Dynamic identification of the St. Martin bell-tower of Burano, Venice

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ABSTRACT: The bell-tower of St. Martin in Burano, Venice, presents a remarkable out-of-plumb, reaching the impressive value of 3.50 m at its top, at the height of 53 m. A visible tilt appeared few years after the construction of the tower, increasing throughout the centuries. A major strengthening intervention was carried out in 1964-65, in order to arrest the tilt increase and to “improve” the mechanical characteristics of masonry. To evaluate the present safety state of the tower, a wide investigation campaign took place in 2007, with the application of dynamic identification tests considering ambient excitation. The paper describes the experimental campaign, intended to evaluate the dynamic response of the building. Experimental data were successively considered for the calibration of finite elements models used for the evaluation of the structural behaviour of the bell-tower.

1 HISTORY AND STRUCTURAL INTERVENTIONS

1.1 The St. Martin church and bell tower

The church of St. Martin is located in Burano, in the northern part of the Venetian lagoon, in Italy (Fig. 1a). Its bell tower is the landmark of Burano, overlooking the island with its characteristic and impressive tilting (Fig. 1b).

![a)](image1) ![b)](image2)

Figure 1: a) the island of Burano in the Venetian lagoon and b) the St. Martin bell tower.
The first construction of the church dates back to the X century, being however the present appearance to be related to the XVI c. reconstruction. The first “version” of the bell tower was quite likely erected during such intervention, since no direct historical sources indicate a defined construction date but the bell tower is firstly represented on a 1690 painting, by Antonio Zanchi (1631-1722), where on the background of the main square of Burano are represented both the church and the bell tower (Fig. 2a).

![Antonio Zanchi, 1690, “the miracle of st. Alban”; F. Tironi, 1790, “Prospectus Venetias versus Insulæ, et Urbis Burianì”.

The bell tower took its present shape during the works of 1703 and 1714, according to the project of Venetian architect Andrea Tirali (1660-1737). Historical sources (1738) indicate that soon after the XVIII c. reconstruction of the bell tower the tilting was already visible. Possible restructuring interventions took place during the XVIII c., and in September 1867 the bell tower suffered damage caused by a violent hurricane, heavily striking the belfry and causing the fall of the angel positioned at the top of the tower (Fig. 2b). Repair interventions were completed in 1929, when the angel was replaced by the current iron cross.

The most important documented structural intervention on the bell tower was carried out in 1964-65, following a noticed increase of the tilting of the tower – related to soil differential settlements - and the successive worsening of the damage pattern on the masonry elements. The intervention considered a two steps approach: the first phase consisted in the execution of a substructure composed by “root” piles, aimed at stopping the foundations’ settlements; the second involved the strengthening of the masonry elements by inserting a “grid” of reinforcing bars connected to the masonry bulk by means of cement groutings (Fig. 3). Minor interventions were also carried out, to locally repair the damaged masonry elements. The out-of-plumb of the tower was monitored throughout the duration of the strengthening intervention, and the following year (Fig. 3a).

![Bell tower, monitoring of the out-of-plumb, top displacement vs. time during the works execution; root piles layout, cross section of foundations; masonry stitching by means of steel bars.

Figure 2: a) Antonio Zanchi, 1690, “the miracle of st. Alban”; b) F. Tironi, 1790, “Prospectus Venetias versus Insulæ, et Urbis Burianì”.

Figure 3: a) bell tower, monitoring of the out-of-plumb, top displacement vs. time during the works execution; b) root piles layout, cross section of foundations; c) masonry stitching by means of steel bars.
A particularity of the intervention at the foundations consisted in prestressing the root piles (both in tension and compression) in order to transmit a stabilising moment to the bell tower foundations.

The masonry stitching was carried out by embedding 10 mm diameter steel reinforcing bars in the cement grout cast in the holes (diameter 50 mm) obtained by coring the masonry walls of the bell tower, following the layout reported in Fig. 3c. Further interventions consisted in the construction of a squared hollow section reinforced concrete structure 0,15 cm thick – connected to the masonry walls in their internal face – at the base of the tower, up to a height of approximately 9 m, in order to widen the resisting cross section in the area of major stress concentration.

In 1970, during the intervention validation tests, the tilting of the tower was checked again, noticing a substantial arrest in the tilting process.

2 MORPHOLOGICAL ANALYSIS

2.1 Geometric and dimensional characteristics of the bell-tower

The bell tower is 53 m high, and it is crowned by an iron cross 3 m tall. The cross-section is squared and hollow, with side measuring 6,15 m, and walls thickness of approximately 0,80 m. The bell tower is connected to the church – on its East side – up to a height of 7,20 m (Fig. 4).

