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Original article

A multilevel approach for the damage assessment of Historic masonry towers

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ABSTRACT

In the paper some case studies of damaged towers in Italy are presented: the Tower Masserano of Palace Ferrero - La Marmora at Biella, the bell tower of the Collegiata of San Vittore at Arcisate, and three medieval towers in Alba. A methodology is put forward for combining laboratory and nondestructive testing methods with monitoring systems in order to reach the goal of evaluating the state of conservation of historic towers and its evolution in time.

1. Introduction

The influence of time on the mechanical behaviour of masonry structures became evident after the collapse of the medieval Tower of Pavia, when the identification of a time-dependent behaviour, probably coupled in a synergetic way to cyclic loads, was identified as a possible explanation of the sudden collapse.

Ancient buildings often show diffused crack patterns, due to different causes in relation to original function, construction technique and loading history. In many cases, the dead load, usually very high in massive monumental buildings, plays a major role into the formation and propagation of the crack pattern.

Laboratory and nondestructive testing methods combined with monitoring systems allow to reliably evaluate the state of conservation of historic towers and its evolution in time.

In this work, the results obtained from an experimental and theoretical point of view from some case studies are shown. The reliability of different available approaches to the study of historical masonry is put forward in order to provide a criterion for the assessment of masonry towers. The results of pseudocreep tests on the medieval and the XVIth century masonry coming from the ruins of the collapsed tower of Pavia and that coming from the XVIth century crypt of the Cathedral of Monza are presented. The interpretation of such results is given in terms of probabilistic and rheological models that account for the detailed meso-structure of the masonry (units, mortar joints and their complex texture), and are able to make previsions on the masonry service life. Nondestructive and slightly destructive tests as sonic tests and flat-jack tests will be presented concerning the case studies of the Tower Masserano of Palace Ferrero - La Marmora at Biella and of the Bell Tower of the Collegiata of San Vittore at Arcisate. On the other hand, the Acoustic Emission technique, which allows for the assessment of the damage evolution based on the crack advancement energy release, will be introduced when dealing with the Alba towers monitoring. It is worth noting that the pseudocreep tests and the interpretation of the rheological behavior of cracked masonry detected by acoustic emission can be considered the most promising techniques to provide the service life assessment of the structures. Finally, an extensive numerical simulation of the whole structure of the three Alba towers is presented. The combination of numerical and experimental results, taking into consideration different loading conditions, allows for a robust assessment of the structural safety margin.

Suggesting rehabilitation procedures and retrofitting techniques, in the case of masonry towers and other typologies of monumental buildings, is not part of the purposes of the present paper. Indication on established intervention approaches can be found in [1–3].

2. Pseudocreep tests on masonry

The unexpected failure of some monumental buildings that took place over the last 20 years have highlighted with great evidence that, in addition to external factors, an intrinsic feature of masonry (as of any rock-like materials) has to be included among the risk factors menacing the safety of ancient structures.

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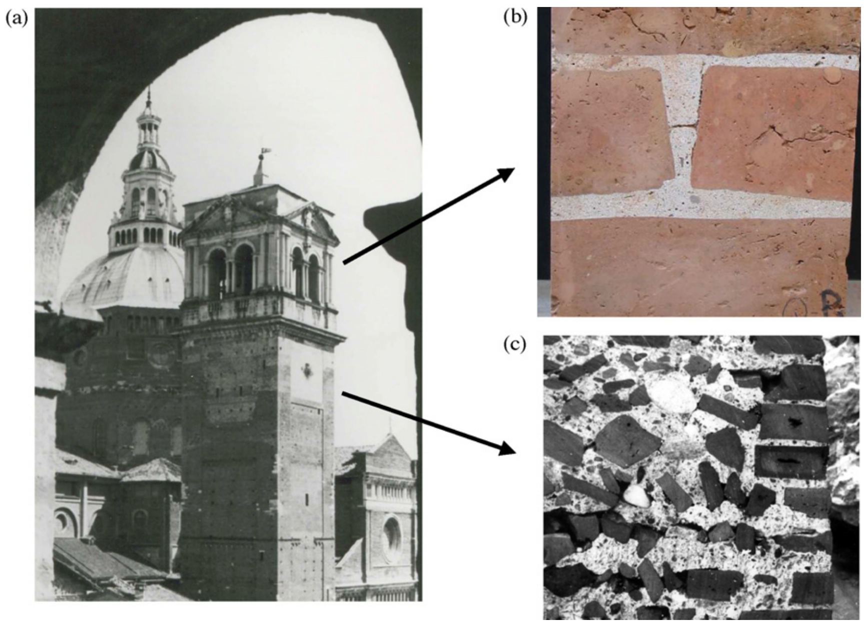

Fig. 1. (a) Civic Tower of Pavia; (b) XVI cent. plain and (c) medieval rubble masonry.

