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Experimental and Numerical Analysis of a Historical Bell Tower

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***Abstract*—**In this paper, a procedure for the evaluation of seismic behavior of slender masonry structures (towers, bell towers, chimneys, minarets, etc.) is presented. The presented procedure is based on a full three-dimensional modal analyses and frequency measurements. As well-known, masonry is a composite material formed by bricks, or stone blocks, and mortar arranged more or less regularly and adopted for many centuries as structural material. Dynamic actions may represent the major risk of collapse of brickworks, and despite the progress achieved so far in science and mechanics; the assessment of their seismic performance remains a challenging task. Then, reliable physical and numerical models are worthy of recommendation. In this paper, attention is paid to the historical bell tower of the Basilica of Santa Maria Gloriosa dei Frari - usually called Frari - one of the greatest churches in Venice, Italy.

***Keywords*—**Bell tower, FEM, masonry, modal analysis, non-destructive testing.

I. Introduction

THE bell tower of Santa Maria Gloriosa dei Frari is the 2nd tallest in Venice, Italy, and since 1396, represents one of the most important artefacts in the city. Completed in 35 years and built following the Venetian tradition, the structure rises over a timber pole foundation and the cores are made of masonry, whereas the belfry consists of a combination of masonry walls and stone columns [1].

During the last seven centuries, several assessments and displacements affected the structure. In particular, during the 15th-20th centuries, a bending movement toward the transept was recorded. This trend was interrupted, during the 20th century, and the bending moment was inverted, ensuring the stability of the structure. The analysis performed during the 20th century demonstrates that the bending movement was induced by the adjacent church structure, hence the decision to separate the bell tower from the church [2]. After the last works, the Italian Ministry for Cultural Heritage installed a monitoring system, in order to indicate the evolution of the structural assessment [3].

Despite some consolidation and separation works, the Frari bell tower remains a complex structure with several boundary conditions induced by the adjacent church.

This research aims to evaluate the influence of such elements on the structural behavior of the bell tower. Hence, a frequency measurements campaign and a free modal analysis have been carried out.

The experimental measurements have been performed with the use of a digital Micromed Tromino tromograph, whereas successive analyses have been carried out using Grilla software [4]. Modal analysis has been performed with Finite Element code Strand7 [5]. The numerical model has been calibrated, for the first vibrating mode, with the same frequency obtained by the measurements. The obtained results have been successively compared and analyzed.

II. Bell Tower Description

As mentioned in the beginning, Basilica of Santa Maria Gloriosa dei Frari is one of the greatest churches in Venice. For its building, Franciscans were granted land to build a church in 1250, but the construction of the present-day church started around 1330 and it was finally consecrated in 1492 [6].

The Latin cross shaped ground plan (Fig. 1) consists of three naves, defined by six pairs of columns and joined by a timber frame system. Its length is 102 m, while the transept is 48 m large. The maximum height is 28 m [7].

From the start of construction, the bell tower was conceived separately from the church, but from the bas-relief by Jacopo de' Barbari (Fig. 2), we can observe that in 1500, the body of the church was already connected to the bell tower [8].

Although many consolidation works have been carried out during the 20th century, the bell tower induced several cracks on the vaults of the transept. Hence, at the beginning of 21st century, several restoration works were performed that led to the structural separation of the bell tower from the transept [9].

Nowadays, the bell tower still preserves the original entrance from the church through the choir area, leaving completely free only the south-east façade. From a structural point of view, it can be considered independent from the vaults of the transept. However, the adjacency of the St. Peter chapel still imposes a boundary condition.

Analyzing the geometry of the bell tower (Fig. 3), the rectangular plan (9.40m x 9.20m) is composed by two concentric walls, joined by the masonry ramp which induces static collaboration between the internal and the external cores. The internal core has more regular geometry than the external one, which is characterized by the presence of asymmetric distribution of the windows and different thickness that decreases with the height. The cores reach a

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height of 50.45 m, whereas the last terrace, on top of the belfry, is situated at 56.6 m. Like almost all Venetian bell towers, the Frari also presents a slight inclination in respect to the vertical axes. Considering that the sloping value, in the NW direction, is lower than 1° respect to the vertical axe, the bell tower, for this research purposes, can be considered vertical.

