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In situ damage assessment and nonlinear modelling of a historical masonry tower

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Abstract

In the present paper the case study of the eighth-century masonry tower called "Torre Sineo" (Alba, Italy) is described. The building has been recently analyzed and monitored because of an emerging damage pattern, and also due to some seismic events during the last few years. Nondestructive evaluation techniques have been adopted to assess the present situation of the tower without perturbations, among them, a detailed geometrical survey and a thermographic analysis. Such investigations reveal the presence of damaged zones concentrated close to the opening regions as well as a deviation of the tower from verticality. In the second part of the paper, the results from a nonlinear analysis are presented, in order to assess the behavior of the tower in the case of increase in the tilt mechanism. Cracking and crushing of the masonry have both been taken into account, as well as the influence of geometrical nonlinearity. The opening of sub-vertical radial cracks suggests compression crushing, while the evolution of the top displacement shows an almost linear evolution with respect to the tilt of the basement. The numerical simulations have also predicted damage in correspondence with the main openings. The numerical analysis has given a valuable picture of possible damage evolution, providing useful hints for the prosecution of structural monitoring.

Keywords: Historical towers; Nonlinear structural analysis; Nondestructive technique

1. Introduction

Damage assessment for historical masonry buildings is often a complex task. It is crucial to distinguish between stable damage patterns and damage evolution leading to a catastrophic structural collapse [1]. Some damage patterns can be subsequently activated by unpredictable events such as earthquakes, or by inappropriate functional extensions and restorations. In addition, the limited ductility of the masonry, combined with the large scale of the tower, provides a rather brittle structural behavior [2]. The masonry building called "Torre Sineo" (Fig. 1(a)), dated as eighth century, is the tallest and most mighty of the medieval towers preserved in the town of Alba (Italy). The square-plan structure (Fig. 1(b)), measuring 5.9 × 5.9 m, is about 39 m high and is leaning to the north side. The foundations lie 3.5 m below from the surface of the street level. The walls are of variable thickness, between 2.0 and 0.8 m.

The sustaining wall is "a sacco" with the external bricks joined with a mortar layer one centimeter thick. The internal filling is composed of remainders and bricks tied by a poor mortar. The tower is incorporated for 15 m in a later dated building. As regards the incorporated part, the floors have been realized through masonry vaults, while in the upper part of the tower the floors are made with wooden structures.

In the following, a report on the experimental and numerical analysis of the last few years is presented. Many of the aspects faced in this study are rather typical and are present for analogous tower structures, widely present in Italian territory [3,4].

2. Nondestructive evaluation tests

The importance of evaluating existing masonry buildings by means of in situ nondestructive investigations has

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been mentioned by many authors [5,6]. Nondestructive evaluation (NDE) techniques can be used for several purposes: (i) detection of hidden structural elements, such as floor structures, arches, piers; (ii) classification of masonry, mapping the nonhomogeneity of the materials used in the walls (e.g. use of different bricks during the life of the building); (iii) evaluation of the extent of the mechanical damage in cracked structures; (iv) detection of the presence of voids and flaws; (v) evaluation of moisture content and capillary rising; (vi) detection of surface decay; and (vii) evaluation of mortar and brick or stone mechanical and physical properties [7].

In the present study, three main aspects have been taken into consideration: the acquisition of geometry and deviation from verticality, the acquisition of compressive stress and deformability by means of single and double flat-jack tests, and the acquisition of the present damage pattern by thermographic analysis.

2.1. Geometrical survey

The geometry of the tower and of the surrounding building has been completely acquired and organized within a CAD system. The positions of openings and the variation of the thickness of the tower with respect to that quoted have been carefully recorded, together with the positions of the main visible cracks in the structure. The CAD model served as a basis for the mesh generation needed for further FEM analysis.

The deviation of the tower from verticality has been evaluated using an optical instrument (Sokkia SEF 4110R). Side 1 of the tower is leaning towards the north. The maximum eccentricity was measured at the top level, and was equal to 39 cm north, and 3 cm west. Measurements performed at different heights suggest that the tower experienced a rigid body tilting, i.e. no appreciable deviation from straightness was recorded.

The cracking network can be observed both in the internal and in the external overview. The most significant cracks are inside the tower, mainly located between the sixth and eighth floors. On the external side we can observe minor cracks, mainly near the windows, more specifically between the sixth and seventh floors. The crack pattern and the tilt of the tower are schematically summarized in Fig. 2.

2.2. Flat-jack tests

Both single and double flat-jack tests have been performed at the ground level and one floor below.