The external factors concern the lack of maintenance, the load increase due to the building modifications, the soil settlements, the mechanical shocks due to earthquakes, fires, etc. On the other hand, this feature is the time-dependent mechanical behaviour or creep, which is basically related to the self-weight. Exploiting the medieval and the XVIth century masonry coming from the ruins of the collapsed tower of Pavia and that coming from the XVIth century crypt of the Cathedral of Monza, several experimental procedures have been adopted to understand the phenomenon, from creep to pseudocreep tests at different time intervals, and various rheological models have been applied to describe the creep evolution and creep-induced damage [4].

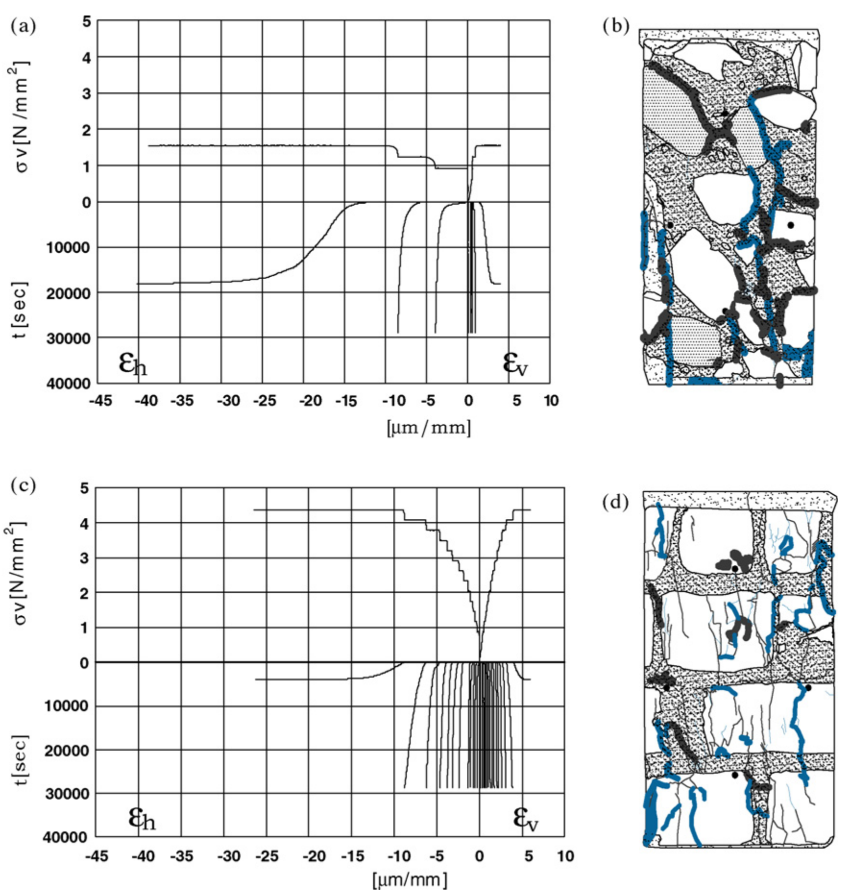

Fig. 2. (a–b) Pseudo-creep test and crack pattern on a prism of the inner leaf of the medieval masonry and (c–d) on a prism of the XVI cent. belfry.

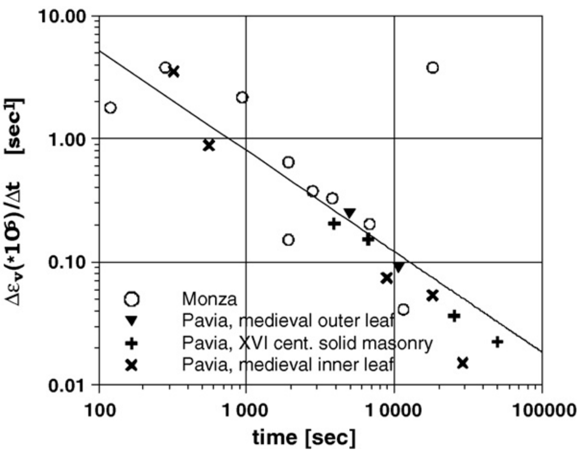

Fig. 3. Strain rate vs. total time of last load step in pseudo-creep tests.