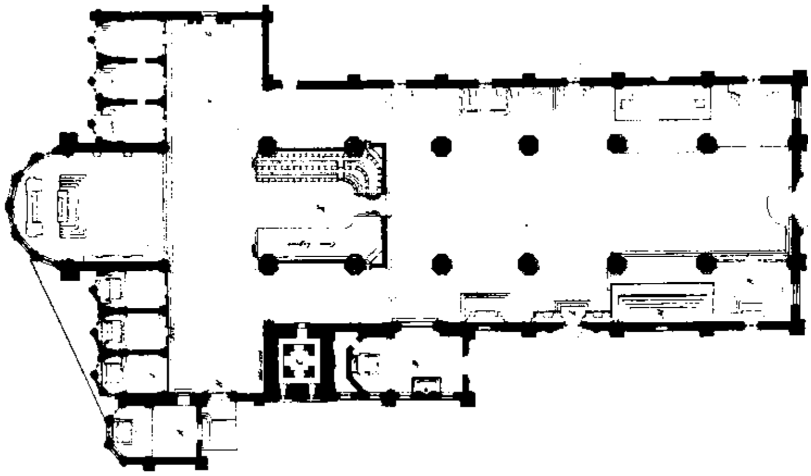


Fig. 1 Frari church, plan

Fig. 2 Jacopo de' Barbari, *Venetie MD*, particular

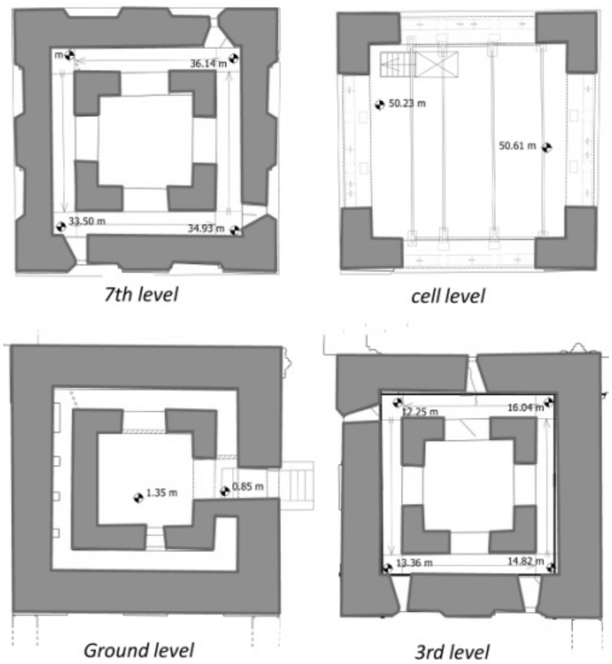


Fig. 3 Frari bell tower, plans

III. Frequency Measurements

In order to study the structural behavior of the bell tower, the first step is the definition of material properties. For this reason, a low cost Operational Modal Analysis (OMA) was performed.

OMA is a non-destructive technique ND technique that aims at identifying the modal properties of a structure. The procedure is based on vibration measurements acquired when the structure is under its operating conditions, without any initial or artificial, excitation. The modal properties of a structure usually include the natural frequencies, damping ratios and mode shapes [10]. Acquisition of the natural frequencies allows for FE model calibration. Hence, material property, i.e. elastic modulus, can be estimated with true approximation.

During an ambient vibration test, the structure can be under a variety of excitation sources, i.e., weak ground motions due to both natural sources (atmospheric and oceanic perturbations) and anthropogenic sources (road traffic). Assuming that stochastic noise is constant during a measurement session, it is possible to use a single, or small number of synchronized, moving, digital tromographs to perform a low-cost OMA. For this research a single digital, compact tromograph, Micromed Tromino, was used to perform non-synchronized vibration measurements.

The tromograph placements are recorded in Fig. 4. The instrument was used for non-synchronized measurements at f_S = 512 Hz, for 16 minutes along the vertical profile defined by windows distribution. The first measurement was performed on the ground floor, the second at the belfry terrace, then the others descending from the top of the structure. The complete survey was performed in less than three hours.

The instrument is equipped with three vibration sensors placed orthogonally one to another; each includes an electrodynamic transducer (velocimeter) and a capacitive transducer (accelerometer). The sampling frequency, f_S, adopted as a reference can be 128 Hz, 256 Hz, 512 Hz or 1024 Hz. The mass of the instrument is 1.1 kg. Although the mass of each sensor is very low (10 grams), the performance of the instrument, whose resonance frequency is 4.5 Hz, is acceptable in the typical frequency range of interest, i.e., from 0.1 Hz to several hundred Hz.