The single flat-jack test concerns the measurements of in situ compressive stress in existing masonry structures by use of a thin flat-jack device that is installed in a saw cut mortar joint of the masonry wall [8]. The method is partially nondestructive. After the slot is formed in the masonry, compressive stress at that point causes the masonry above and below the slot to get closer. Inserting the flat-jack into the slot and increasing its internal pressure until the original distance between points above and below the slot is restored can thus give a measurement of the compressive stress in the masonry. The slots in the masonry are prepared by removing the mortar from masonry bed joints, avoiding disturbing the masonry. Care must be taken in order to remove all mortar in the bed joint, so that pressure exerted by the flat-jack can be directly applied against the cleaned surface of the masonry units. The state of compressive stress in the masonry is approximately equal to the flat-jack pressure multiplied by factors which account for the ratio K_a of the bearing area of the jack in contact with the masonry to the bearing area of the slot, and for the physical characteristic of the jack K_m. In fact, the flat-jack has an inherent stiffness which

Fig. 1. The elevation view of the tower (a). The plan of the tower and of the surrounding building (b).

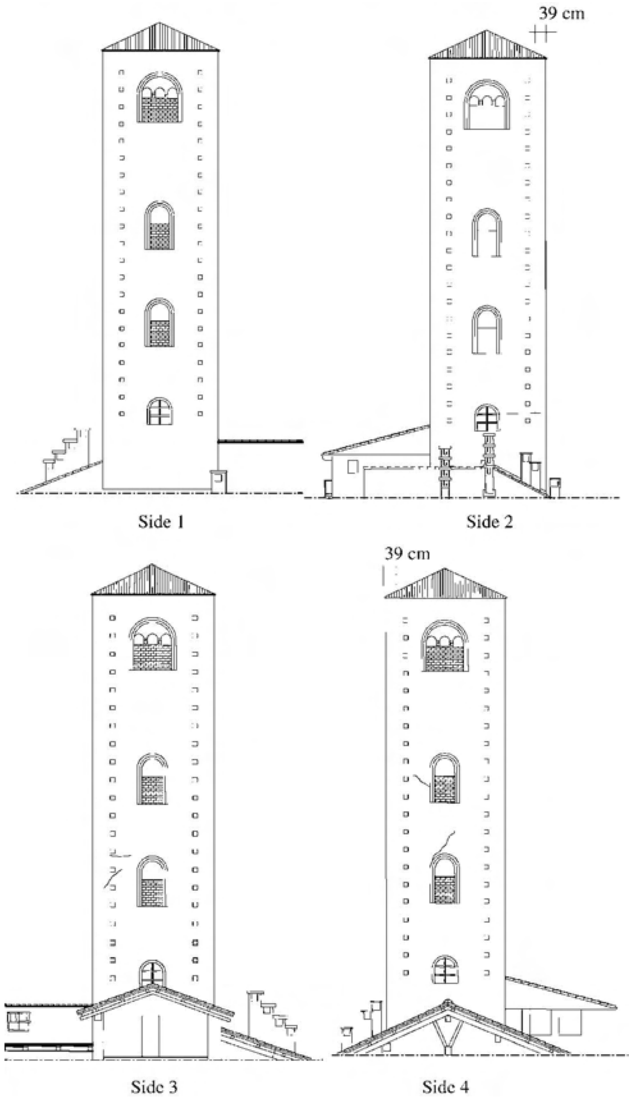

Fig. 2. Elevations of the four sides of the tower. Notice the presence of cracks close to the openings and the deviation from verticality of the tower.

opposes expansion when the jack is pressurized. Therefore, the fluid pressure in the flat-jack is greater than the stress that the flat-jack applies to masonry, and a conversion factor K_m is necessary to relate the internal fluid pressure to the stress really applied. The average compressive stress in the masonry, f_m, can be calculated as

$$f_m = K_m K_a p, \tag{1}$$

where p is the flat-jack pressure required to restore the gage points to the distance initially measured between them. The usual coefficient of variation of this test method can be estimated as equal to 20%; therefore, at least three tests have been carried out on each area of interest.

The double flat-jack test provides a partially nondestructive method for determining the deformation properties of existing unreinforced solid-unit masonry [9]. The test is carried out by inserting two flat-jacks into parallel slots, one above the other, in a solid-unit masonry wall. By gradually

