Experimental and numerical analysis of the structural behaviour of St Stefano's bell-tower in Venice

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ABSTRACT: The paper illustrates the results of two series of in situ tests for the analysis of the mechanical behaviour of the masonry structures: flat-jack tests, coring and video camera survey as well as a detailed crack-pattern survey. All the tests were carried out with the aid of climbers, without any kind of scaffolding. An automatic monitoring system was installed, including direct pendulum, crack-gauges, long-base extensometers, clinometers and thermal gauges for measuring air and masonry temperature. The diagrams of the measures recorded during a period of about two years are presented. A FE model was implemented in order to better interpret the results of both the experimental investigations and the monitoring.

1 FOREWORD

The construction of St. Stefano's bell-tower, facing the Malatin canal on the south-west side, started in 1450; the first stage of the construction was interrupted when the bell-tower was 27 m high due to an initial tilting movement of the tower, than in 1550 the second section was built following a different inclination: nevertheless it is not recognizable any discontinuity between the two masonry structures either relating to the structural characteristics or to the architectural language.

The St. Stefano's bell-tower is a single pipe structure with a squared plan (side 7.25 m); the thickness of the masonry at the level of the basement is 1.83 m and it reduces, by following offsets, to 0.95 m measured at the top of the tower. The construction is 61.90 m high and it measures 44 m up to the level of the bell's cell.

The first evidence of a leaning of 0.80 m towards the east-west side, dates 1774; in 1900 it was observed a sensible increasing of the leaning (about 7 mm per year) which leads the deviation from verticality to 1.70 m, measured at the level of the bell's cell.

After the fall of St. Marco's bell-tower the 14th of July 1902, the governmental commission, entrusted to verify the stability of St. Stefano's bell-tower, being seriously worried about the leaning, decided to proceed with the demolition of the second section of the tower and ordered to build up the scaffolding necessary for the works.

Following sharp discussions about St. Stefano's bell-tower, the municipality decided to consult a second technical commission including the engineers Caselli and Antonelli from Turin; considering that the leaning was increasing homogeneously since long time without any recent impulse, the commission decided that it was possible to proceed with the conservation and the consolidation of the bell-tower in spite of the decay of the structure in some parts and partial failures in the foundation structures. Many proposals for the consolidation of the tower were submitted to the Municipality which finally charged, for the design and the direction of the works, the engineers Caselli and Antonelli who proposed both the consolidation and the strengthening of the bell-tower in order to guarantee its stability.

2 THE RESTORATION AND STRENGTHENING INTERVENTION OF 1904

The consolidation works carried out in 1904 are fully illustrated through the drawings and the technical reports produced by the designers, and by the daily reports of the assistant on site who describes in details the execution of the works. With the help of the rich documentation available, it has been possible to reconstruct quite exactly the conditions of the bell-tower at the beginning of the XX century; all the problems concerning the structural conditions of the masonry of the tower and of its foundations have been described in detail as well as the consequent restoration and strengthening interventions.

It was discovered that the primary cause of the decay of the structure was due to the erosion of the mortar

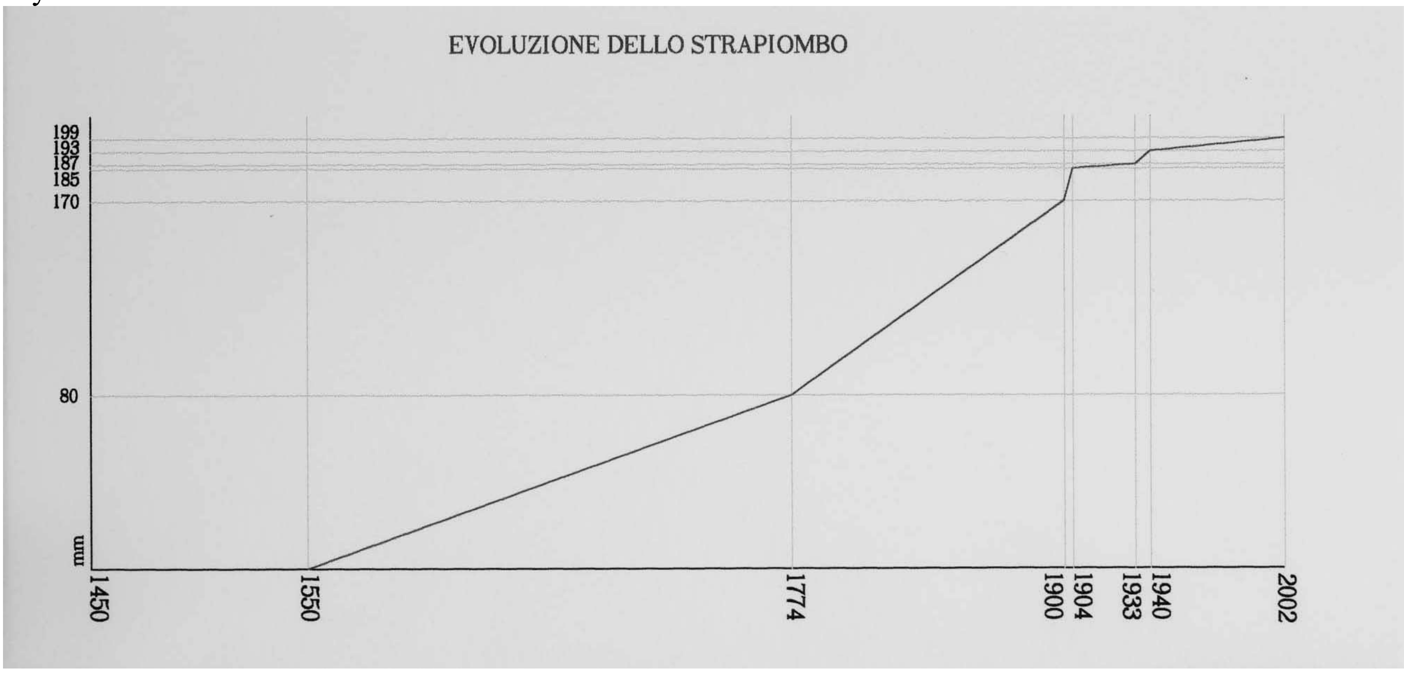

Figure 1. History of the leaning of St Stefano's bell-tower measured al the level of bell's cell (44 m)

between the stone blocks of the foundation, induced by the water of the canal. From the scaffolding, the engineers could verify directly the conditions of the structural masonry that had revealed to be "sound, regular and resistant" excluding some detachments of the bricks forming the pilaster strips and the arches of the cornice from the inner masonry.

