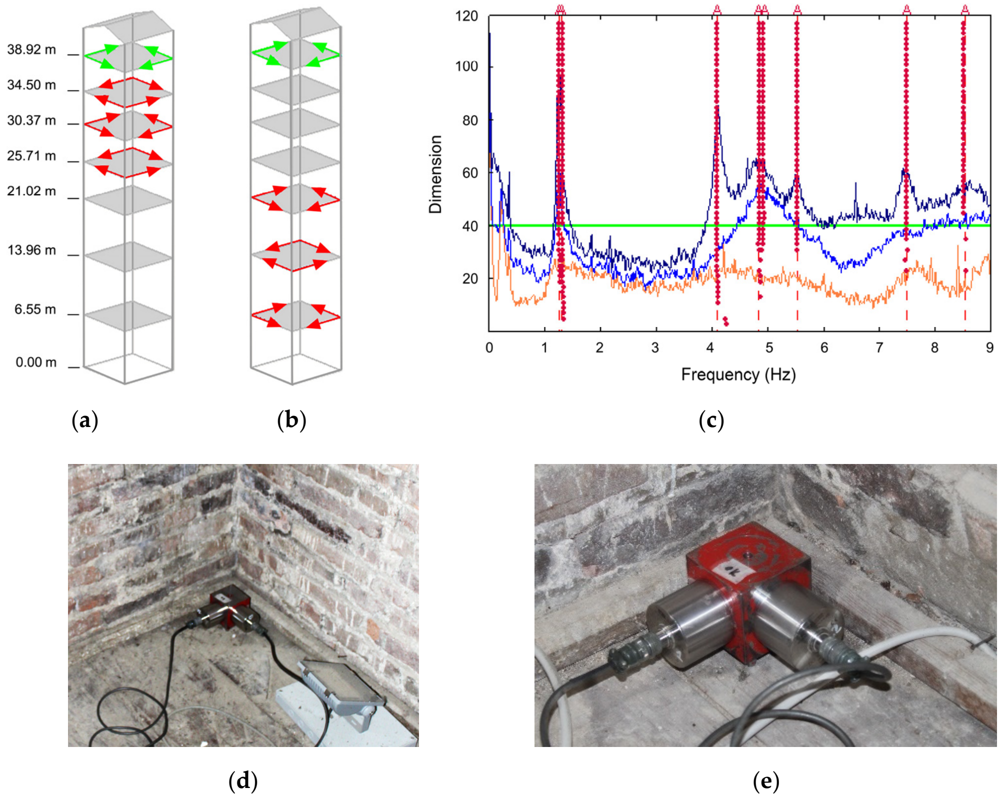


Figure 12. Instrumented cross-sections and arrangement of the sensors during the AVTs of: (**a**,**b**) 11–12 December 2017; (**c**) stabilization diagram and automatic (A) identification of natural frequencies (SSI data); (**d**,**e**) typical positioning of the sensors.

The modal identification was performed by applying the data-driven Stochastic Subspace Identification method (SSI data) [37] available in the commercial software ARTeMIS Extractor 2011 [38]; in addition, the estimated natural frequencies have been verified by inspecting the first singular value (SV) line of the spectral matrix, which is the mode indication function used in the Frequency Domain Decomposition (FDD, [39]) method.

Figure 12c summarizes the results of the second AVT in terms of natural frequencies by illustrating the stabilization diagram (SSI data) and the first SV line of the spectral matrix (FDD). The inspection of Figure 12c highlights that seven normal modes are clearly identified in the frequency range of 0–9 H through the alignments of the stable poles in the stabilization diagram of the SSI method, and those alignments generally correspond to well-defined local maxima in the first SV line of the FDD technique. The corresponding mode shapes are shown in Figure 13. The modal deflections (Figure 13) reveal that the sequence of the identified modes of the tower is as follows: (i) two well defined modes involving dominant bending in the N-E/S-W plane (f_x) were identified at 1.23 (f_{x1}) and 4.78 (f_{x2}) Hz;

(ii) two modes involving dominant bending in the N-W/S-E plane (f_y) were identified at 1.28 (f_{y1}) and 4.97 Hz (f_{y2}); and (iii) one torsion mode (f_t) and two modes involving warping distortion (f_w) of tower cross-sections were identified at 4.10 (f_{t1}), 5.50 (f_{w1}), and 7.47 Hz (f_{w2}).