The bell tower starts with a base composed by a rusticated masonry. On the south side of the tower, four pairs of openings of reduced dimensions (0,40 x 0,80 m) are aligned along the shaft and vertically spaced of approximately 6 m. Three pilasters on each side – whose overhang is equal to the length of a brick unit - mark the shaft of the tower, ideally projecting on the eight pillars of the belfry (three each side), which support the upper part of the tower. The pillars present a height of 6,5 m, and the floor of the belfry is composed by a pavilion vault. Above the belfry the shaft of the bell tower continues for other 6 m, where an Istria stone (white limestone) gable on each side, projecting for approximately 1 m, clearly mark the shape of the tower. The spiring roof, 10,5 m long, terminates on a egg-shaped Istria stone element, in its turn supporting the iron cross. The internal staircase, as the floors at different levels, vertically spaced of approximately 3,5 m, are composed by wooden elements.

Figure 4: the St. Martin bell tower: a) North, b) West, c) South and d) East elevations; e) vertical cross-section
2.2 Materials properties
The bell tower is composed by a solid brickwork masonry with hydraulic lime mortar. Two main masonry typologies emerged during the investigation phase: from the base of the tower up to approximately 10 m the brick units present a reddish colour, from this height to above a more yellowish one. Several local repair interventions are visible by the different materials used: portions of the rusticated masonry at the base, on the South and West sides, part of the left pilaster on the North side, the right pillar of the South side of the belfry, and others.
Several elements in Istria stone are embedded in the brickwork masonry, with a random pattern, however more concentrated in the northern side and in the spire (at the corners). All of the decorative elements (capitals, masks, etc.) are composed by Istria stone.

2.3 Damage survey
The masonry of the bell tower presents localized decay due to different causes, such as biological colonization and presence of vegetation fostered by soil accumulation. Furthermore, diffuse lack of mortar was noticed, especially in the West and South sides, and several successive intervention of repointing and brick units substitution.
Part of the damages manifested by the masonry elements of the tower can be related to its remarkable out-of-plumb, cause of increased compressive stresses in the South side and shear and tensile forces on the others (Fig. 5a). However, the most worrying cracks may be connected to the oxidation process of the embedded steel bars, in some cases affecting structural elements already in “severe” conditions, as in the case of the pillars of the belfry (Fig. 5b). Other metallic elements, as the tie rods of the spire, manifested a complete oxidation of the cross section with consequent breaking (Fig. 5c).

![Image](image_url)

Figure 5: a) main crack between pilaster and bell tower, East side; b) vertical crack in the S-West pillar of the belfry; c) breaking of the tie-rods of the spire.

3 DYNAMIC TESTS AND MODAL ANALYSIS
3.1 The investigation campaign
A thorough experimental investigation campaign, aimed at defining the principal physical and mechanical characteristics of the structural elements of the St. Martin bell tower, took place in September-November 2007.
On site tests focused on the definition of the mechanical characteristics of masonry and the local state of stress in several positions (flat jack tests), on the localization of the steel bars embedded in masonry (radar tests), on the evaluation of the present tilting of the tower (topographical survey), on the definition of the dynamic characteristics of the tower (operational modal analysis) and on the evaluation of the involved geotechnical aspects. Core samples were extracted and masonry inspected, besides testing some steel bars extracted, to evaluate their state of conservation, that proved to be sound.
In the following paragraphs, the operational modal analysis carried out to evaluate the dynamic behaviour of the bell tower is described.

3.2 Operational modal analysis of the St. Martin bell tower

The dynamic identification of tall structures such as towers, generally gives satisfactory results, because of the structural amplification of motion with the height (Ramos et al. 2006, Gentile et al. 2004). Dynamic investigations carried out in the bell tower (November 2007) considered ambient vibrations, mainly consisting in wind excitation. The aim of the acquisitions was to define the modal parameters (natural frequencies, mode shapes) of the building in order to update a reference FE model, with structural assessment purposes.

Acceleration transducers were connected to the masonry elements of the tower by means of aluminium bases, steadily fixed to the structure with screws (Fig. 6b, c). Sensors were positioned at different levels (Fig. 6a), measuring the accelerations along the two principal directions of the tower. Levels height was chosen by considering the results of the natural frequency analysis of a preliminary FE model, indicating the points of maximum amplification of the bending modes of the structure.

Seven sensors setups were considered in the data acquisition, using a maximum of 6 acceleration transducers. Reference sensors (orthogonal channels 1 and 2) were positioned at the base of the belfry, at a height of approximately 26 m from the ground.

The acquisition system was composed by a compact unit provided with 24-bit digital acquisition modules, connected to piezoelectric single axis acceleration transducers. Once fixed the transducers to the structure in the selected positions, tests consisted in acquiring data over a predetermined period, at a determinate sample rate. A typical acquisition consisted in a record length of 65'536 samples, resulting in an acquisition time of approximately 11 mins at a sample rate of 100 SPS (Samples Per Second). The sampling frequency was considered appropriate, since the significant structural frequencies emerged to be comprised between 0 and 10 Hz.

Figure 6: a) dynamic tests, acquisition layout; b) and c) acceleration transducers in biaxial configuration.

