ON SITE INVESTIGATION AND MONITORING OF THE "TORRAZZO" OF CREMONA

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SUMMARY

The Torrazzo of Cremona, the highest masonry tower in Europe shows signs of mechanical damage which can proceed toward risky state. The authors have carried out on site and laboratory investigation which ruled out short time risk but suggested the necessity of a long term monitoring to study the progress of the damage.

1. INTRODUCTION

Figure 1: The *Torrazzo* of Cremona

The bell-tower of the Cathedral of Cremona, an historic town near Milan (Italy), was given the nickname "Torrazzo". The tower is situated at the northern side of the facade of the Cathedral and it is connected to it by a Loggia called "Bertazzola". The geometry of the tower is rather complex: (i) a bottom massive part with a square plan of 13m side and 70m high; (ii) an upper, lighter, part more than 40m high with an octagonal plan (2.5m side), with arches and large openings, culminating with a conical masonry roof supported by a ring of 16 twin light columns, called "Ghirlandina". The *Torrazzo* is known as the tallest Medieval Bell-tower in Europe being 112m high (Fig. 1).
The staircase from the ground level up to the *Ghirlandina* was built within the thickness of the walls (approximately 3.3 m thick). Along the staircase, covered with a barrel vault, the external wall is about 1m thick, while the thickness of the internal wall is 0.7 to 1m. The staircase

allows to reach some internal vaulted rooms.
Archive research did not clarify completely the date of construction, that can be located between the 8th and the 13th century. In 1491 the porch of the *Bertazzola* was added connecting the *Torrazzo* with the Cathedral. Maintenance works were carried out starting from the 15th century, mainly concerning the highest part of the tower, damaged by storms and lightening. The last intervention at the *Ghirlandina* was carried out in 1977 by Mrs. M.T. Saracino architect of the Cultural Heritage Office in Milan [1].

2. ON SITE INVESTIGATION

Both the *Ghirlandina* and the lower part of the Tower are still suffering physical and mechanical damages. An experimental investigation was carried out on site in order to state the level of damage of the structure and eventually design the necessary monitoring system [2].

2.1 Geometrical and crack pattern survey.

The first step was the geometrical survey of the tower carried by the DIIAR, Department of the *Politecnico* of Milan. A principal network defining reference points in the horizontal and vertical plan was set up having 21 nodes inside and around the Tower. The vertical and horizontal profiles were determined by rays starting from the network nodes. A photogrammetric survey of the external prospects was also carried out and the prospects were obtained by a Rollei special software, MSR (Fig. 2). The *Ghirlandina* and the *Bertazzola* could not be surveyed and the prospects were successively completed with the topographical external profiles and the missing parts by scanning existing drawings. The survey allowed to find some irregularities of the structure: (i) a 21cm horizontal displacement of the center of the tower in NE direction, calculated from the ground level to the top at 112m, (ii) a non symmetrical reduction of the square plan dimensions from the ground to the top 70m level: 31cm for the NE corner and 66cm for the SW corner, (iii), a slight counter-clockwise rotation of the plan of the *Ghirlandina,* not perfectly aligned on the square part of the Tower.
The presence of a diffused crack pattern particularly on the West, on the East side of the tower and on the *Ghirlandina* can indicate high states of stress due to the dead loads, the temperature variations and/or to a the slight leaning. The highest temperature variations certainly occur between the North and the Southern side. In fact, the Western and Eastern sides have a diffused fissuration with passing-through cracks starting from approximately 20m up. Important cracks appear also between 48 and 60m from the ground level (Fig. 3). Fewer cracks appear in the Northern side (between 27 and 40m and at the NE corner) and in the Southern side (between 14 and 27m.).
The *Ghirlandina* shows the most important cracks on the buttresses and on the brick columns particularly on the SW corner. Also the internal part of the tower, along the staircase and inside the rooms shows a diffused crack pattern with some passing through cracks.