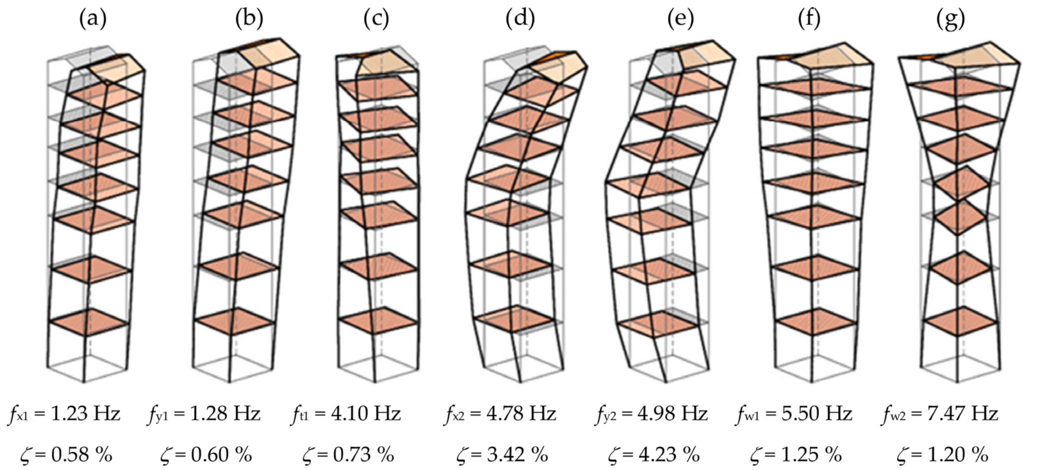


Figure 13. Vibration modes identified from the records collected on 11–12 December 2017 (f frequency and ζ damping related to each mode, x, y flexural; t torsional, w warping mode).

It is important to underline that the identified resonant frequencies and classification of the mode shapes obtained in the first test (by using only four seismometers installed on the ninth floor of the building) are very similar.

Since the ratio between two eigenfrequencies of a vertical cantilever beam depends [40] only on the dimensionless ratio $C = (4\pi)^2 EJ/KH^2$ (between bending and shear stiffness) characterizing the nature of a Timoshenko beam, the experimental knowledge of the ratio f_{x2}/f_{x1} (≈3.89 in the case of Zuccaro's tower) allows to state (Figure 14) that the dynamic behavior of the tower is dominated by both bending and shear.

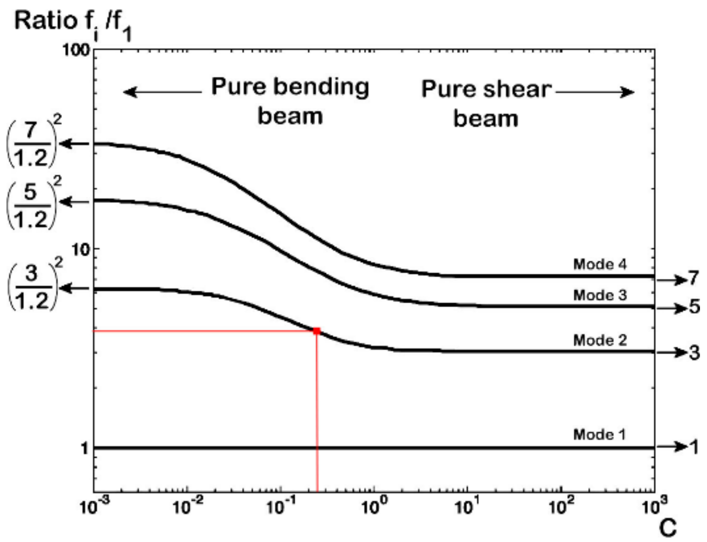


Figure 14. Evolution of frequencies ratio f_i/f_1 in function of parameter C (modified from [40]).

5. Simplified Finite Element Modeling

A simplified modeling of the tower—such as that involved in the so-called LV1 approach recommended by the Italian Code [12] and oriented at estimating the seismic risk of monumental buildings at a territorial scale—was developed before developing a refined FE model; the uncertain inputs were identified by minimizing the difference between the lower five frequencies of the model and the corresponding experimental ones (Figure 13a–e).