The initial strengthening project of the bell-tower aimed at reducing the leaning of the tower through the construction of two big wooden blocks with a cylindrical steel pin fixed in the middle and the realisation of a concrete bed on the bottom of the canal necessary to oppose to the pressure. The excavation works, carried out for this initial project, revealed that the level of the stone foundations of the bell-tower was -2.47 m and that the stone blocks laid directly on top of the poplar piles (diameter 0.15 m, 2 m long); moreover they showed marked erosion of the mortar of the joints and a crack of the foundation block with a differential settlement of the two parts of about 0.34 m.

This delicate situation, and a sensible increasing of the leaning of about 0.15 m during the works, convinced Caselli and Antonelli to modify the project. The two engineers preferred to use the modern techniques instead of the traditional ones and they introduced the new materials (steel and concrete) which they considered more reliable and allowing a quicker execution of the work.

The original idea of decreasing the leaning of the tower was abandoned and it was decided to build up on the side of the canal five buttresses made with bricks and trachyte stone from Monselice. The buttresses, 12 m high, were inserted in the existing masonry of the bell-tower and connected by a system of straight and inverted arches reinforced with steel elements; the load of the new buttresses was discharged on a new foundation system realised with inverted arches and vaults connected by steel chains as those used for the reinforced concrete, which guaranteed both a rapid execution and a reduction of the load on the bottom of the canal.

The intervention on the foundation block was realized in three stages: preliminarily the safety of the foundation structure was guaranteed by inserting a confining steel chain where the block was cracked and grouting a concrete mixture in order to reinforce the mortar and to fill up the empty areas between the stone blocks. Afterwards it was realised a sort of hollow space in brick all around the foundation block in order to prevent the erosion of the joints that Caselli and Antonelli considered the primary cause of the decay of the structure; at last it was realised on the leaning side of the bell-tower a concrete bed where the buttresses discharged their load. On the base of the results of the investigations carried out in the soil, the two engineers thought that the wooden piles of the foundation of the original construction probably didn't reach the really good layer of 'caranto' (a clay layer that constitutes the natural seabed of the Venice lagoon) about one meter lower. Using a special screw system, they built up new concrete piles at the bottom of the canal in order to reach the layer of 'caranto'.

The strengthening of the bell-tower was realised by the new five buttresses (with a section of 1.50 × 0.75 m) and enlarging the foundations with a concrete bed 50 cm thick, laid on top of 124 concrete piles 0.28 m in diameter and 3.00 m long that covered an area of 58 m^2.

At the end of the works, it was installed inside the pipe of the bell-tower a direct pendulum with manual reading which allowed a systematic control of the leaning of the tower.

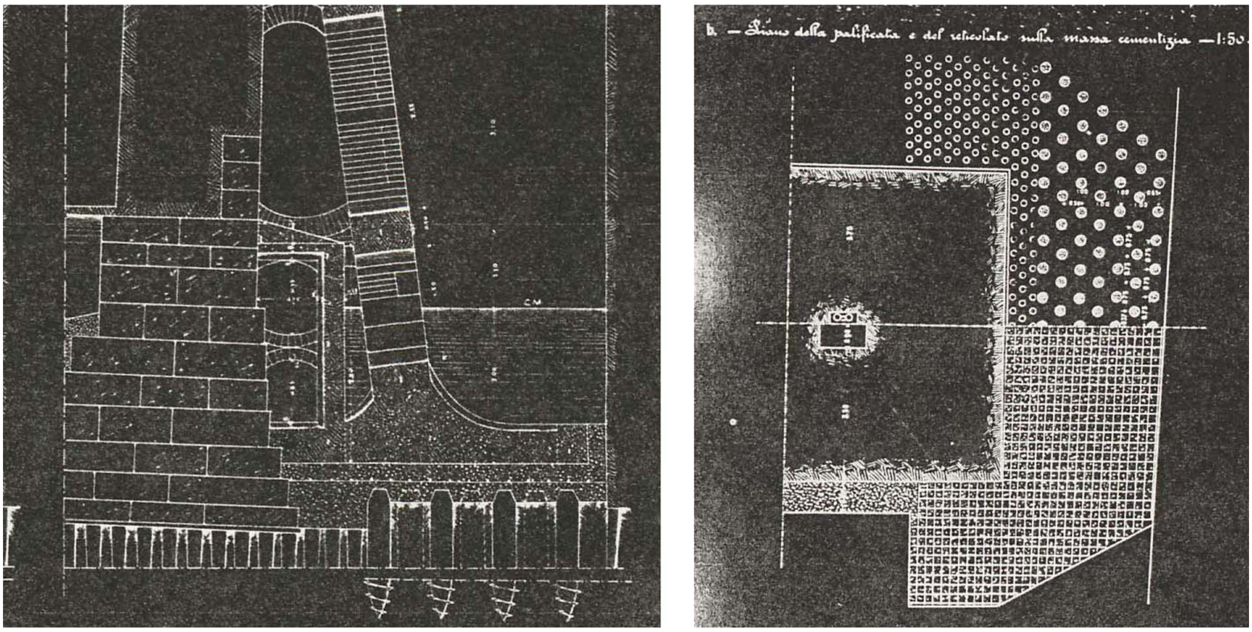

Figure 2. Consolidation intervention carried out in 1904

3 EXPERIMENTAL INVESTIGATIONS

In 1940 the Civil Engineers Office measured a leaning of St. Stefano's bell-tower of 1.93 m towards east-west and 0.12 m towards north-south; since this moment there is no interest anymore for the tower and the leaning is not measured until 1994 when the Municipality of Venice charged the CISE Company to make a first experimental investigation in order to assess the structural conditions of the bell-tower.

