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Structural Evaluation of the Leaning Bell Tower of S.Stefano in Venice

Analyse structurale du clocher penché de Saint Stéphane, Venise

Konstruktionsanalyse des Schiefen Glockenturms der St. Stephankirche in Venedig

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SUMMARY

The San Stefano bell tower, Venice, approximately 60 m high, was constructed in two separated phases between the end of the 15th and the first half of the 16th century. It suffered severe structural deterioration due to a continuously increasing inclination caued by differential foundation settlements. The situation is not yet stabilised even after the extensive strengthening interventions executed at the beginning of the 20th century. The paper presents the main results of the experimental and theoretical investigations recently carried out in order to assess the actual safety conditions of the tower and, if needed, to suggest new interventions.

RÉSUMÉ

Le clocher, de 60 mètres de hauteur environ, a été construit en deux phases à la fin du 15e siècle et dans la première moitié du 16e siècle. Il a été soumis à des dégradations structurales très importantes résultant des tassements différentiels de sa fondation. Le phénomène n'est pas encore arrête malgré les restaurations effectuées au début du 20e siècle. L'article présente les résultats principaux de recherches expérimentales et théoriques qui ont été conduites récemment pour établir les réelles conditions de sécurité de la tour et définir des interventions supplémentaires.

ZUSAMMENFASSUNG

Der Glockenturm ist ungefähr 60 Meter hoch und wurde in zwei verschiedenen Zeitabschnitten zwischen dem Ende des 15. Jahrhunderts und der ersten Hälfte des 16. Jahrhunderts gebaut. Infolge einer ständig wachsenden Neigung bedingt durch das Nachgeben und die Senkung der Grundmauern, erlitt der Glockenturm schwerwiegende strukturelle Verschlechterungen. Dieser Zustand hat sich bisher noch nicht stabilisiert, obwohl zu Beginn des Jahrhunderts Eingriffe zur Verstärkung und Festigung vorgenommen wurden. In den Unterlagen werden die wichtigsten Ergebnisse der experimentellen und theoretischen Untersuchung dargelegt, die kürzlich zum Nachweis der derzeitigen Sicherheitsbedingungen des Glockenturms und gegebenenfalls zur Einführung neuer Eingriffe durchgeführt wurde.

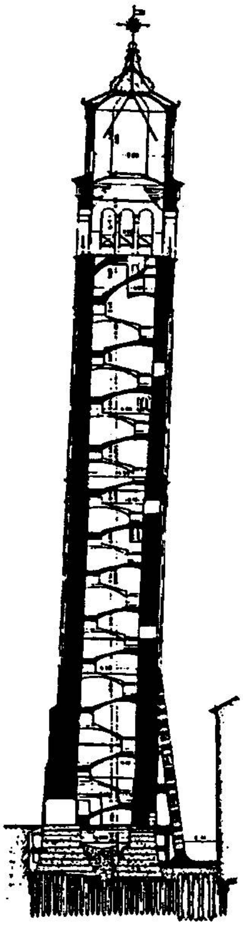


Fig.1 Section of the bell tower (1905)

1. INTRODUCTION

The S. Stefano bell tower, standing in the very center of Venice, suffered very similar structural deterioration phenomena as many historical tall masonry constructions in Italy. The tower has been built in two phases, the first in the second half of the XV century and the second in the middle of the XVI century. Its history is characterized by a continuously increasing inclination and sever damages, the most important due to lightning and fires.

What makes the case very peculiar are the very important strengthening interventions executed at the beginning of the 20th century. At the end of the XIX century the inclination was considered in fact to be increasing at a rate of the order of 7 mm per year, and several inspections of the cracks and measures of the inclination were being made. The collapse of the S.Marco tower made of course the situation even more dramatic, and after long discussions the project was approved to widen the foundation system and to construct inclined buttresses, in addition to execute several local strengthening interventions on the existing masonry walls (1903-1905).

In the following period (1903-1933), the inclination was still increasing, even if at a reduced rate of the order of 1.5 mm per year and new deterioration phenomena in the mean time appeared, regarding above all the steel ties installed at the beginning of the century.

For such reasons the tower is currently subjected to an extensive program of experimental and theoretical investigations with the aim of assessing its current structural behavior and safety level and deciding any possible interventions.

2. PRINCIPAL CHARACTERISTICS OF THE TOWER

The structure is an isolated tower, without any structural connection with the surrounding large buildings, among which is the S.Stefano church. Its height is approximately 60 m and the main dimensions of the horizontal plan section are 7.3 and 7.3 m. The thickness of the walls varies from 180 cm at the base to 110 cm at the belfry.