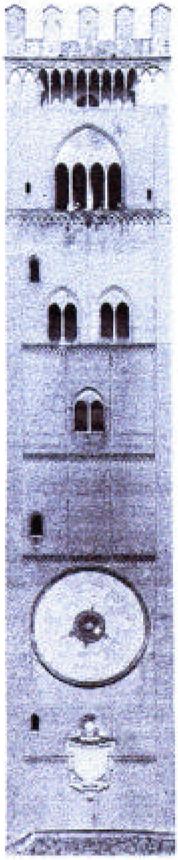
Figure 2: West side Photo-rectification

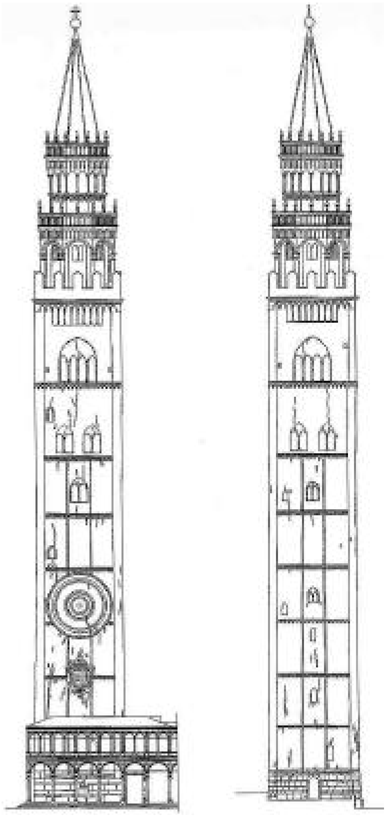

Figure 3: West and East side – crack

2.3 Material characteristics and decay

The inspection of the masonry surface and of the inside of the walls allows to draw the following remarks: (i) the walls are made with solid bricks and no rubble was used for the inner part of the section; (ii) the bricks are regular, with various colours and dimensions: 24?28 ? 10?12 ? 5.5?7cm. (iii) the mortar joints are regular with thickness variable from 1 to 3cm, (iv) an external leaf, one brick thick, with a weak collar joint, is certainly present along the staircases and in the internal rooms and at the level of the Bertazzola, (v) several scaffolding holes externally closed can be seen along the masonry walls.

Together with the geometrical survey an accurate survey of the material decay was carried out. Spalling, scaling and powdering are the most diffused signs of deterioration for the bricks. The decay of the mortar joints is mostly sanding. Also stone structural elements, as columns and pinnacles, are interested by soiling and erosion.

In order to detect the suspected existence of an external cladding, in use during the Middle Age as a false curtain to hide the roughness of the real wall, bricks were sampled from the external wall of the Bertazzola and from the walls of internal rooms. The sampling allowed to find large areas where the leaf (12 cm thick) seems to be detached from the rest of the wall. NDT were applied in order to map the detached areas.

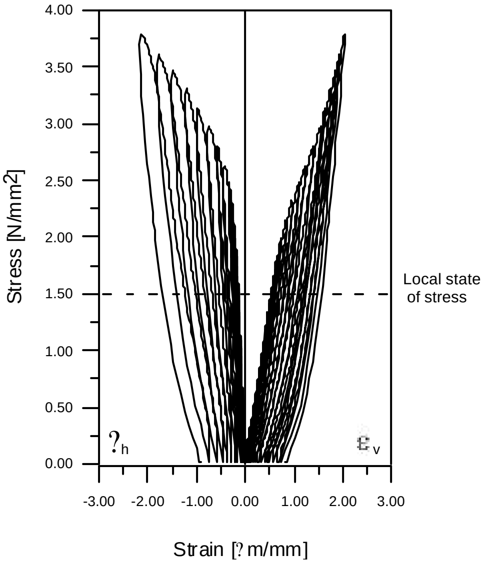

Figure 4: Stress-strain plot and state of stress

2.4 Flat-jack tests

Both single and double flat-jack tests [3] were carried out on the Torrazzo. The single flat-jack test was also used to study the behaviour of the external leaf of the wall. 21 tests were carried out, 19 of which with single flat-jack and 2 with double flat-jack, between 1m and 22m from the ground. It was impossible to carry out tests at higher levels.

The results of the single jack tests show clearly two situations: a stress varying between 0.4 and 0.9 N/mm^2 where a detached leaf was found, a stress varying from 1.01 and 1.81 N/mm^2 where no detatchment was detected, the higher values being found where the staircase reduces the section of the wall from 3.3m to 0.7?1m.

In Figure 4 the stress-strain plots are shown of a double flat-jack performed at 7.2m level, together with the measured stress level.