At first a geometric survey was carried out together with a detailed crack-pattern survey of the external walls with the aid of climbers. The main cracks were described as well as the damaged and deteriorated zones and the severe oxidation and deformation process of the steel chains which tie the bell-tower.

The composition and the structural characteristics of the masonry was investigated by means of coring and video-camera survey which showed that the inner core of the walls has a composition similar to the outer leaf without the presence of significant voids and cavities.

The state of stress of the bell-tower was measured at different heights by flat-jack test with special attention to the buttresses located on east side, where the deviation from verticality is maximum.

By using two parallel flat-jacks, also the deformability characteristics of the masonry were determined. The masonry of the lower part of the tower, built in the XV century, shows a mean deformability modulus of about 4000 MPa (in the range of stresses 0.4÷0.8 MPa) while the masonry of the upper part of the tower, built in the XVI century, shows a lower modulus with a mean value of 2800 MPa. In the lower part of the tower it was possible to perform a flat-jack test also in the inner core of the XV century masonry; a modulus greater than 2800 MPa was found with a behaviour approximately linear-elastic until the maximum imposed stress level of 2.4 MPa.

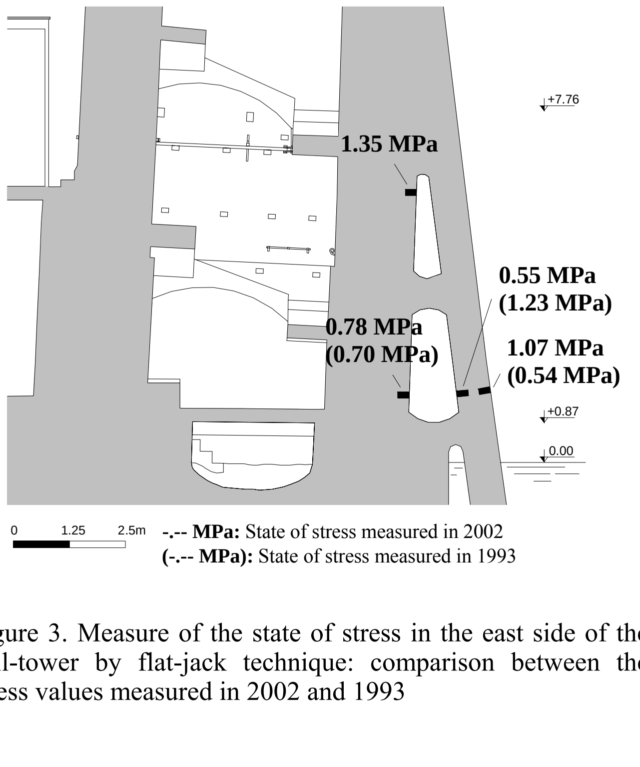

Figure 3. Measure of the state of stress in the east side of the bell-tower by flat-jack technique: comparison between the stress values measured in 2002 and 1993

This detailed analysis of the state of stress and the deformability characteristics of the different parts of the tower was used for constructing the FE model of the structure.
Nine years later, in 2002, a second experimental investigation was carried out in order to examine in detail the state of stress on the east side of the tower and on the buttresses. In the section of Fig. 3 the mean stress values measured on the east side of the tower in 2002 are compared with those determined in 1993.

It can be observed a redistribution of the stresses in the buttresses: on the outer side, the mean stress is increased from 0.54 MPa to 1.07 MPa, while in the inner one a decrease is observed from 1.20 MPa to 0.55 MPa. At the base of the east side of the tower the stress is nearly the same. The redistribution of the stresses in the buttresses could be partially related to the movements of the bell-tower in the period (9 years) between the two testing phases.

4 NUMERICAL MODEL

4.1 *Description of the model*

A 3-D FE model was chosen to evaluate the structural behaviour of St Stefano bell-tower. In particular the modelling aimed at detect the changing, in terms of principal compressive stresses, noticed in the masonry buttresses.

The model includes 17,650 brick and 540 shell plate elements, these last used for the belfry. The average dimension of the solid elements is 0.4 m in the buttresses, 0.5 m in the lower masonry walls of the tower, 1.0 m in the central/upper part (Fig. 4).

The mechanical properties chosen to describe the materials follow the results of previous tests performed on the masonry structures. According to the experimental investigations, three sets of material properties were used. In particular, a Young modulus of 4000 MPa was chosen for the lower part of the bell-tower (from the foundations up to a height of 27 m), while for the upper elements an elasticity modulus of 2800 MPa was considered. A slightly higher elasticity modulus (5000 MPa) was assigned to the buttresses, dating back to 1904. The density of all the sets of material (brick masonry) was kept constant and equal to 1800 kg/m^3.

Firstly, the model was used to assess the mechanical behaviour of the tower in the year 1994, when the first measuring campaign was carried out. According to the survey results, the starting model presents an out of plumb, at the height of 44 m, of 1.98 m in the East direction, and of 0.12 m towards South.

Subsequently, different restraint conditions were imposed at the base of the model to fit the geometry of the bell-tower in 2002, according to the displacement tendency recorded by the direct pendulum. At the height of 44 m, an increase in terms of out of plumb of 10 and 2 mm towards East and South were considered, respectively.