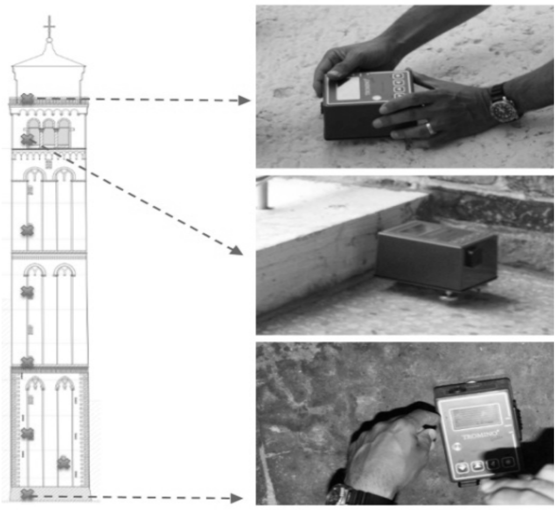


Fig. 4 Tromograph placements

Some of the advantages of using a compact digital tromograph in a frequency measurement campaign are, certainly, related to the small dimensions of the instrument, as well as the possibility to reduce the equipment to a minimum. Many examples of structural analysis performed with the use of Tromino are available in technical literature, and in particular, the analyses performed on bell towers showed reliable results [11].

IV. Data Analyses

The accelerogram in Fig. 5, reports in abscissa time of recorded ambient vibration, while on the vertical axis represents the velocity propagation intensity. Time series represents the velocimetric components (North, East and vertical direction) recorded by means of the high sensitivity sensors. Successive analyses performed with software Grilla, allowed to calculate the amplitude spectra (Fig. 6).

The fast Fourier transform is performed on the acquisitions at the different levels of the building. Consequently, for each building level the amplitude spectra are derived for each velocimetric component.

Plotting the ratios of the velocimetric amplitude spectra acquired at the different levels versus the amplitude spectra at the ground level, can be observed that spikes around 1 Hz and 1.25 Hz should represent first and second mode respectively (Fig. 7). This trend should be related to a bi-modal resonance.