The original foundation rests on short timber piling and the east side of the building is bordered by a narrow channel, the "Rivo Menuo". At the height of 44 m above the sea level the tower show an out-of-plumb in the East direction of about 2m and in the South direction of 0.3 m, corresponding to an inclination of 2.6°.

The following intervention, executed during the period 1903-1905, are worthnoting from the structural point of view.

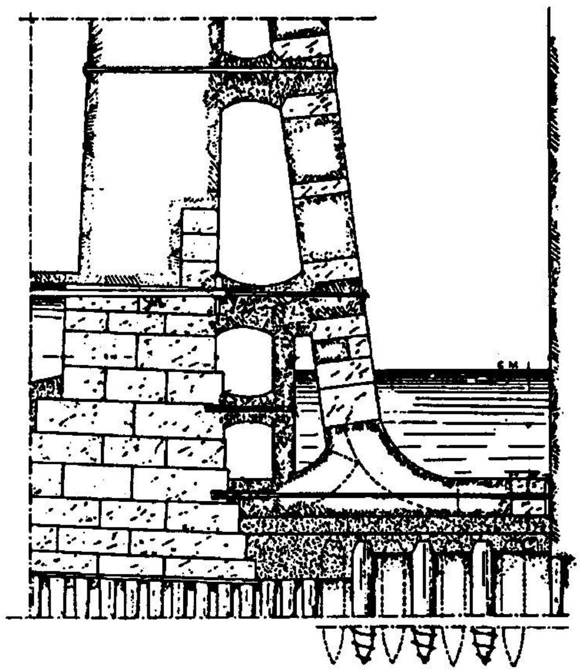


Fig. 2 Detail of the strengthening works executed in 1903-1905

Several steel ties were installed at different levels, starting from the foundation. The masonry walls were extensively repaired and strengthened by means of local reconstructions and injections of cement-water admixtures. The original foundations were first strengthened and then widened over the channel-bed constructing a 50 cm thick concrete slab over a concrete piling. New strong masonry buttresses were built with cement mortar, inclined with respect to the vertical direction as indicated in figure 1. They were securely connected to the tower walls and to the foundations, in the second case through masonry arches (Fig. 2).
The structural behavior of the tower was in this way substantially modified. The variation of the inclination was since then efficiently counteracted , even if not eliminated as expected by the designer, by the new structures as demonstrated by the sudden reduction of the increasing rate of the inclination. A mechanical pendulum was in that occasion installed to control the effectiveness of the interventions.

3. EXPERIMENTAL AND NUMERICAL INVESTIGATIONS

The first studies, which are here summarized, regarded the masonry structure in order to assess the actual safety conditions and to adopt, if necessary, adequate and immediate protection measures, while the investigations on soil-foundation system have been scheduled for a following phase.
At first a geometric survey was made together with an accurate detection of cracks and deterioration phenomena using mountain-climber's techniques to examine the external side.
Besides the deterioration phenomena which were already described at the beginning of the century, in particular the cracks pattern, new phenomena resulted to be clearly connected to the recent interventions. In particular the oxidation processes of all type metallic components resulted to be very severe, in some cases jeopardizing the efficiency of the important ties.

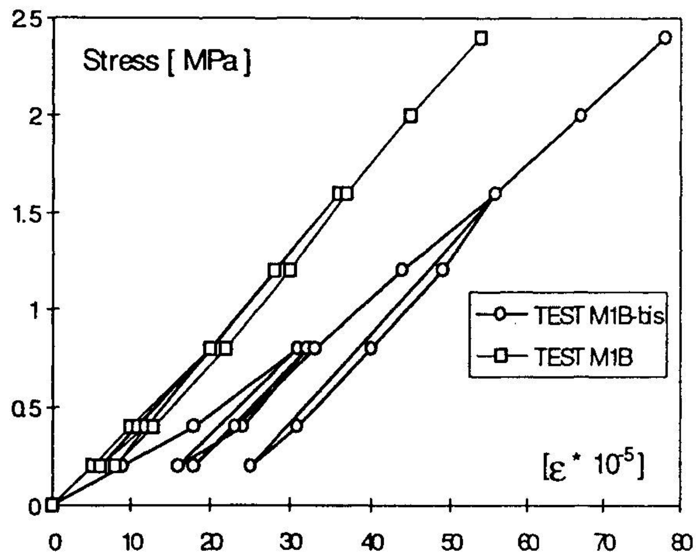


Fig. 3 Stress-strain diagram measured on the external leaf (M1B) and in the internal leaf (M1B-bis)