4.2 *Results of the modelling – 1994/2002*

The analyses carried out, for both the models simulating the structural behaviour of the bell-tower in 1994 and 2002, are linear elastic.

The limits of a linear elastic analysis in identifying a masonry structure are well known, see Lourenço (2001); nevertheless this choice follows the consideration that the masonry structure of the tower is mainly subjected to compression.

The maximum compressive stress found in the masonry structures is besides lower than the fixed limit stress reached when performing the double flat jack tests, still presenting the masonry a linear elastic law, see also Bettio et al. (1995).

As load conditions, only the dead load was considered. Figure 5a and 5b respectively show the principal compressive and tensile stresses obtained in the “1994 model”. It is possible to notice that quite all of the elements are subject to compression; the maximum σ_{33} value is localized at the connection between the buttresses and the structure of the bell-tower, and is equal to 1.75 MPa.

The tensile stresses, quite negligible in the model, are considered “acceptable” within the limits of the linear elastic analysis, showing a maximum value, in a very reduced area, of 0.27 MPa.

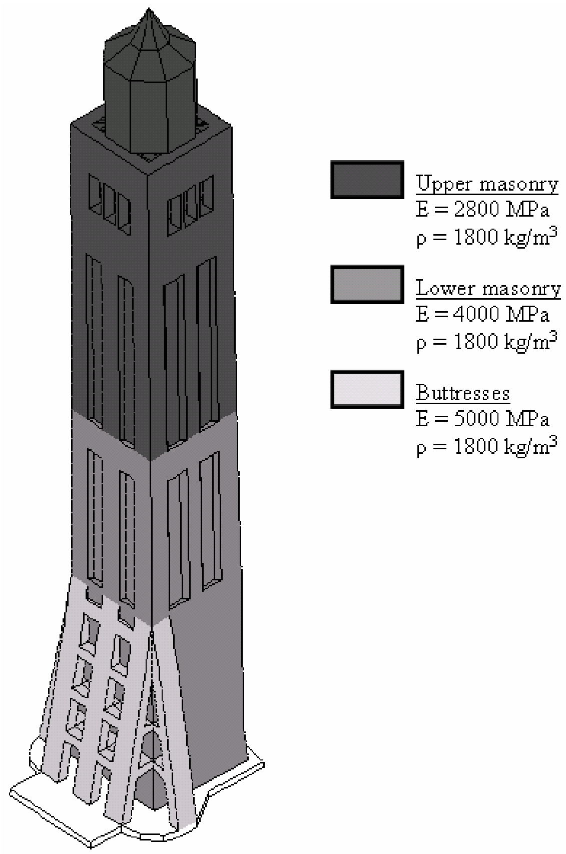

Figure 4. The 3-D model, materials and parameters

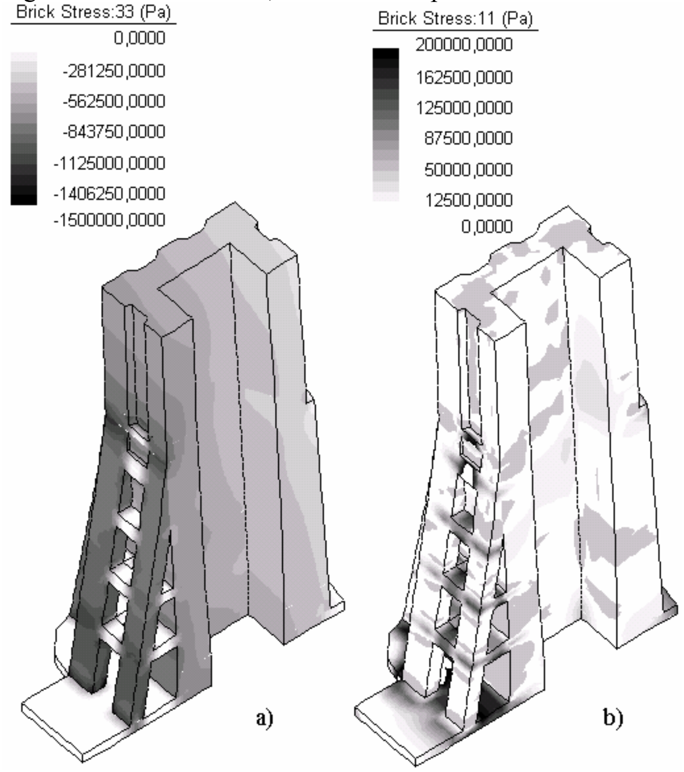

Figure 5. 1994 model results: a) principal compressive stresses; b) principal tensile stresses

The results, in terms of principal compressive stresses, are in the same range as those obtained by several flat jack tests performed on the masonry structures (see Fig. 6a).

As expected, the stress distribution in the two models (1994-2002) is quite the same: the above described small amounts of rigid body rotations/displacements at the top of the bell-tower required to achieve the geometrical configuration of 2002 do not lead the numerical results to match the experimental outcomes.

Hence, noticing that the stress tendency on the buttresses is to increase in the external face and decrease in the internal, inverting the experimental results of 1994, it was supposed that the bell-tower was tilting with the figures of paragraph 5, while the complex of the buttresses was not showing the same rotation. This can also be assumed because of the improved foundation arrangement provided to the buttresses with respect to the foundations of the bell-tower, see Figure 2.

This process, implemented in the model by imposing lower rotations to the buttresses' foundations respect the bell-tower substructure, implies that the first present lower settlements.

Figure 6b shows a comparison between theoretical and experimental distributions of principal compressive stresses due to gravitational load. A fairly good agreement with the experimental results is thus obtained. Some local inconsistencies between the results were found, both in the "1994" and in the "2002" models, likely due to material inhomogeneity, not included in the numerical simulation.