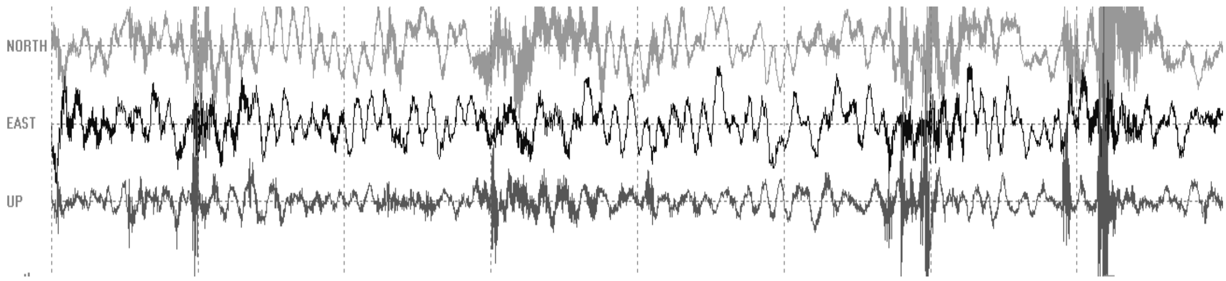


Fig. 5 Velocity propagation intensity

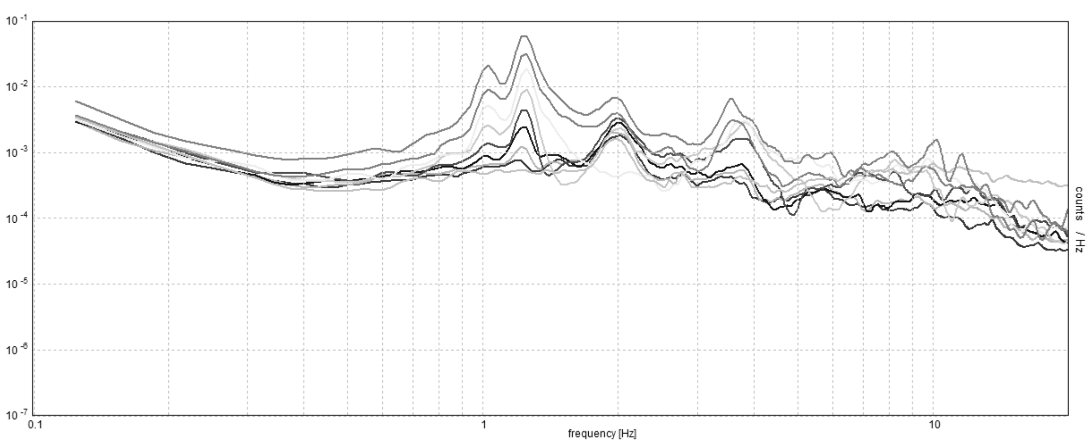


Fig. 6 Amplitude spectra

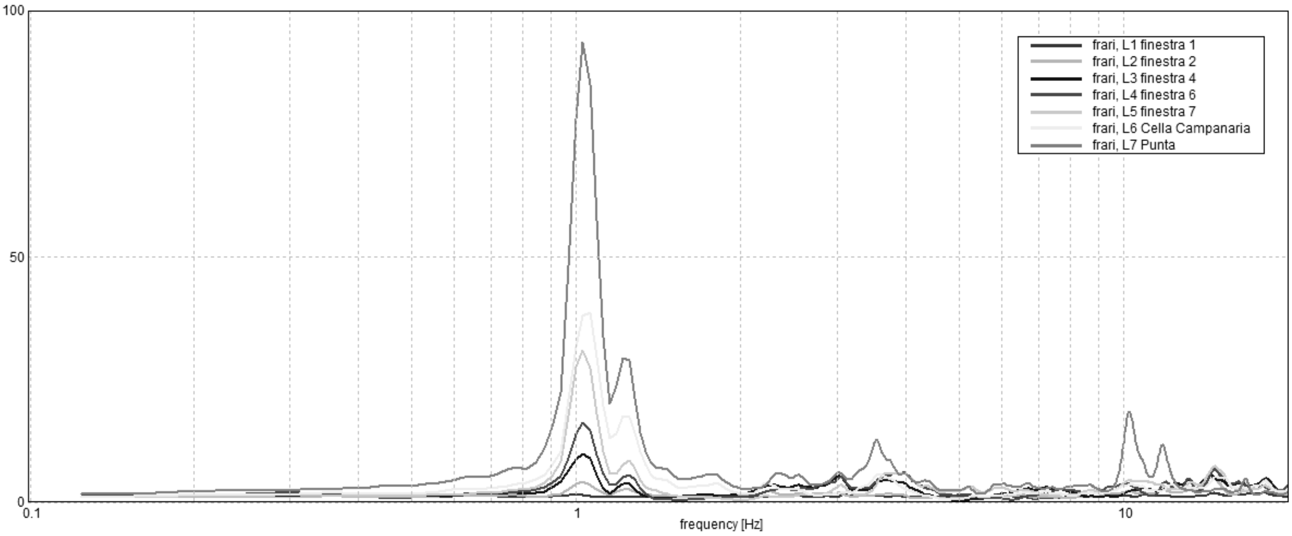
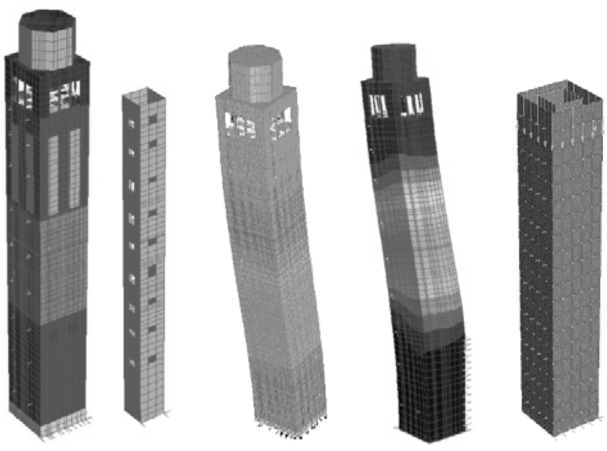


Fig. 7 Frequency peaks

V. FE Models

Some considerations about FE modeling of the Frari historical bell tower have been already presented in a previous paper [12]. The research demonstrated that defined model, which takes into account all the geometric irregularities as well as both inner and outer core, is the most suitable for modal analysis. Starting from the modelling consideration, and taking into account recent works that led to the structural separation of the bell tower from the transept, in this research, the attention has been paid on the boundary conditions and seismic behavior of the bell tower. Hence is proposed an FE model (Fig. 8 (a)) with 2 different boundary conditions: i) Model 1, with only base bonded (Fig. 8 (b)); ii) Model 2, with base and part of external core bonded (Fig. 8 (c)).