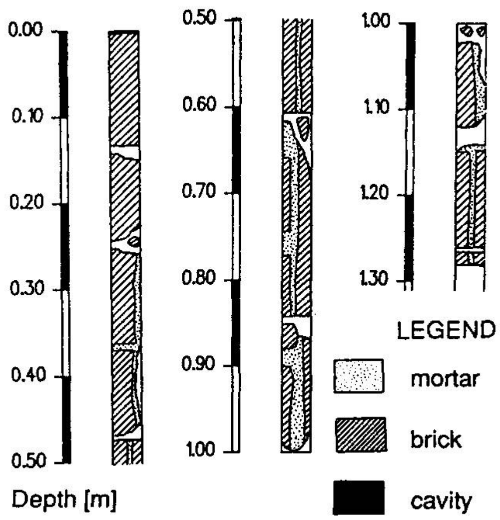
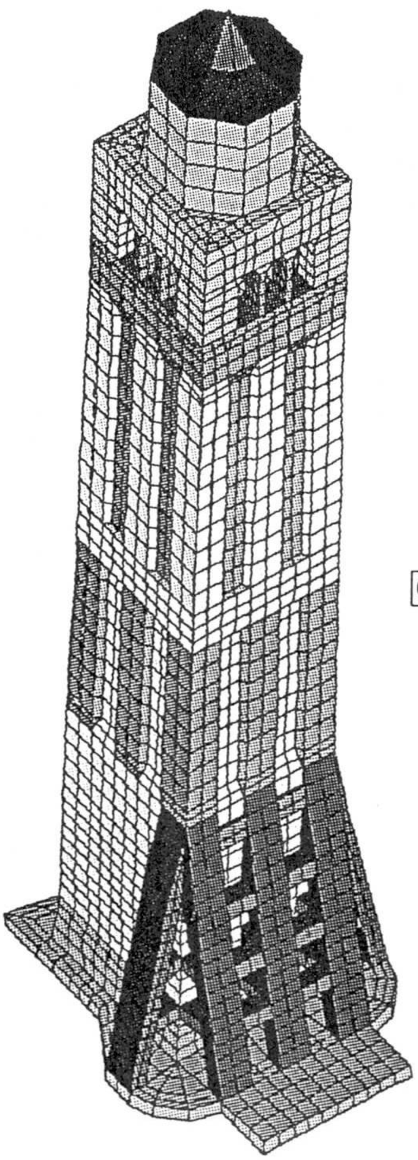


Fig.4 Typical "through-the-thickness" masonry composition

The masonry composition was then investigated by means of cores boring and bore-hole video surveys while the mechanical properties and the state of stress have been measured by flat-jacks tests.
A typical example of masonry composition is shown in Figure 4, where the inner core of the wall appears similar in composition to the outer leaf and almost without voids and cavities.

The masonry constructed in the XV century exhibits a mean modulus of elasticity in the vertical direction of about 4000 MPa (in the 0.4-0.8 MPa range of applied stresses) while the upper part of the tower, built in the middle of the XVI century, show an higher deformability with a mean value of the modulus of 2800 Mpa.
One of the tests was performed both in the external leaf and in the inner core of the XV century masonry. The comparison of the stress-strain curves (Fig. 3) clearly show the difference in the deformability, the inner core modulus is however greater than 2800 MPa and the behaviour is approximately linear-elastic until the maximum imposed stress level of 2.4 MPa.
The modulus of elasticity of the buttresses has been estimated (by means of the results of the single flat jack tests) to be of the order of about three time higher than the modulus of the original masonry.
Several single flat-jack tests were performed at different height to assess the distribution of vertical stresses with a special attention to the problem of the efficiency of buttresses and their effects on the overall structural behaviour of the tower.
Such results were compared with those obtained numerically by means of a FE model (Fig. 5) constructed assuming the mechanical properties of the materials obtained from the above tests.
The FE model has been subsequently calibrated also by comparing dynamic parameters experimentally obtained by recording and elaborating environmental and forced vibrations with the theoretical ones.
On the model thus calibrated static analyses were performed in both linear and non-linear ranges of material behavior mainly to indagate the influence of the buttresses on the distribution of the internal stresses (Fig.7).
The comparative analyses demonstrated that, as there are not no-tension zones, the influence of the material non-linearity is negligible.