It is worth stressing that the strong assumptions made in order to calibrate the numerical model of 2002 with the experimental results, are not aimed to describe the general state of stress of the structure; on the contrary, the outputs of the calculations are to be intended as a tendency, to assess with further investigations and the results of the static monitoring.

5 MONITORING SYSTEM

As mentioned in paragraph 1, the movements of the bell-tower started just after the construction and were measured in 1770 and 1900. In 1904 a direct pendulum was installed and allowed an easier and more frequent control of the movement of the tower.

In 2001 the total deviation from the verticality of the bell-tower was 1.99 m.

The progressive movement which is involving the bell-tower advised the installation of a structural automatic monitoring system able to check in real time the deformation behaviour of the tower.

The monitoring system, installed in 2001, includes the following instruments (Fig. 7):

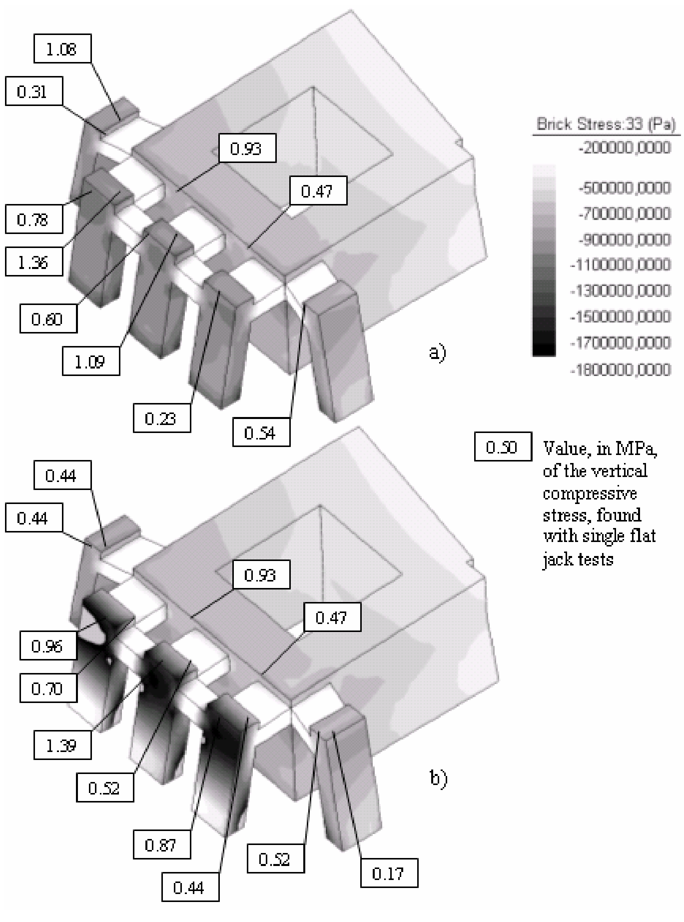

Figure 6. Comparison between the experimental and numerical vertical stresses – a) year 1993; b) year 2002

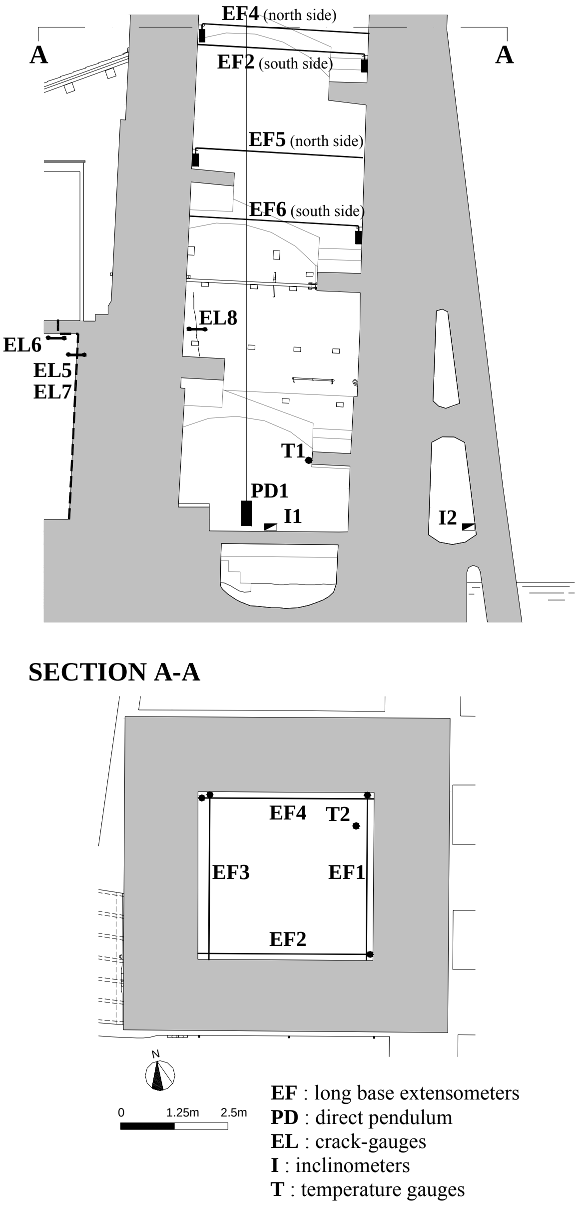

Figure 7. Scheme of installation of the instruments of the structural monitoring system

– direct pendulum equipped with automatic telecoordinometer, for the measure of the absolute horizontal movements of the top of the tower at the level of the bell's cell (44 m);
– 6 long base extensometers which invar wire kept in tension by a weight, installed in two horizontal sections of the tower in order to measure the relative displacements between the opposite walls;
– 8 crack-gauges installed on the main vertical cracks of the tower and on the joint between the tower and the adjoining buildings on west side;
– 2 inclinometers for the measure of tilting at the base of the tower and at the base of the concrete plate which supports the buttresses;
– several thermal-gauges to measure the temperature of the air (internal and external) and inside the masonry at different depths from the outer wall.