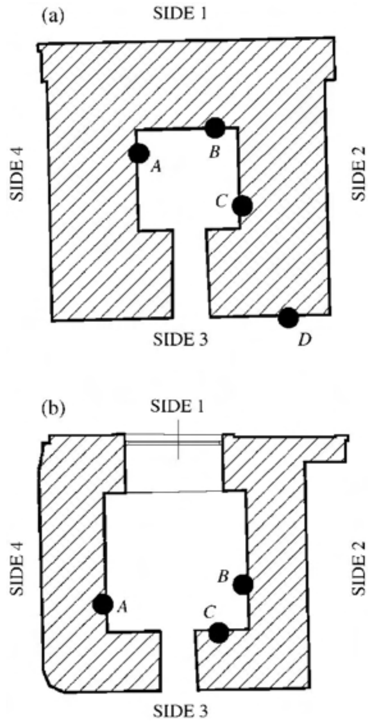

Fig. 3. The plan scheme of the sites of the single and double flat-jack tests. Test sites at the foundation floor (a), and at the ground floor (b).

increasing the flat-jack pressure, a compressive stress is induced on the masonry located in between. The stress–strain relation can thus be obtained by measuring the deformation of the masonry. In addition, the compressive strength can be obtained, if the test is continued to local failure. However, this may also cause damage to the masonry in the area adjacent to the flat-jacks. The tangent stiffness modulus at any stress interval can be obtained as follows:

$$E_t = \frac{\delta\sigma_m}{\delta\varepsilon_m}, \tag{2}$$

where $\delta\sigma_m$ is the increment of stress, and $\delta\varepsilon_m$ is the increment of strain. On the other hand, the secant modulus is given by

$$E_t = \frac{\sigma_m}{\varepsilon_m}, \tag{3}$$

where σ_m and ε_m are the stress and strain in the masonry.

In situ vertical stress and elastic modulus have been acquired at two different height levels, following the scheme shown in Fig. 3. The results are summarized in Table 1.

The complete stress–strain curve during the double flat-jack test is shown in Fig. 4. The sudden increase of the lateral deformation is due to the opening of vertical splitting cracks; therefore it useful to assess the first cracking load.

2.3. Thermography

Thermovision is a NDT which has been applied for several years to artwork and monumental buildings [6]. The thermographic survey has the advantage of being applicable

Table 1
Results from single and double flat-jack tests

Points	Foundation floor		Ground floor	
	σ_z	E	σ_z	E
A	2.455	–	0.871	–
B	0.297	–	0.746	–
C	1.059	–	–	–
D	0.502	–	–	5000

Average compressive stresses and Young's moduli are in MPa.

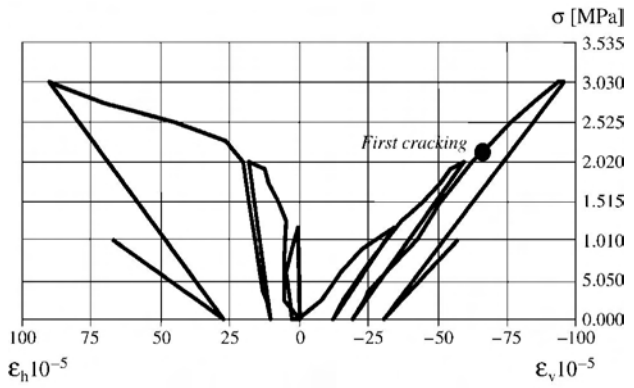

Fig. 4. The outcome from the double flat-jack test; vertical and horizontal strains are plotted with respect to the stress in the wall.

to wide surfaces of walls; it is a telemetric method and presents high thermal and spatial resolution.

Thermographic analysis is based on the thermal conductivity of a material and may be passive or active. The passive application analyses the radiation of a surface during thermal cycles due to natural phenomena (e.g. insulation and consequent cooling). If the survey is active, forced heating of the analyzed surfaces is carried out.

The thermal radiation is collected by a camera sensitive to infrared radiation. In fact, each material emits energy (electromagnetic radiation) in this frequency field; this radiation is characterized by thermal conductivity, i.e. the capacity of the material itself for transmitting heat, and its own specific heat.

Each component of an inhomogeneous material such as masonry shows different temperatures. The thermovision detects the infrared radiation emitted by the wall. The result is a thermographic image on a colored or black and white scale. Each tone corresponds to a temperature range. Usually, the difference of temperatures is around the fraction of degree level.

The total flux of energy E emitted by a surface is the sum of the energy E_c emitted by the surface by thermal excitation and of the flux E_r that is emitted by the surface around each point:

$$E = E_c + E_r. \tag{4}$$

The infrared camera measures the energy flux E. The test is carried out at a certain distance, without any physical contact with the surface.

Active thermovision can also be carried out for tests on depth. The surface of the tested wall should be heated for a certain time. In this way, the thermal conductivity of the internal part of the masonry is shown up to a certain depth. The infrared camera transforms the thermal radiation into electric signals, successively converted into images. These images can be visualized on a monitor and recorded on a computer. In the video camera, the infrared radiation that reaches the objective is transmitted by an optical system to a semiconductor element. The latter converts the radiation into a video signal, while the surveying unit signal processes the video camera signals and shows the thermographic image.