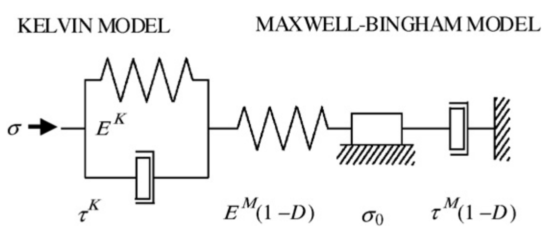

Fig. 4. Rheological model [6].

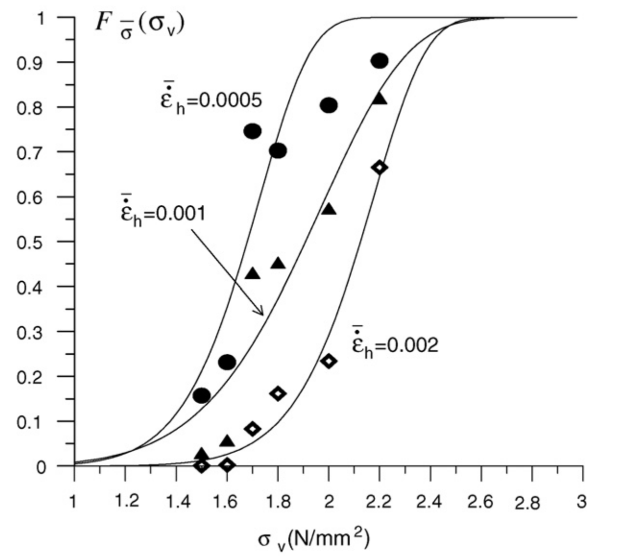

Fig. 6. Horizontal strain-rate: experimental (●) and theoretical (—) fragility curves.

The purpose of the testing activity has been initially the identification of the creep behaviour as a possible cause of the collapse of buildings, then the study of factors affecting creep (rate of loading, stress level...) and the set-up of the most suitable testing procedures to understand the phenomenon, and finally the determination of significant parameters (strain rate of secondary creep phase...) that may be referred to as risk indicators in real structures. In the following, the most advanced results obtained from