3.3 Extraction of the modal parameters

Obtained data were processed by using the package ARTeMIS (SVS, 2007). The modal pa-
The parameter extraction method selected was the FDD - Frequency Domain Decomposition technique (Brincker et al., 2000) which estimates the modes, with the assumption that the excitation is reasonably random in time and in the physical space of the structure, using a Singular Value Decomposition (SVD) of each of the spectral density matrices. The data series acquired at 100 SPS were processed by a decimation of 2 (Nyquist frequency of 25 Hz), with segment length of 2048 points and 66.67% window overlap. Peaks in the frequency domain related to structural frequencies were selected, and the corresponding mode shapes defined. Cross correlations were established between the spectral density matrices obtained from different sensors, comparing coherence and phase angle between meaningful channels.

Modal parameters were identified quite clearly (Fig. 7), except for some inconsistencies found in the mode shapes of the highest identified modes. From the analysis of results it was possible to estimate the interaction between the tower and the adjacent church, read by the tower as a fixed boundary constraint. The small differences found in the frequencies corresponding to pairs of bending modes may be related to the tilting of the tower, affecting its symmetry, and because of the constraints offered by the church.

Figure 7: St. Martin bell-tower, dynamic tests results, identified modes and average of the normalized singular values of spectral density matrices of all test setups: a) 1\textsuperscript{st} N-S bending, 0.79 Hz; b) 1\textsuperscript{st} E-W bending, 0.84 Hz; c) 2\textsuperscript{nd} N-S bending, 3.47 Hz; d) 2\textsuperscript{nd} E-W bending, 3.63 Hz; e) 1\textsuperscript{st} torsion, 4.85 Hz; f) 3\textsuperscript{rd} bending, 6.91 Hz; g) 2\textsuperscript{nd} torsion, 10.43 Hz.
4 NUMERICAL ANALYSIS

4.1 Numerical model calibration

The finite element model used in a first instance for evaluating the most suitable positions where to position the acceleration transducers was successively calibrated according to the experimental results. The first five identified frequencies were considered for the model calibration purposes (Simonato, 2008). The elastic parameters of the numerical model were modified in order to match the frequencies experimentally determined. The identification problem was simplified by the assumptions of fixed restraint at the base – also considering the foundation substructure (micropiles) provided by the 1964-65 interventions - and a single material property for all of the masonry elements of the tower (similar double-flat jack tests results were found for the different masonry qualities composing the tower). Once assumed the material density (1800 kg/m³), the Young’s modulus was made vary across a reasonable range of values (2600-4000 N/mm²), also according to the double flat jack tests results. The boundary conditions related to the presence of the church were simulated through the application of translational springs at the interface between church and tower, with varying stiffness.

The final comparative results (numerical model vs. experimental data) are reported in Table 1. in Fig. 8 are reported the visualization of the boundary conditions of the bell tower and the mode shapes emerged from the numerical modelling.

<table>
<thead>
<tr>
<th>Mode nr.</th>
<th>FE Frequency (Hz)</th>
<th>EXP Frequency (Hz)</th>
<th>error (%)</th>
<th>Mode description</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>0.789</td>
<td>0.787</td>
<td>0.25</td>
<td>1st N-S bending</td>
</tr>
<tr>
<td>2</td>
<td>0.840</td>
<td>0.842</td>
<td>0.24</td>
<td>1st E-W bending</td>
</tr>
<tr>
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<td>3.473</td>
<td>0.29</td>
<td>2nd N-S bending</td>
</tr>
<tr>
<td>4</td>
<td>3.694</td>
<td>3.625</td>
<td>1.89</td>
<td>2nd E-W bending</td>
</tr>
<tr>
<td>5</td>
<td>4.492</td>
<td>4.846</td>
<td>7.58</td>
<td>1st torsion</td>
</tr>
</tbody>
</table>

Figure 7: St. Martin bell-tower, above: visualization of the constraint offered by the church and below: FE model results, mode shapes individuation: a) 1st N-S bending, 0.79 Hz; b) 1st E-W bending, 0.84 Hz; c) 2nd N-S bending, 3.46 Hz; d) 2nd E-W bending, 3.69 Hz; e) 1st torsion, 4.49 Hz.
5 CONCLUSIONS

The Burano bell tower was studied in order to define its present day safety state, that proved to be satisfactory in a global sense, considering as load conditions - besides evaluating the marked effect of its remarkable tilting on the stress pattern - the earthquake action and the wind force.

As often happens for bell towers, an area at risk emerged to be the belfry, manifesting a marked seismic vulnerability both for the dynamic amplification of motion with the height and the local state of damage found, mainly related to material decay.

An important role for the tuning of suitable numerical models – considered in the successive stages for the structural assessment of the tower – was played by the experimental activities, especially by the dynamic identification. Several natural frequencies and corresponding mode shapes were clearly identified, also by using a limited number of sensors. The non perfectly symmetric behaviour of the tower (e.g. the relative difference between the first two bending frequencies corresponds to 6.3%) may be ascribed to its remarkable tilt and to the presence of the adjacent church which act as a constraint up to approximately 7 m in the E-W direction.

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