All the instruments are connected to an automatic data acquisition and recording system which is equipped with a modem to transfer the data to a remote controller.

The scheme of installation of the main instruments is shown in Figure 7.

It must be pointed out that, in the designing phase of the monitoring system, the principle of "redundancy" was respected in order to have several instruments able to give information on the main movements of the tower.

6 ANALYSIS AND INTERPRETATION OF THE MONITORING SYSTEM DATA

The methodological approach for the analysis of the structural monitoring system data aimed at highlighting the deformation behaviour of the bell-tower by evaluating the trend of deformations.

Direct analysis of the time-history diagrams of each instrument allows an evaluation of the main characteristics of the phenomena involved, such as the main periodicities, the amplitude variations of the signal recorded, correlation with temperature measurements, possible significant strain trends, signal irregularities and interruptions. For example, Figure 8 shows the diagrams of the temperature of the air inside the tower and of the two components of the absolute horizontal movements of the top of the tower measured by the direct pendulum. In the diagrams the average daily values are plotted. This direct analysis is also important for planning numerical elaborations.

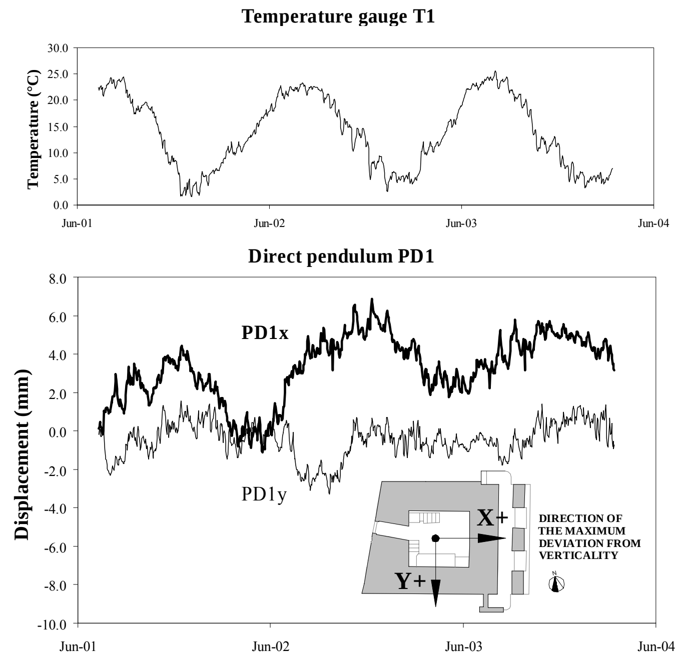

Figure 8. Diagrams of the data of the temperature gauge T1 and of the two components of the absolute horizontal move-

ments of the top of the tower measured by the direct pendulum. The average daily values are reported.

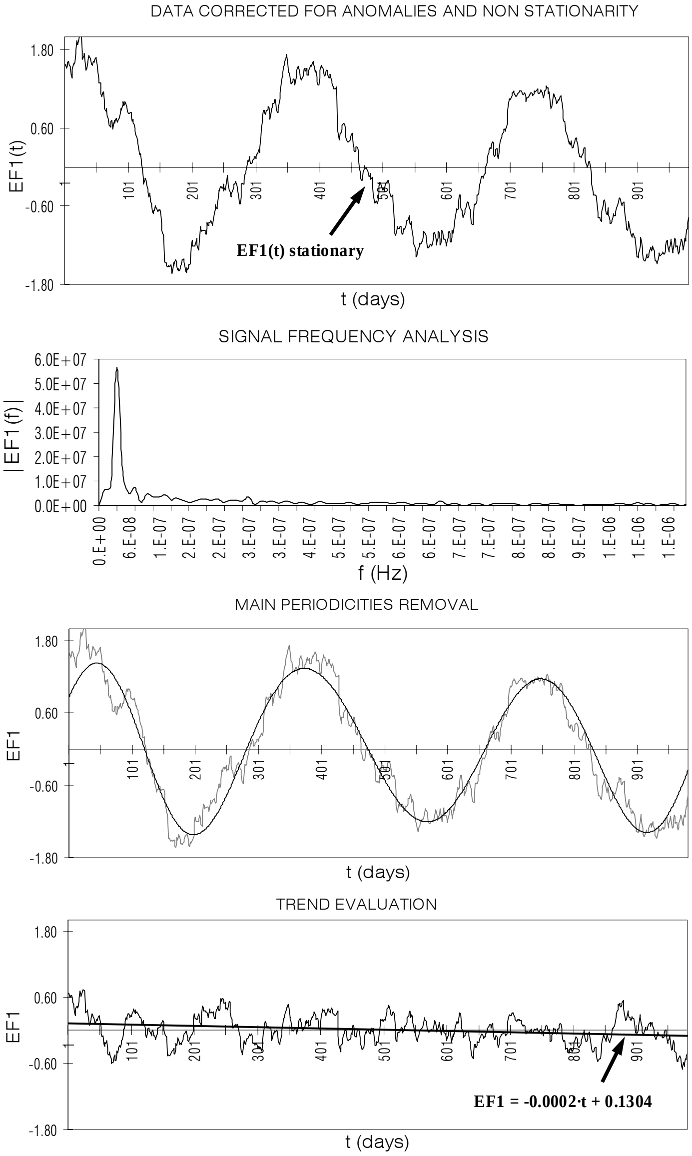

Figure 9. Methodological approach for signal analysis described for the data coming from the long base extensometer EF1. The analysis is implemented by the following steps: elimination of irregularities and non stationarity of the signal; signal frequency analysis; main periodicity removal by filtering the signal (in this case the main periodicity is about 341 days corresponding to a frequency of 3.39E-8 Hz); trend evaluation by "least squares" linear regression. This trend obtained is pertinent to the stationary process: the real trend can be obtained by considering the method used to render signal stationary.