(a) (b) (c) (d)

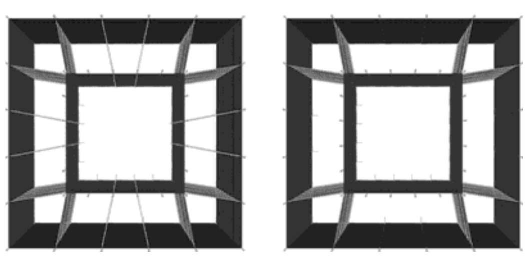
Fig. 8 FE Model: External and internal core , detailed model (a); 1st modal shape at different boundary conditions (b), (c); Simplified model (d)

The model was done with 4112 four nodded plate elements, grouped in 13 properties, and 96 links. Each property reproduce different geometrical thickness of masonry, while the specific weight (γ) and Poisson ratio (v) are considered constant for all the properties (Table I).

TABLE I
MODEL PROPERTIES

Property	Thickness (m)	Element	E (Mpa)	v	γ (Kg/m^3)
1	1.40	external core			
2	1.42				
3	1.40				
4	1.23				
5	1.40				
6	0.90				
7	1.25	inner core	1000	0.2	1800
8	0.74				
9	0.32				
10	1.05	belfry			
11	0.70				
12	1.31				

Link elements have been chosen due the capacity to ensure the structural bond between two cores and distributed connecting each corner node of the inner core with two nodes of the external core (Fig. 9 (b)).



(a) (b)

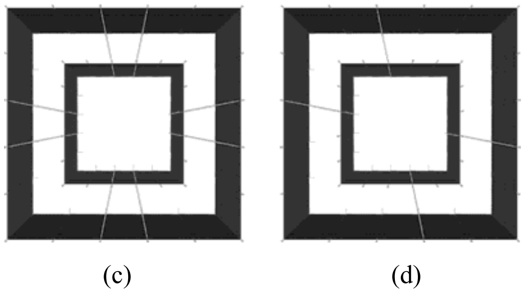


Fig. 9 Link distribution scheme, top view

In order to study the appropriate disposition and number of links, a simplified model (Fig. 8 (d)) reproducing the external and the internal core has been developed. Then, cores, have been joined to each other following schemes described in Fig. 9. Hence modal analysis has been performed, in order to compare the results obtained from each model (Table II) and to evaluate the percentage error between different schemes (Table III, Fig. 10). For these analyses, the scheme in Fig. 9 (a) was considered as reference.

Observing the analyses results, in particular from 1st to 7th mode, can be outlined that distribution scheme proposed in Fig. 9 (b), departs from the reference value for 1.61%, for the 1st and 2nd modes, and less than 1% for superior modes. At the same time, schemes proposed in Figs. 9 (c) and (d), recorded variations from minimum 5.22% to, maximum 35.31%, respect to the reference value. This high value is certainly related to the inadequate link distribution that leads to an independent behavior between two cores. For this reason and, in order to reduce the computational burden, has been decided to apply the scheme proposed in Fig. 9 (b) to the detailed model.

TABLE II
MODAL ANALYSIS, FREQUENCIES (HZ)

Mode	*a*	*b*	*c*	*d*
1	0.870	0.856	0.664	0.603
2	0.871	0.857	0.671	0.659
3	2.985	2.959	2.632	2.572
4	2.986	2.960	2.637	2.606
5	4.055	4.054	3.410	2.623
6	5.941	5.923	4.050	4.050
7	5.953	5.935	5.642	5.620
8	7.138	6.048	5.652	5.632
9	8.897	7.819	7.059	6.317
10	9.114	8.063	7.295	6.622

TABLE III
ERROR EVALUATION (%)