Thermovision can be very useful in diagnosis; in fact it is used to identify areas under renderings and plasters that can hide construction anomalies. It is particularly interesting for studies on frescoed walls, where it is not possible to take samples or to use testing techniques that come in contact with the frescoed surfaces. Other applications can be: (i) survey of cavities; (ii) detection of inclusions of different materials; (iii) detection of water and heating systems; and (iv) detecting moisture presence. In the case of moisture, the camera will find the coldest surface areas, where there is continuous evaporation. The evaporation, on the other hand, is due to the difference in relative humidity between the inside of the masonry and the environment outside, as well as to natural air movements.

In the diagnosis of old masonries, thermovision allows the analysis of more superficial layers, in the absence of thermal irradiation. It should be pointed out that the penetration depth of this technique is limited, so it is unable to locate anomalies which are hidden in the inner part of the masonry. The technique is often sensitive to the boundary conditions of the test. Sometimes shapes are detected which are caused by different local emissions and not by effective temperature variations.

In order to get a more accurate interpretation of thermographic data, the software IRWIN Report 5.21 has been used.

In Fig. 5, a sample of the comprehensive thermographic analysis is shown, relating to one of the most damaged zones in the tower. In Fig. 5(a), (b) a comparison is shown between the view of the tower and the acquisition of the temperature diagram. In Fig. 5(c), the temperature diagram is filtered in order to emphasize the differences in the local heat emission. Some of the measured temperatures are summarized in Table 2.

In general, thermography has revealed that zones respectively above and below the apertures suffered from damage more than elsewhere in the tower.

3. Numerical simulation

3.1. The numerical model

A complete three-dimensional FEM model of the tower has been built using twenty-node isoparametric solid brick

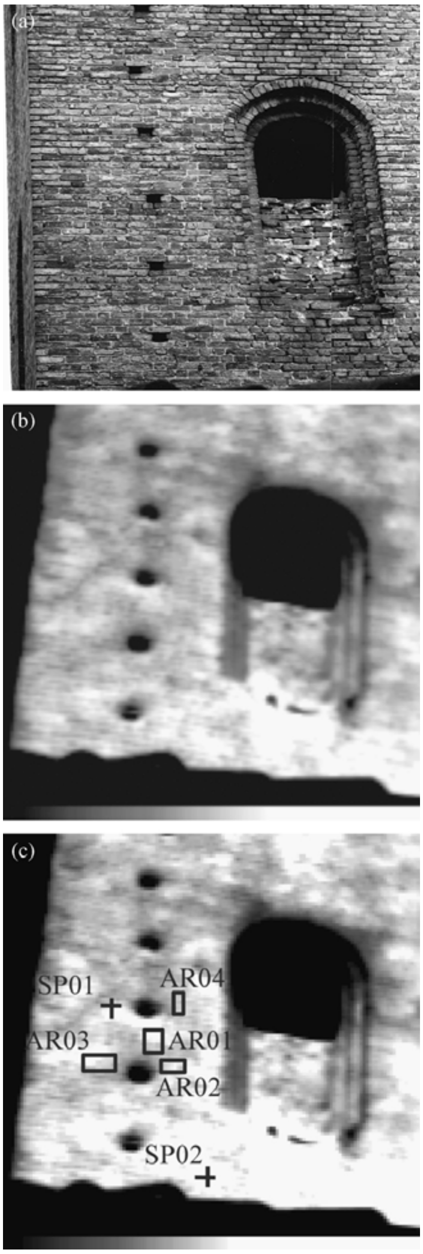

Fig. 5. Thermography: a view of the analyzed detail (a); a diagram of temperature in the range 28.2–34.0 °C (b); the temperature diagram in the range 28.6–32.9 °C (c).

elements, in order to perform the analysis with the commercial code DIANA. At least five nodes are present in the thickness of the tower wall. The model takes into account the presence of openings and the variation of the wall thickness at different levels. On the other hand, the presence of wooden floors has been disregarded. The structure is mainly subjected to its dead load. In addition, the effect of an increasing tilt of the foundation has been considered, combined with the load provided by the wind action exerted on the upper region of the tower.

The effect of wind is taken into account by means of equivalent static loads. Those loads are calculated as overpressure and underpressure acting on the portion of tower higher than the surrounding building. The magnitude of the overpressure, according to the Italian prescriptions, is equal to 1000 Pa, while the underpressure is equal to 500 Pa. For the sake of simplicity, both the load distributions are assumed to be uniform. In order to be conservative, the direction of wind is supposed to be acting accordingly with the tilt mechanism.