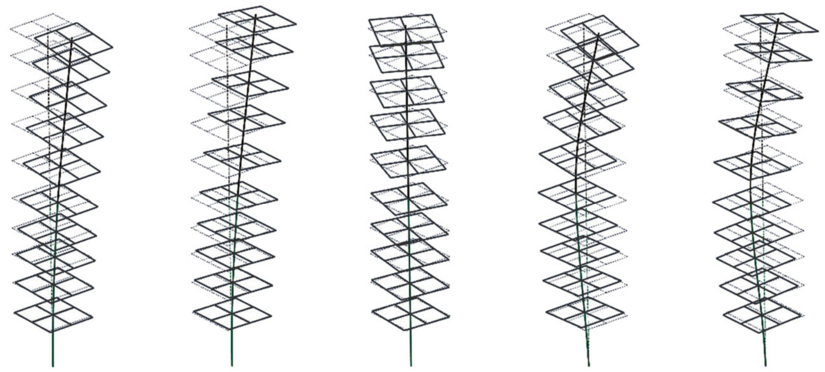
The tower was modeled as a simple Timoshenko beam, with the mass distribution being represented by rigid diaphragms linked to the axis of the beam tower. The uncertain quantities of the numerical model include the following:

- The Young's modulus E_L of the load-bearing walls corresponding to the lower three levels (height $\leq$ 21 m);
- The Young's modulus E_U of the load-bearing walls corresponding to the upper levels (height > 21 m);
- The ratio α between the shear modulus and the Young's modulus;
- The stiffness of the springs ($\sum k_x$, $\sum k_y$) introduced in the model to account for the interaction between the tower and the neighboring buildings.

The mode shapes of the optimal model and the comparison between the experimental and the numerical frequencies of the lower five modes are summarized in Figure 15. Beyond the excellent agreement between numerical and experimental modal parameters (despite the simplified nature of the model), it should be noticed that the values of the optimal parameters are fully consistent with the expectations: for example, the optimal values of the Young's modulus turned out to be

$$E_L = 3.04 \text{ GPa} \quad E_U = 1.60 \text{ GPa}$$

with the average value resulting from the double flat-jack tests in the lower region (Figure 9) being 3.18 GPa.



Mode Id.	B_{y1}	B_{x1}	T_1	B_{y2}	B_{x2}
f_{EXP} (Hz)	1.23	1.28	4.10	4.78	4.98
f_{FEM} (Hz)	1.23	1.26	4.08	4.83	4.95

Figure 15. Lower vibration modes of the optimal FE model.

The optimal estimates of the other updating parameters are as follows:

$$\alpha = 0.36 \qquad \sum k_x = 9.30 \times 10^5 \text{ kN/m} \quad \sum k_y = 1.62 \times 10^5 \text{ kN/m}$$

Of course, the simplified model cannot represent the higher modes exhibiting warping distortion of the cross-sections (as the model consists of stiff diaphragms linked to a central spine); nevertheless, the first couple of bending modes in each main direction of the tower model involve more than 82% of the modal participating mass.

6. Conclusions

Reliable structural assessment procedures at the territorial level are fundamental for the effective preservation of historical towers. Prior research has demonstrated the limitations of the simplified approach (LV1) proposed by the Italian Code [12], particularly regarding the simplifications in geometry and boundary conditions [13]. In this study, a synergistic methodology is proposed to account for ND investigations in developing simplified numerical models, contributing to the development of prompt investigations for territorial-level applications.

The research shows an effective procedure for the structural assessment of historical towers based on an accurate collection of information from several subjects. This paper resumes the outcomes of historical research, visual inspection, ambient vibration tests, and simplified numerical modeling of the tower. The merging of such information drives toward the evaluation of qualitative and quantitative parameters for the development of effective numerical models of the tower: the results of the whole investigation represent a reliable procedure for the structural assessment of a historic building.

Direct inspection and stratigraphic survey of the load-bearing walls clearly show that the tower has several discontinuities and local lack of links, due to the past transformation of the building.

The poor condition of some regions was confirmed by the observed dynamic characteristics and specifically by the warping distortion of the tower cross-sections in several modes.

In conclusion, the reliable information concerning the geometry and of the masonry materials, supplemented with direct survey and the large number of identified vibration modes guided the development of a simplified numerical model for the preliminary evaluation of the seismic vulnerability of the heritage structure. Based on the present knowledge of the building, more refined numerical models will be developed as well.

It is worth underlining the following:

(a) The selected modeling strategy was able to represent the experimental results with excellent accuracy, obtaining an average discrepancy between experimental and numerical natural frequencies equal to 0.4%;
(b) The results of the model updating procedure demonstrated the foremost importance of selecting the appropriate modeling assumptions based on historical research and ND investigations;
(c) The elastic parameters of the models, which were estimated by minimizing the difference between numerical and experimental resonant frequencies, showed an excellent agreement with the results of the masonry tests carried out in the 90s and found during the archival research;
(d) The developed models reproduce with good accuracy the structural layout and the inclination of the tower, the links with the neighboring buildings, and the observed features of the identified modes;
(e) The proposed methodology only involves prompt ND investigations that can be performed in a relatively short time, ensuring its applicability for structural assessment of masonry towers at a territorial level.

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