Mode	*b Vs. a*	*c Vs. a*	*d Vs. a*
1	1.61	23.68	30.69
2	1.61	22.96	24.34
3	0.87	11.83	13.84
4	0.87	11.69	12.73
5	0.02	15.91	35.31
6	0.30	31.83	31.83
7	0.30	5.22	5.59
8	15.27	20.82	21.10
9	12.12	20.66	29.00
10	11.53	19.96	27.34

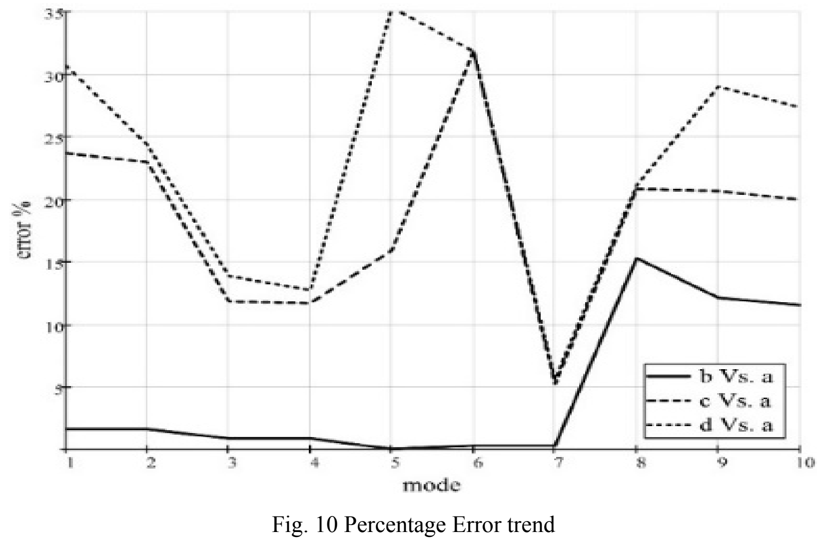


Fig. 10 Percentage Error trend

VI. MODAL ANALYSIS

A. Model 1

As the successive step of the research, a free modal analysis was performed on Model 1 and Model 2. In particular, the modal analysis was performed determining the first 10 vibrating modes and the corresponding frequencies. Moreover, for each vibrating mode, the percentages of participant mass (PFs - Participation Factors) in direction X and Y (Fig. 12) have been evaluated. These values allow to define the main vibration modes and to evaluate the oscillating direction of the model along XY plane. For simplicity, the mass participation in Z axis has been ignored, since the structural behaviour in vertical direction is less significant than the others in plan.

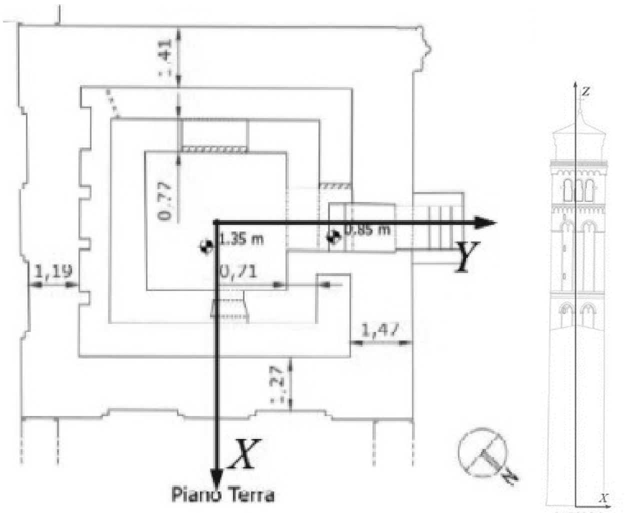


Fig. 11 Modal analysis axes reference

Table IV collects results of modal analysis carried on model with bonded base (Model 1). It is possible to observe that modes 1-2 have comparable frequencies, as same as the participant mass, even if in reverse directions. The result allows to identify a main vibration mode (Mode I), characterized by a sinusoidal deformation with one half-wave, with frequency equal to 0.8 Hz, which mobilizes the sum of the participant masses obtained for each mode (about 70% of total mass). The same consideration can be done for the modes 3 and 4. Together they constitute a main mode, Mode II, characterized by a sinusoidal deformation with one half-wave (Figs. 12 (b) and 13).