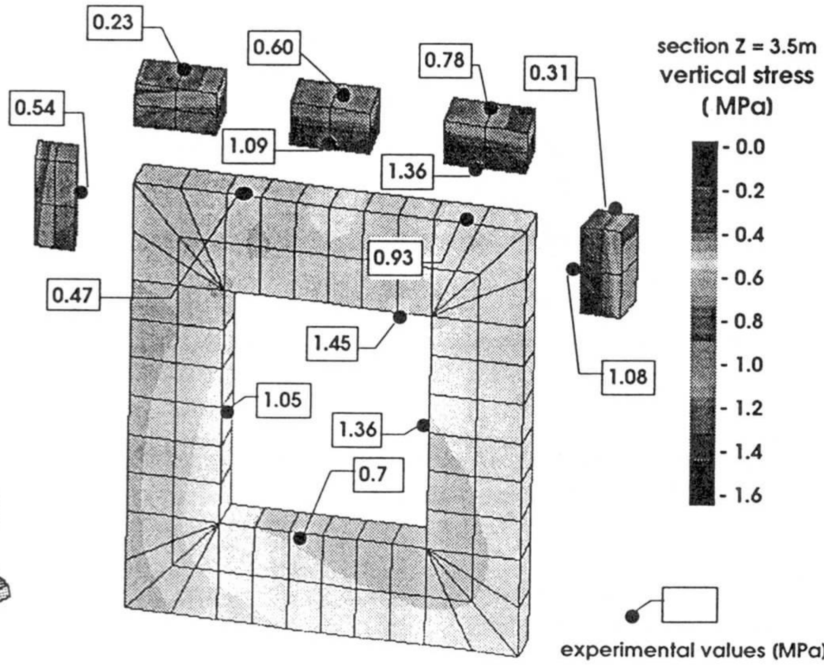


Fig. 5 Mesh of the numerical model

Fig. 6 Experimental and numerical stresses (horizontal section)

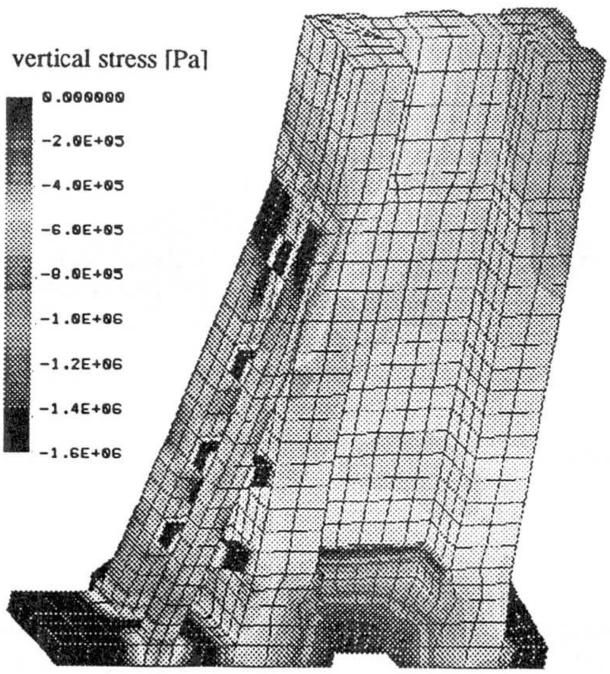


Fig.7 Vertical stresses at the base of the numerical model (vertical section)

Figure 6 shows one example of comparison between theoretical and experimental distribution of normal stresses due to gravitational load. As it can be seen, a fairly good agreement exists between the experimental and theoretical results, except for local inconsistencies which can be explained taking into account the material inhomogeneity, experimentally demonstrated but not yet included in the model. The state of stress is in general compatible, with acceptable safety margins, with the compressive strength of the masonry.

The analysis of the model reproducing the configuration before 1903 (i.e. without buttresses) with the present inclination, shows that the compression stresses would be very close to the masonry strength.

Dynamic investigations have been also performed and helped in both model identification procedures (as discussed above) and in practical evaluations, especially as far as the excitations induced by the bells are concerned.

The measurements were made by means of seismic piezoelectric accelerometers fixed to the structure at different height on the two lateral directions. On those tests two types of dynamic excitation were analysed: vibrations from wind and vibrations from bell movements.