Table 2
Surface temperatures acquired through thermography

Points	Temperature (°C)
Spot 1	30.2
Spot 2	32.7
AR 01 mean	31.6
AR 02 mean	30.8
AR 03 mean	31.6
AR 04 mean	30.8

The points relate to Fig. 5(b).

The main mechanical parameters of the model have been directly obtained from the single and double flat-jack tests described in the previous section. Additional parameters, such as the fracture energy of masonry, have been assumed on the basis of destructive experimental tests carried out on similar structures. A crack model based on total strain has been adopted to represent the nonlinear behavior of the masonry [10]. More detailed models can be adopted in the case of laboratory specimens [11], but these become extremely demanding from the computational point of view when real-scale structures are considered. Both tensile mode I cracking with linear softening and compressive crushing are taken into account. Since a fixed crack model was chosen, the shear retention behavior is explicitly evaluated by the code with the provided factor $\beta = 0.01$. The mechanical parameters used in the analysis are summarized in Table 3.

Table 3
Mechanical parameters adopted in the numerical analysis

Parameter	Value
Young modulus E	5000 MPa
Poisson ratio ν	0.2
Density	1600 kg/m^3
Tensile strength	0.3 MPa
Fracture energy G_F	50 J/m^2
Compressive strength	2 MPa

3.2. Elastic analysis

A first elastic analysis was performed taking into account the presence of the dead load and of the tilt of the tower. The effect of tilt is considered in an indirect way, imposing inclined ground acceleration. This allows solving the linear problem in the framework of small deformation and small displacement hypotheses. The vertical stress in the whole

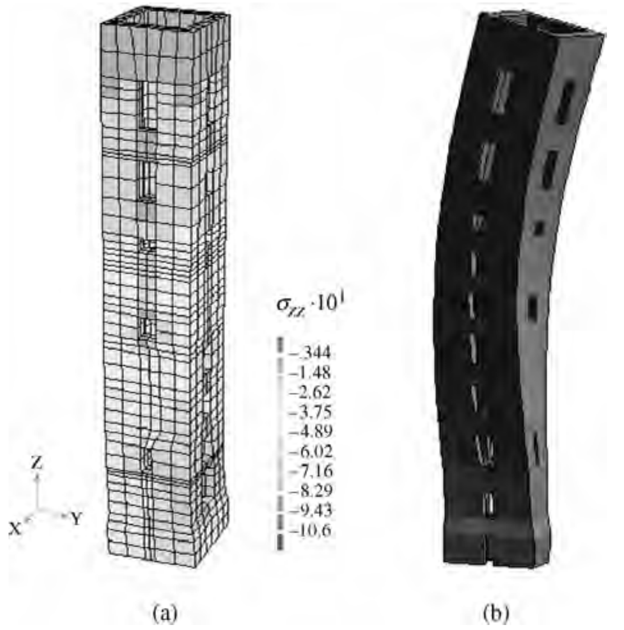

Fig. 6. The mesh of the FEM model and contour plot of the vertical stress field σ_{zz} [MPa] (a). The first torsional modal shape of the tower related to the fourth natural frequency (b).

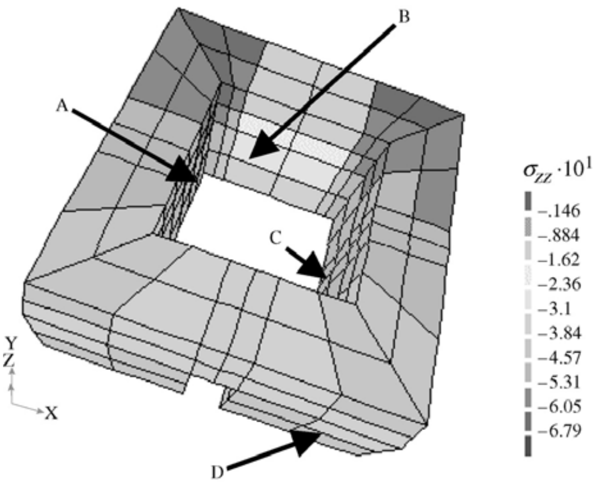

Fig. 7. Detail of the foundation floor wall and of the calculated vertical stress field σ_{zz} [MPa]. Arrows indicate the sites where flat-jack tests have been carried out.

structure is depicted in Fig. 6(a). In Fig. 7, on the other hand, only the foundation floor wall is shown. Vertical stresses are predicted that are in good agreement with experimental flat-jack results. Point B, that is located beneath a large opening in the upper floor, is lightly loaded with a stress of about 0.3 MPa. On the other side, point C is much more loaded, with a stress greater than 0.7 MPa. Point D, placed on the external wall opposite to the tilt of the tower, presents a vertical stress of about 0.5 MPa. The very high stress measured at point A (2.4 MPa) is not predicted by the numerical analysis, but it is likely to be ascribed to a local heterogeneity of the masonry wall and to the corresponding stress concentration.