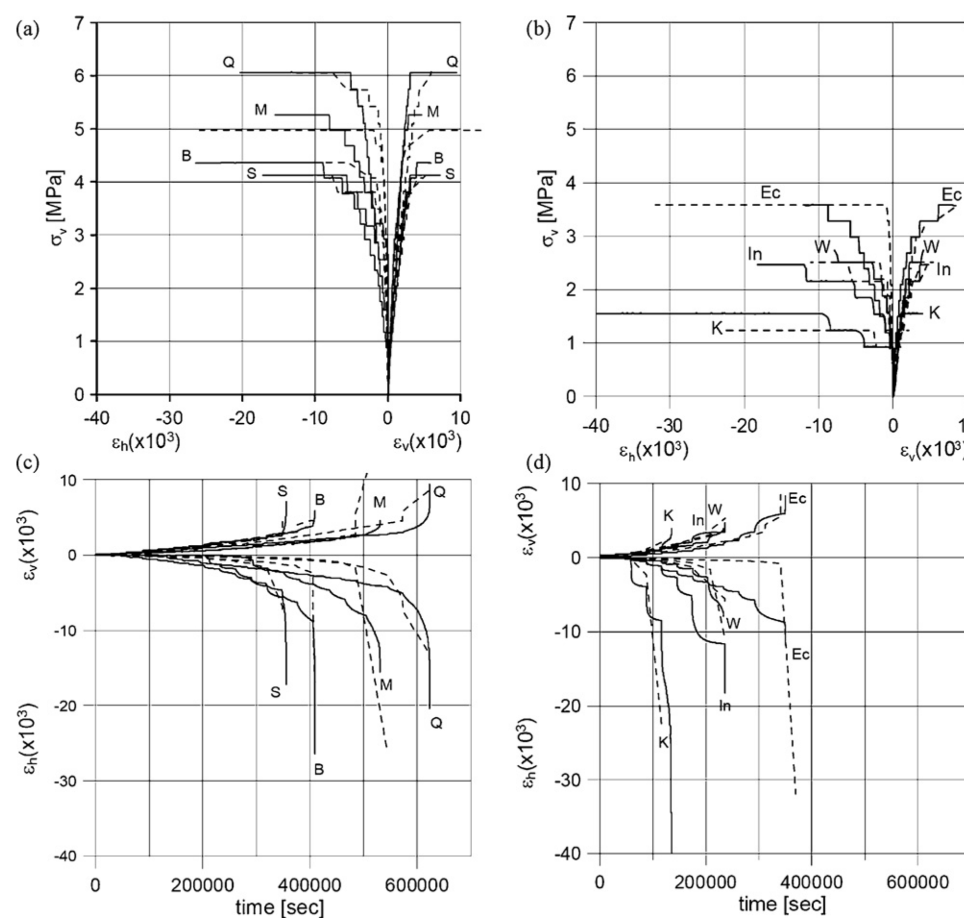

Fig. 5. Numerical (- - -) and experimental (—) results of pseudo-creep tests: (b–d) medieval masonry, (a–c) 16th cent. masonry.

pseudocreep tests are presented. After characterizing the materials by monotonic tests and nondestructive sonic tests, a total of four prisms from the XVIth century plain masonry belfry (Fig. 1b) and four coming from the inner medieval rubble masonry (Fig. 1c) were tested applying subsequent load steps of 0.3 MPa kept constant for intervals of 28,800 s. On the average, higher peak stress and lower strain were registered on the masonry of the belfry. In Fig. 2, the results of a test carried out on the masonry of the inner leaf and those of a test carried out on the masonry of the belfry are respectively shown.

Considering the last load step for each specimen tested with pseudocreep tests, the secondary creep rate, which is the strain rate during the phase of stable damage growth, has been calculated before collapse and then related to the duration of the last load step, which can be regarded as the residual life of the material. In Fig. 3, these values have been plotted comparing the masonry