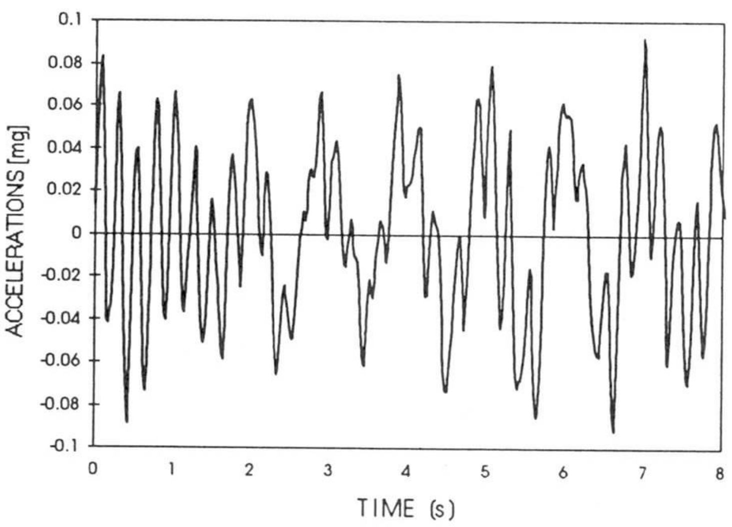


Fig. 8 Wind induced vibrations

From the analysis of wind induced vibrations the first five vibration modes of the structure (two flexural modes in both North-South and East-West direction and the first torsional mode) were determined [2].

The analysis of horizontal forces associated with bells (strike rate of 40 impacts/min. and swing angle of about 90°) showed that the third harmonic component of the force predominates and it is very close to the frequency of the first flexural mode of vibration of the structure causing a dangerous resonance phenomena. The effects measured on the structure were vibrations clearly perceptible to the persons [1] with a displacement amplitude of over 1 mm.

In Figure 9 and 10 a typical record of an acceleration time-history and the correspondent evolutionary power spectra are displayed, from which the above consideration were derived.

Finally, the mechanical pendulum was restored allowing for a new measure of the inclination, which demonstrated that since 1933 the inclination increased at a rate of 1.3 mm per year.

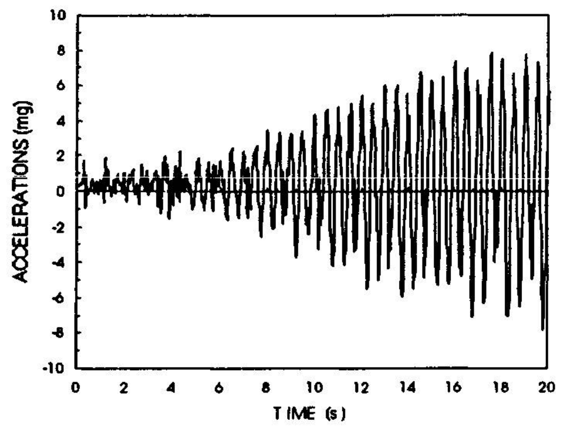


Fig. 9 Vibrations induced by bell movements
Acceleration Vs Time diagram

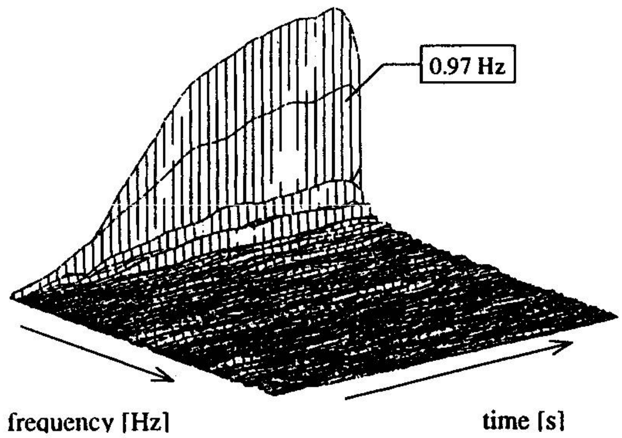


Fig. 10 Vibrations induced by bell movements:
Short Time Fourier Transform

4. CONCLUSIONS

The investigations demonstrated that, in the average, the tower is presently in acceptable safety conditions. The state of stress is in fact compatible with the masonry strength and the inclination will attain critical values for the global stability [4], with the current increase rate, not before 60 years.
They also demonstrate the crucial role played by the strengthening intervention of the beginning of the century in ensuring the current safety condition of the structure.
The problem of the stability of the foundations has not been however definitively solved, and adequate investigation and intervention must be carried out within the above indicated time limit. Moreover. the bell system must be changed in case it will be used again in order to avoid the previously mentioned resonance problems.

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