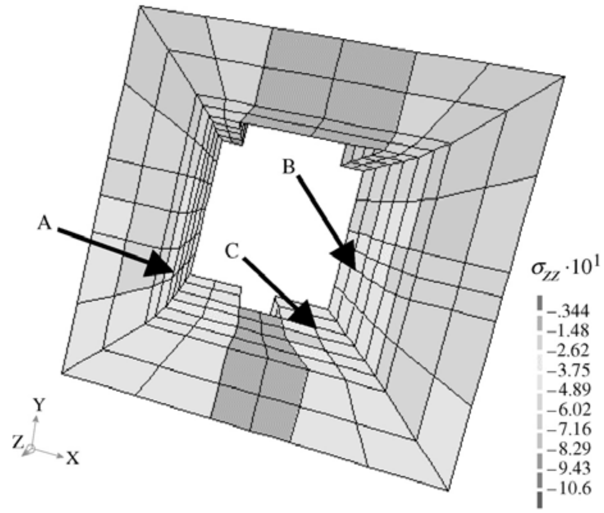

Fig. 8. Detail of the ground floor wall and of the calculated vertical stress field σ_{zz} [MPa]. Arrows indicate the sites where flat-jack tests have been carried out.

The vertical stress field in the ground floor wall is shown in Fig. 8. The stresses at the points A and B, respectively equal to 0.8 and 0.7 MPa, are both in good agreement with the measured values.

The analysis reveals that the structure is basically in elastic conditions, since the level of stresses is everywhere smaller than the intrinsic strength. Such consideration is still valid when the effect of the wind load is taken into account.

The linear investigation was extended to a modal analysis, in order to give a first estimate of the dynamic response of the structure. The first natural frequency is computed to be equal to 0.123 Hz, which corresponds to a period of 7.38 s. The first two modal deformations are basically connected to bending in the two orthogonal directions. The first torsional shape is linked to the fourth natural frequency, and is shown in Fig. 6(b). A more detailed dynamic study of the structure is currently under development, in order to give a structural interpretation of the damage growth due to some recent seismic events.

The good agreement between measured and numerically calculated stresses provides the necessary validation of the FEM model; therefore, the analysis was extended in order to predict damage evolution.

3.3. *Nonlinear analysis*

The aim of the nonlinear analysis is to provide an assessment of the structural stability in the case of an increase in the tilt of the tower. The analysis has been carried out taking into account both material and geometrical nonlinearity. Material nonlinearity concerns the nonlinear stress–strain constitutive equation due to smeared cracking once the tensile strength is exceeded. On the other hand, geometrical nonlinearity means that the geometry is updated after each load step. This is necessary to have a correct

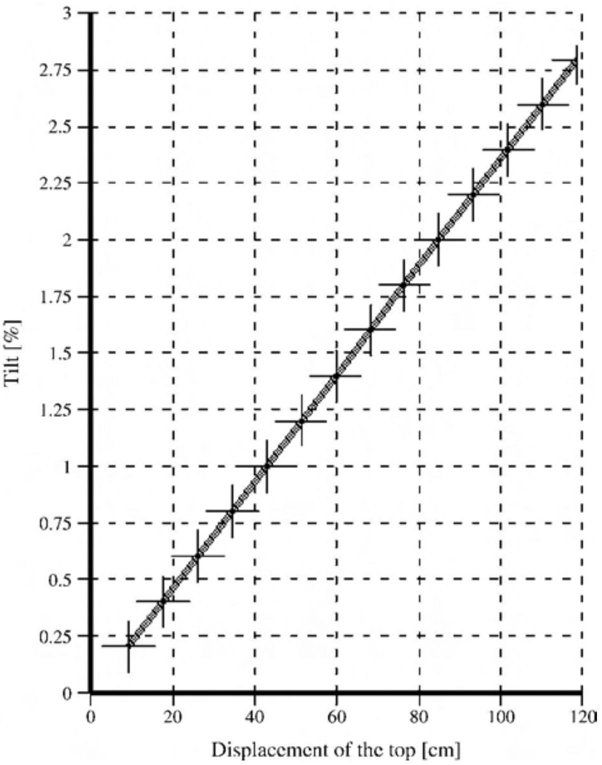

Fig. 9. Evolution of the displacement measured at the top of the tower with respect to the tilt.

estimate of the stresses induced both by the tilt and by the bending of the tower itself.