3. MONITORING SYSTEM

The experimental investigation showed that the tower is suffering of high state of stress due to the dead load, to which are superimposed the effects of wind action and temperature variation [4]. Nevertheless the situation does not seem to ask for a quick intervention.

A long term monitoring of the behaviour of the structure was considered necessary in order to choose the right type of repair. A computer-controlled measurement system has been designed and installed, with two different purposes: analysis of the wind-excited dynamic response of the structure and continuous monitoring of its static conditions [5]. In order to reach these objectives, two different acquisition systems have been designed to work in parallel. The first one is aimed to store dynamic signals (wind velocity and direction, tower accelerations) with 50 Hz sampling frequency, allowing to capture the most significant natural frequencies of the structure. The second one is designed to acquire continuous information on slowly-varying physical quantities (temperature and cracks opening): therefore a low sampling frequency is needed (one sample each 10 minutes). As a consequence, the two measurement systems are totally independent, in terms of electrical connections and acquisition equipment.

In order to measure the instantaneous speed and direction of the wind on the structure, 4 cup anemometers, coupled with a wind vane, have been installed close to the top (85 m), one on each of the four sides of the tower. The section location, the transducers distance from the structure and the fastening system have been carefully chosen, in order to reduce as much as possible the flow-structure interference in the measured signals. The dynamic wind response and the experimental identification of the structure vibration modes, accelerations are measured in 21 different points along the tower. The modal deformations, numerically evaluated, allowed to select the optimal locations of 7 measurement sections, positioned at different elevations (see Fig. 9). In each section, three accelerometers can be fixed: two on the West wall (N-S and E-W measuring axis), and one on the South wall (E-W measuring axis). In this way, by means of proper elaboration, it is possible to distinguish flexural vibration modes in different planes from torsional ones (Fig. 5).

Figure 5: Servoaccelerometer

Figure 6: LVDT

4. ELABORATION OF THE FIRST RESULTS

4.1. Temperatures and cracks opening

As previously mentioned, the masonry temperature is monitored in 22 different locations by means of RTD transducers. The most significant cracks of the structure, at different heights and mostly in internal locations, have also been monitored by means of 15 LVDT displacement transducers, giving continuous static information on the structure status and allowing as well dynamic monitoring in the full frequency range of the tower dynamics (Fig. 6).

A historical data base is being recorded for the 37 channels with a 10 minutes sampling frequency.

Fig. 7 shows an example of the stored historical data for a 7 days period: a general descending temperature trend is superimposed to the day-night cycle and a clear correlation with the crack opening is recognisable.

4.2.Wind and acceleration monitoring

The high-frequency data acquisition system, although being continuously activated, stores blocks of 10 minutes data only according to specific selection criteria, essentially driven by the wind speed intensity. As a consequence, only the most significant samples of wind excited structural dynamics are automatically stored. In the meanwhile, a history data base of the average wind speed and acceleration rms levels (in 10 minutes time base), is continuously stored, synchronous with the corresponding temperature and cracks opening data.

A quite interesting example of the structure excitation by wind is shown in fig.8. Due to a sudden increase of the wind speed from 5 to 22m/s (then stabilising at 15m/s), a very clear forced oscillation pattern of the tower in its first mode (0.437 Hz) is recognisable.

This is a typical example of forced oscillations due to vortex shedding excitation. Similar behaviours were found at different wind speeds, exciting structural modes at different frequencies.