TABLE IV
MODEL 1 - MODAL ANALYSIS

Mode	Frequency (Hz)	PF-X (%)	PF-Y (%)
1	0.80	5.21	65.15
2	0.83	65.67	5.25
3	2.70	0.73	12.10
4	2.74	11.80	0.69
5	3.76	0.00	0.00
6	4.72	0.51	4.34
7	4.75	4.34	0.50
8	6.81	2.50	0.41
9	6.82	0.17	2.81
10	6.86	0.67	0.17
Total participant mass		91.63	91.47

As mentioned before, the bell tower is characterized by a non-exactly symmetry of the cross-section referred to X and Y axes, due the presence of asymmetric openings and the inclination of the structure. This aspect causes a structure oscillation that doesn't occur exactly along the X and Y axes (Figs. 12 (a) and (b)). Analogous consideration are valid also for the modes III and IV (Figs. 12 (c) and (d)), that are constituted respectively by modes 6-7 and 8-9. These modes are characterized by sinusoidal deformation with two half-waves (Fig. 13).

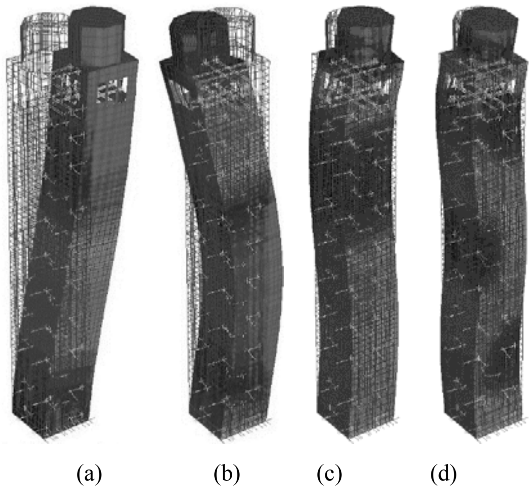


Fig. 12 Modal shapes obtained from Model 1

The following modes involve lower percentages of mass and also they are characterized by more complex sinusoidal deformations with respect to previous modes. A particular consideration should be offered to the mode 5, which doesn't involve the whole structure but just the belfry. Due the presence of internal walls, a torsional mode does not occur among first 10 modes.

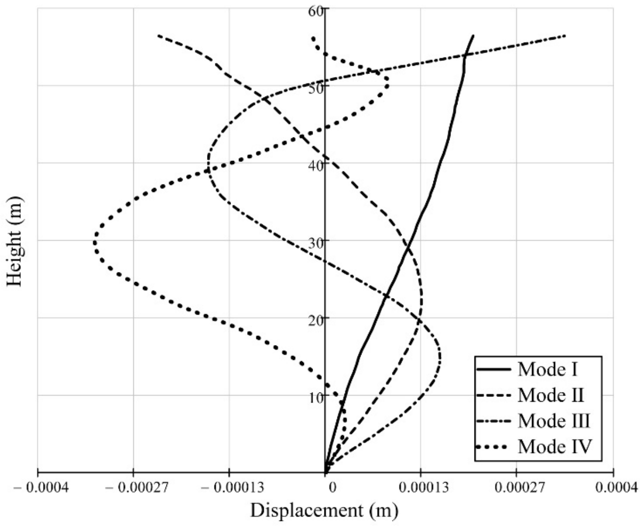


Fig. 13 Displacement evaluation on Model 1

In Fig. 13 is proposed an evaluation of the maximum displacement of the structure due the natural frequencies. From the graph, we can observe that for all main modes, the maximum displacement is lower than 0.4mm.

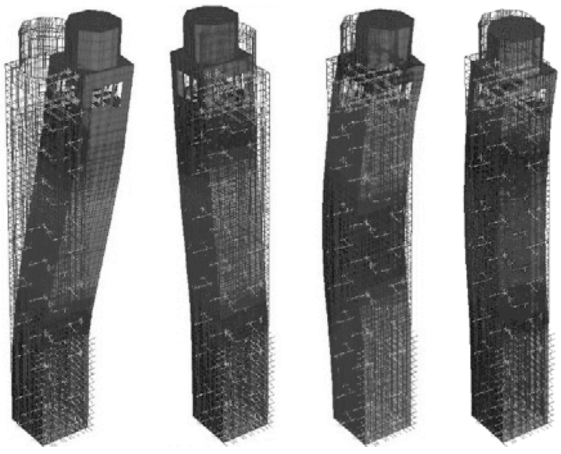
B. Model 2

A second modal analysis has been performed on the model with bounded base and partially bonded external core (Model 2). The results are reported in Table V.