Although the tower is rather tall, it is not particularly slender and structural instability is not likely to occur. The reason that it is useful to take into account the geometrical nonlinearity is a bit more subtle. In fact, one of the loads to be considered, namely the tilt of the tower, is actually an imposed displacement of the tower basement. This displacement is compatible with the tower constraints, since the tower is basically a statically determined vertical cantilever. If the analysis is performed under the hypothesis of small strains and displacements, the equilibrium is verified only in the undeformed configuration; therefore no stresses arise due to the effect of tilting.

On the other hand, the eccentricity of the dead load due to the tilt can be taken into account by updating the geometry of the structure during the loading procedure (i.e. performing a geometrical nonlinear analysis).

The loads are applied following a classical nonlinear incremental scheme. First of all, the dead load and the wind load were applied to the structure. At the end of this first loading step, no damage has arisen in the structure.

After that, the tilt is increased. The choice of the increment magnitude is driven by the arc-length algorithm. The diagram in Fig. 9 shows the evolution of the calculated displacement at the top of the tower with respect to the tilt. A tilt of 1% relates to a displacement of the tower top equal to 42 cm, slightly greater than the actual amount, due to the fact that the analysis considers also the wind action (with a mean speed of 25 m/s, corresponding to a renewal time of 50 years).

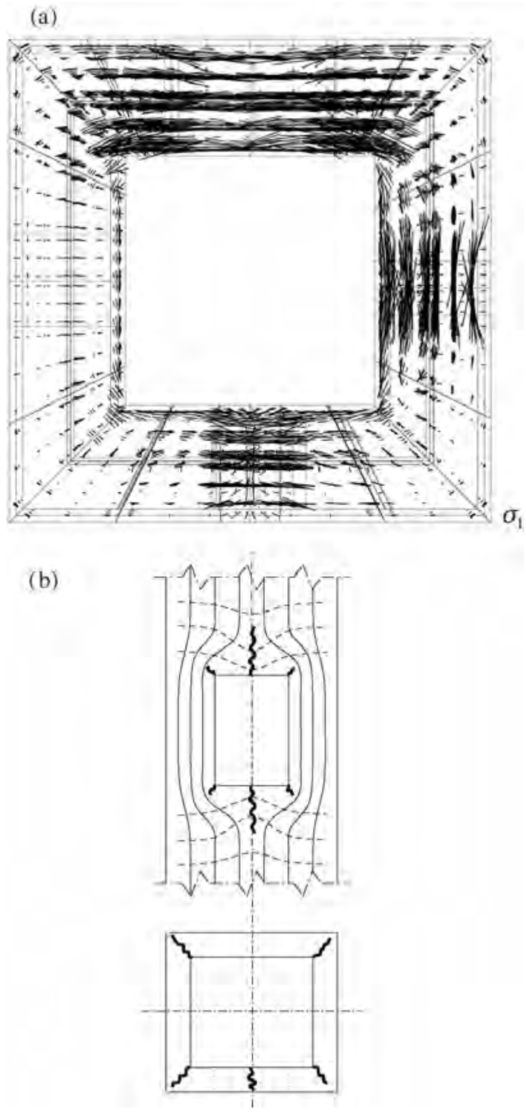

Fig. 10. The principal tensile stress field σ_1 in the foundation wall (a). The scheme of principal stress trajectories and cracking: tension (dashed), compression (continuous), and crack path (bold) (b).

As the tilt is increased, the first cracks nucleate in the model. The region that is most sensitive to cracking is placed in the lower part of the tower. As shown in Fig. 10(a), in the foundation floor wall, principal tensile stresses arise along the circumferential direction, and their magnitude is higher close to those for the apertures in the tower. When the tensile strength is reached, the cracks start to open.

When the tower tilts it is reasonable that compression at the left side increases, while the right side is progressively unloaded. If the tower can be likened to a Saint Venant beam subjected to an eccentric axial force, tensile stress does not arise until the eccentricity is kept inside the *central core of inertia* of the tower cross section. On the other hand, when the axial force is applied outside the central core of inertia,