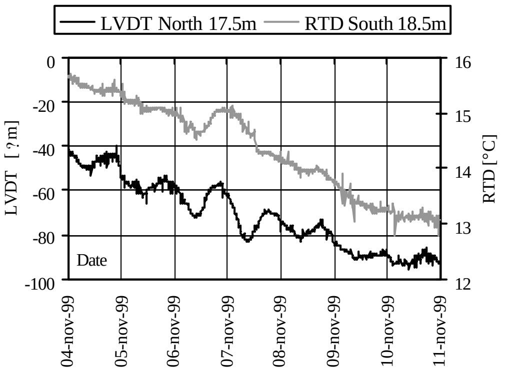

Figure 7: Temperature and crack opening correlation

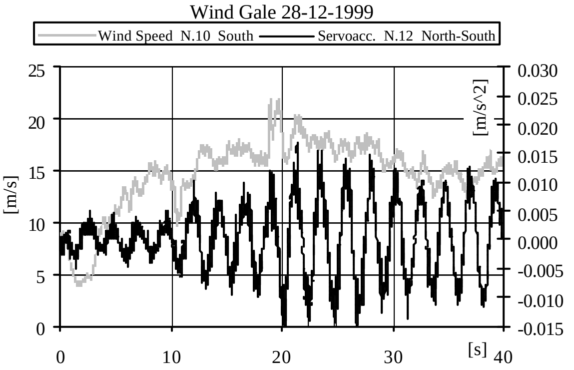

Figure 8: First mode excitation due to vortex shedding

4.3. Tower vibration modes identification

Experimental signals measured by the accelerometers installed along the tower, excited by wind, are the basis to identify structure vibration modes. By means of FFT algorithm, acceleration time histories are converted in frequency domain. A large amount of acceleration spectra are then averaged to avoid noise effect. As a result of these operations, simultaneous FRFs (Frequency Response Functions) of the different acquisition channels are obtained. These FRFs characterise tower response to environment excitation.

Table 1- Modal frequencies [Hz]

mode	experimental	calculated	mode type
1	0.437	0.436	1st flexural
2	1.66	1.57	2nd flexural
3	2.36	2.27	1st torsional
4	2.81	2.72	3rd flexural
5	2.91	3.29	2nd torsional
6	3.26	4.48	4th flexural

4.4. Tower vibration modes: numerical-experimental comparison

The principal vibration modes of the structure experimentally identified have been compared to those obtained throught a numerical model, in terms of frequencies and modal deformations.

The results of this comparison (Tab.1) are very promising: in fact the calculated natural frequencies are very close (especially for the lower vibration modes) to the experimental values.

As an example, Fig. 9 represents the first flexural vibration mode, corresponding to the deformations of fig. 8, They are compared to the corresponding FEM modal shape. The numerical model and the material characteristics can be calibrated against the real dynamic behaviour of the structure. In particular, Young's modulus, Poisson's coefficient, mass distribution and boundary conditions are typical turning parameters that can be adjusted to obtain the best numerical vs. experimental correspondence. The experimentally validated model can be a useful tool to analyse the structure static conditions and to provide useful information for reinforcement and repairing.

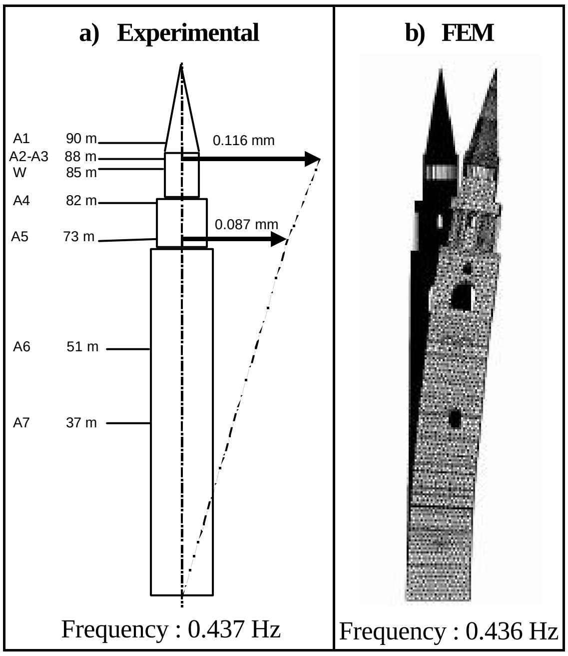

Figure 9: Numerical-experimental comparison (first flexural mode) and measurement sections layout (A = acceleration, W = wind).

5. CONCLUSIONS

The multidisciplinary approach to the work proved to be the fundamental key for the successful development of the research. In fact the historical, laboratory and on site (both static and dynamic) investigation allowed not only to outline the geometry of the tower, its state of damage and fessuration, but also to evaluate the static and dynamic loading due to the structure-environment interaction. The FE model was validated experimentally and showed to be an useful instrument for the analysis of the structural static and dynamic loading conditions. The surveys and numerical model were also a very helpful support for the design of the monitoring system set up. In addition to environment excitation, in the near future inertial forcing will be adopted for a closer identification of the structural behaviour.

6. AKNOWLEDGEMENTS

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