The methodological approach proposed for signal analysis of the static monitoring system data, illustrated in Figure 9, includes signal analysis, evaluation of signal's harmonic component and evaluation of the signal trend.

Data processing can be described according to the following steps:

1) Signal irregularities and interruption: the elimination of signal interruption is an indispensable operation for any kind of data processing for frequency domain analysis. The irregularities are removed by cleaning the signal of anomalies due to accidental impacts against the instruments. These anomalies can cause an interpretation that does not correspond to the real phenomenon.

2) Treatment of signal for non stationarity: the signal analysis proposed is based on the assumption of stationarity of the stochastic process that represent the phenomenon. Thus, the signal must be made stationary.

3) Signal frequency analysis: this operation, consisting in the identification of the main periodicities of the phenomenon, is carried out analyzing the frequency characteristics of the signal by a transformation from time domain to that of frequency.

4) Main periodicities removal: this step involves filtering the signal in the frequency domain. This operation is analogous to treating the signal in time-domain by subtracting a sinusoid, containing the main period of the signal after having treated it for non stationarity.

5) Deformation trend estimation from filtered signal: the removal of main periodicities of the signal permits the use of a "least squares" linear regression to evaluate the annual deformation trend of the signal. This trend is pertinent to the stationary process: the real trend can be obtained by considering the method used to render signal stationary.

In Figure 10, the results obtained for the direct pendulum (component in direction of maximum deviation from verticality) are shown.

The values of the annual trend estimated for the static monitoring system signals are presented in Table 1. The same analysis implemented for the temperature gauge T1 and T2, measuring the air temperature inside the bell-tower, gave as a result a trend of the temperature close to zero.

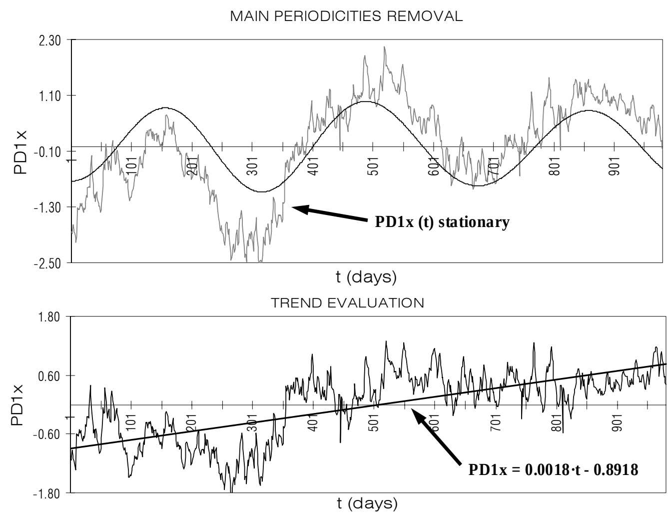

Figure 10. Signal analysis implemented for the direct pendulum (component in direction of maximum deviation from verticality). The trend of 0.0018 1/day obtained is pertinent to the stationary process: the real trend can be obtained by considering the method used to render signal stationary; for the X-component of the pendulum the real trend is 1.076 mm/year.

Table 1. Annual deformation trends.

measurement device	number	mm/year
direct pendulum	PD1x	1.076 **
	PD1y	0.218 **
long base extensometer	EF1	- 0.022
	EF2	- 0.044
	EF3	- 0.014
	EF4	- 0.022
	EF5	- 0.021
	EF6	- 0.021
crack-gauges	EL1	0.013
	EL2	0.022
	EL3	0.008
	EL4	0.010
	EL5	0.063**
	EL6	0.235**
	EL7	0.139**
	EL8	0.005
inclinometers	I1	- 0.011*,**
	I2	- 0.010*,**

* mrad/year.
** trends which are meaningful, according to the transducers' accuracy.
For the long base extensometer a positive (+) trend means a increase of the distance between the vertical walls; for the crack gauges a positive (+) trend means an opening of the crack; for the inclinometers a positive (+) trend means a tilting toward the maximum deviation from verticality; for the pendulum a positive (+) trend means a increase of horizontal movement of the top of the bell-tower in the directions of the maximum deviation from verticality, in according to the scheme of Figure 3.

This means that a correction of the deformations trend of the tower, due to the influence of the temperature trend, was not needed.

In Table 1 are evidenced the annual trends which are meaningful according to the accuracy of the transducers of the instruments.

The analysis of the calculated trends clearly shows that the bell-tower is moving as a rigid body tilting at the base in the direction where the deviation of verticality is maximum:

- the absolute horizontal movements of the top of the bell-tower, measured by direct pendulum, shows a significant trend which is about 1.08 mm/year in the direction of maximum deviation from verticality (X-component) and 0.22 mm/year in the Y-component.
- all the long-base extensometers EF1÷EF6 and the crack-gauge EL1÷EL4 and EL8, installed on the bell-tower leafs, do not show significant deformation trends;
- the crack-gauges EL5, EL6 installed on the joint between the tower and the vertical walls of adjacent building, show a positive trend of opening of the joint with an average value of 0.10 mm/year;
- the inclinometers show a trend of the rotation of about 0.011 mrad/year toward the direction of maximum deviation from verticality.

7 CONCLUSIONS

The analysis of the data obtained by the monitoring system during the first three years of observation clearly shows that the bell-tower is moving like a rigid body which is tilting at the base. The moving of the cracks as well as the relative movements of the opposite faces of the tower is only influenced by temperature changes and no significant trend due to other causes is observed.