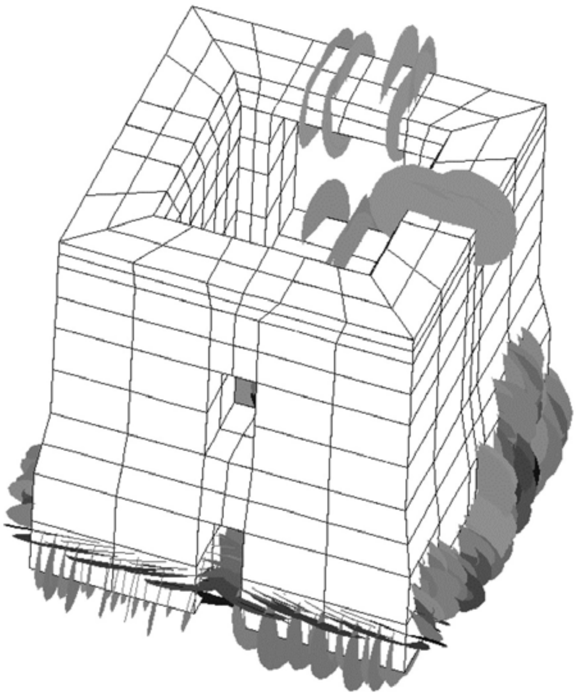

Fig. 11. Discs represent crack planes at the Gauss points in cracked elements. Note that elements close to the aperture of the tower are more likely to crack (i.e. for high concentration of discs).

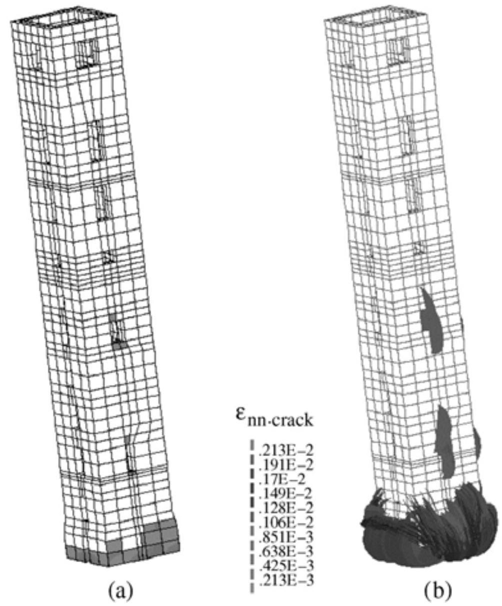

Fig. 12. Cracked elements relating to a 3% tilt of the foundation (a). Sub-vertical radial cracks arise in the foundation wall, as well as cracks close to the openings at higher floors (b).

cracks arise in no-tension material orthogonal to the beam axis.

In the present case, the tower is much more complicated than a Saint Venant beam due to its hollow core and the presence of several apertures such as windows. Therefore, the trajectories of the compression principal stresses are not straight and parallel. Above and below the apertures those trajectories are forced to curve because of the abrupt change of the tower cross section, as shown schematically in Fig. 10(b). As a consequence, tensile stresses arise orthogonal to those directions. The magnitude of those tensile stresses can be relevant well before the eccentricity exceeds that of the central core of the inertia region. As soon as those tensile stresses reach the tensile strength of the masonry, cracks nucleate both on lintels and window sills (as often detected in historical masonry constructions).

A similar effect is caused by the hollow cavity in the tower. In particular, the presence of four re-entrant corners is responsible for the well known stress singularity described by linear elastic fracture mechanics.

In both cases cracks are characterized by sub-vertical cracking planes that are almost radial with respect to the tower axis.

On the other hand, cracking due to the axial force exceeding that of the central core of inertia is characterized by crack planes that are horizontal with respect to the tower, and are located on one side of it.

In Fig. 11, discs represent the plane of cracks at Gauss points in the cracked elements.

It is worth noting that, although the displacement of the tower top evolves almost linearly with the tilt, the damage increases in the structure. After the foundation region, the parts close to the openings in the upper segment of the tower start to crack. This can be seen in Fig. 12, which relates to a tilt of about 3%. Moreover, this evidence is in good agreement with results from thermography, which indicate the area close to the openings to be particularly sensitive to damage. When the tilt is greater than 3%, also crushing of elements, due to the reaching of the ultimate compressive strength, comes into play. It can be concluded that the value of tilt equal to 3%, and the corresponding horizontal displacement of the tower top equal to 125 cm, should be considered as the ultimate conditions for the structure.

4. Conclusions

A general procedure for assessing the stability of a historical masonry tower has been proposed, that combines NDE techniques with appropriate nonlinear numerical simulations. The approach has been followed in the study of the medieval tower called “Torre Sineo” in Alba (Italy). The main damage mechanisms in the structure have been clearly individuated, discriminating between stable and evolving patterns. The evolving scenarios have been studied using a nonlinear numerical model, in order to provide a stability assessment and to describe the progressive decrease of the safety factor when the deviation from verticality increases. The results from the numerical analysis, combined with monitoring of the structure [12], give valuable hints for assessing not only how, but also when, structural restoration has to take place. In addition, nonlinear input parameter values are specified, based also on the experimental validation, which are usually hard to decide, and thus could help in similar analyses.

Acknowledgements

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