Also in this case can be observed that modes 1 and 2 have comparable frequencies, as same as the participant mass, even if in reverse directions. Again, can be identified a main vibration mode, Mode I (Fig. 14 (a)), characterized by a sinusoidal deformation with one half-wave, with frequency equal to 1.03 Hz, which mobilizes the sum of the participant masses obtained for each mode (about 55% of total mass). In this case, due the boundary conditions the participant masses don't report same values in opposite directions.

TABLE V
MODEL 2 - MODAL ANALYSIS

Mode	Frequency (Hz)	PF-X (%)	PF-Y (%)
1	1.03	3.14	54.11
2	1.08	52.04	3.28
3	3.24	0.05	12.74
4	3.38	8.43	0.06
5	4.29	0.31	0.00
6	5.12	0.01	8.26
7	5.62	4.59	0.01
8	6.56	0.00	1.17
9	7.20	0.00	0.60
10	7.52	0.00	3.95
Total participant mass		68.56	84.18



(a) (b) (c) (d)

Fig. 14 Modal shapes obtained from Model 2

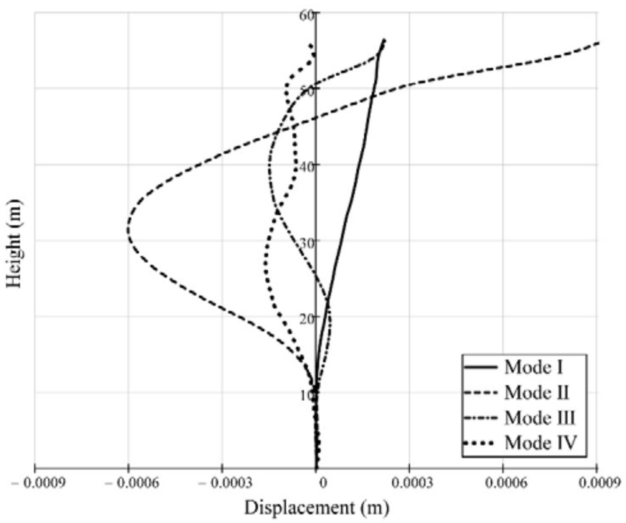


Fig. 15 Displacement evaluation on Model 2

Considering the geometric irregularities, cited before, and the boundary conditions imposed on the base and on the external core, can be observed a structure oscillation that does not occur exactly along the X and Y axes, like in the previous case (Fig. 14). Moreover, as reported in Fig. 15, for all main modes the displacement and oscillations start from an approximate quote of 10m, due the presence of St. Peter's chapel walls. This aspect provokes a maximum oscillation of the structure that reaches the displacement of 1mm.

VII. CONCLUSION

The research performed on the Frari bell tower aimed to check the structural behavior of a historic masonry slender building in complex urban context. In particular, this bell tower has been chosen due its intrinsic complexity. Thanks to the knowledge of structural works that interested the building, it was possible to concentrate the analysis on the influent elements for the dynamic behavior of the bell tower.

Both the analyses performed, numerical and experimental, showed important aspects of the structure. In particular: i) frequency measurements allowed to evaluate the structural behavior under operating conditions without any excitation; ii) modal analyses outlined the importance of the boundary conditions and its influence on the global structural behavior. Moreover, modal analysis allowed to evaluate the modal shapes and maximum displacement of the structure under operating conditions. This analysis, if compared with in situ frequencies acquisition, becomes a valid tool for the structural knowledge and design of structural improvement devices, when necessary. Table VI reassumes the frequency values obtained with different techniques for the 1^{st} vibrating mode.

TABLE VI
FREQUENCY COMPARISON

Mode	Frequency (Hz)	Analysis
1	0.80	Model 1
1	1.03	Model 2
1	1.00	Tromino

Observing the results, it can be underscored that the model with bonded base and partially bonded core (Model 2) evaluates the 1^{st} vibrating mode with the same frequency as the digital tromograf acquired in situ.

A further prospective for the evaluation of the correct modelling approach and the obtained results, should be confirmed by more sophistical numerical analyses and a comparison with recorded crack patterns [13].

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