The absence of significant trends of the transversal deformation of the tower means that the steel chains which tie the tower are able to guarantee a sufficient confining effect.

It must be pointed out that after the installation of the monitoring system new chains made of high-resistance cables were installed by climbers in order to guarantee a reliable confining effect able to eliminate the transversal deformation of the tower.

On the contrary, the absolute horizontal movements of the tower, measured by direct pendulum, show a significant trend which is about 1.08 mm/year in the direction of maximum deviation from verticality. This value is a little bit higher than the value (1.0 mm/year) measured by the old pendulum which was installed in 1904. This small difference is probably due to the short period of observation of the new monitoring system (3 years) which could be influenced by anomalous temperature variation.

Very significant is also the opening of the joint between the tower and the vertical wall of adjacent building, which is measured by instruments EL5 and EL7. These crack-gauges show an average deformation trend of about 0.10 mm/year at a height of about 4.0 m from the ground which means 1.10 mm at the top of the tower where the wire of the direct pendulum is anchored (Fig. 11).

It can be observed that this value is very close to the value measured by the direct pendulum.
Also the tilting at the base of the tower, measured by the inclinometers shows a trend of the rotation of about 0.011 mrad/year which means a horizontal movement at the top of the tower of about 0.5 mm, which is lower than that measured by direct pendulum and crack-gauges. This difference is probably due to local non-linear deformations which can affect the measure of the inclinometers and to the lower sensitivity of the instrument in comparison with that of the pendulum and crack-gauges. The principle of "redundancy" which was assumed at the base of the design of the monitoring system, allows, through the comparative analysis of the data coming

from different instruments, to select the most reliable

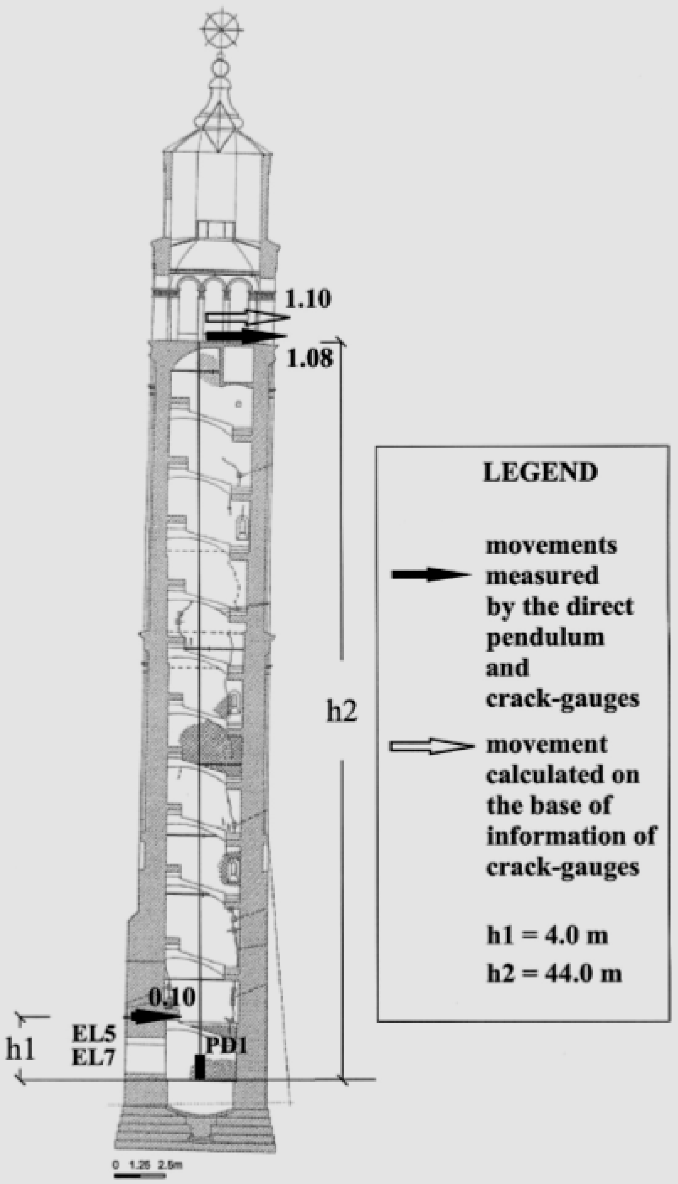

Figure 11. Comparison between the deformations trends measured by the crack-gauges EL5 and EL7 and the component X of the direct pendulum. The crack-gauges show an average deformation trend of about 0.10 mm/year at a height of about 4.0 m from the ground which means 1.10 mm at the top of the bell's cell (44 m) where the wire of the direct pendulum is anchored. This value is very close to the value measured by the direct pendulum (1.08 mm).

instruments able to check the real deformation behaviour of the bell-tower.

The progressive tilting movements of the bell-tower, clearly shown by the monitoring system, indicates the need of urgent strengthening interventions aimed at eliminate or strongly reduce this trend. These interventions must involve the soil-foundations system of the tower, taking into account the complexity deriving not only by the particular properties of the soil in Venice but also by the interaction between the quite different original foundation of the tower and those of the buttresses.

For the design of these strengthening interventions, which involve both geotechnical and structural problems, a proper geotechnical investigation is starting, including coring and in situ and laboratory tests, in order to examine in detail the physical and mechanical characteristics of the different soil layers under the tower up to a depth of more than 30 m.

Very interesting in this sense are the indications coming from the described structural analyses. The inversion in terms of compressive stresses on the opposite faces of the buttresses is not recognizable by the numerical model just by assuming a rigid body rotation of the bell-tower structure, in order to obtain the actual (2002) leaning of the tower. Only imposing a very small differential rotation between the bell-tower's and buttresses foundations, meaning a "flexible" behaviour of the substructures, an acceptable accordance between the numerical and the experimental results is obtained.

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