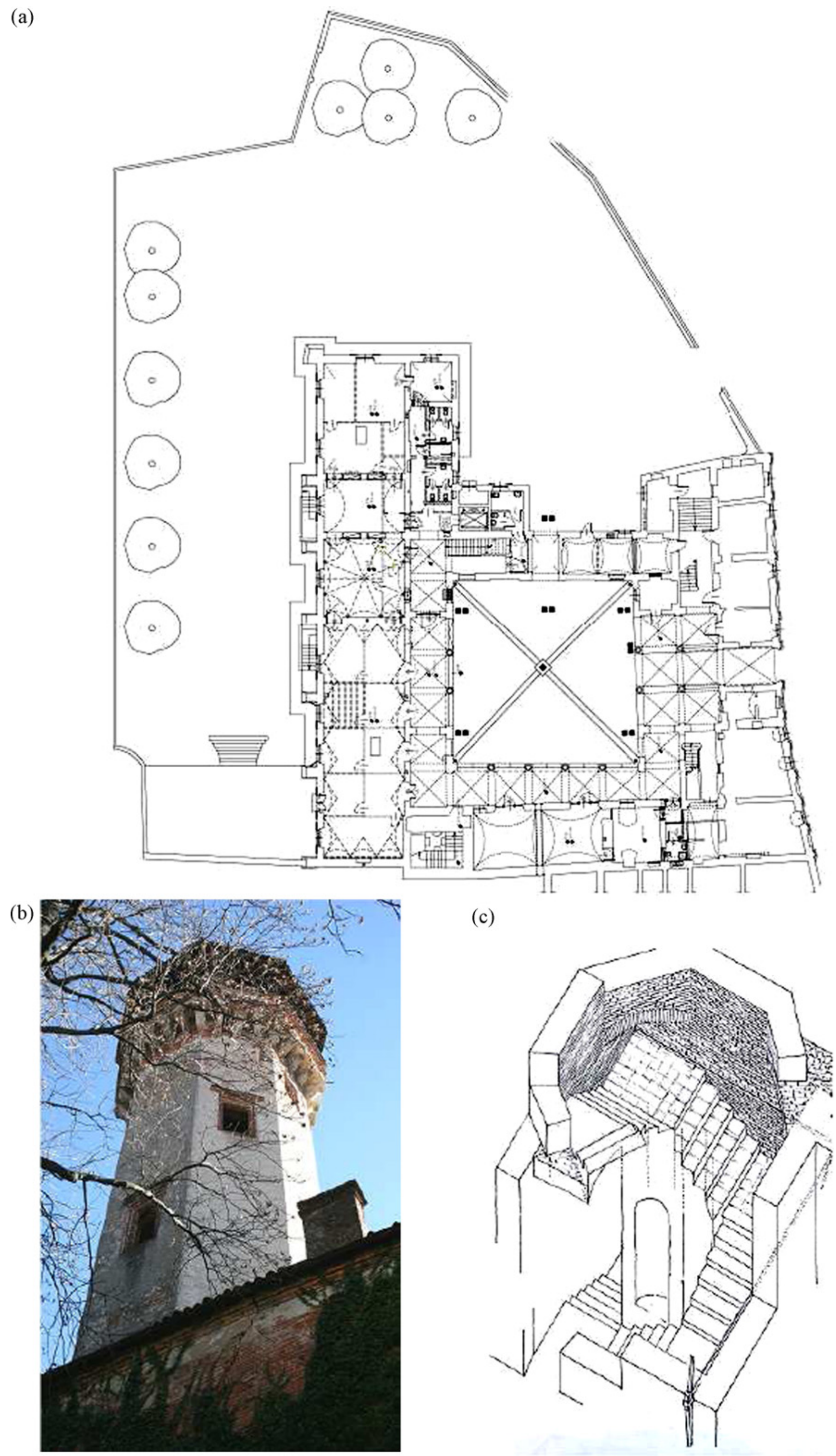

Fig. 7. (a) Plane of Palace Masserano with the Tower; (b) Photographical view; (c) Sketch of the inner part of the tower with the central pillar.

of Monza with those taken from the ruins of the tower of Pavia. Though the number of test results is not particularly large, an interesting inverse relationship can be found, which seems to apply to the two materials considered together, as well as to other brittle materials subjected to creep and fatigue tests. In this respect, a useful comparison with concrete behaviour could be made since the analysed relationship is well known in the case of concrete [5]. A strong correlation exists between creep time to failure and secondary creep rate, which accordingly, can be used as a reliable parameter to predict the residual life of a material element subjected to a given sustained stress. In the following, after presenting a rheological model to interpret the experimental results, a probabilistic approach is proposed to evaluate the critical creep strain rate, in view of preserving the historical heritage.

2.1. Interpretation by a rheological model

The experimental results shown in Fig. 2 have been interpreted by a viscoelastic rheological model shown in Fig. 4 [6]. The model consists of a Kelvin block, accounting for primary creep, connected in series with a Maxwell block, accounting for secondary creep; the stress σ_0 relative to the Bingham element corresponds to the threshold above which secondary creep activates; tertiary creep is accounted for by the introduction of damage variables, governing the decrease in the static and viscous parameters (elastic stiffness and relaxation time of the Maxwell-Bingham element).

Considering a Cartesian reference frame (x_1, x_2, x_3); at the application of a stress increment $\Delta\sigma_{(i)}$ along x_1 during a time interval $\Delta t = t_i - t_{i-1}$, the relevant strain increments along x_1 and in x_2 (and x_3) can be respectively expressed as:

$$\Delta\varepsilon_{1(i)} = \frac{\Delta\sigma_{(i)}}{E_1} + \varepsilon^{in}_{1(i)}, \quad \Delta\varepsilon_{2(i)} = -\nu\frac{\Delta\sigma_{(i)}}{E_2} + \varepsilon^{in}_{2(i)} \tag{1}$$

where $\varepsilon^{in}_{1(i)}$ and $\varepsilon^{in}_{2(i)}$ are inelastic strains. Assuming the material to be isotropic at the virgin (undamaged) state, the apparent stiffness coefficients can be expressed by:

$$\frac{1}{E_j} = \frac{1}{E^M(1-D_j)}\left(1 + \frac{\Delta t}{2\tau^M(1-D_j)}\right) + \frac{1}{E^K}\left(1 - \frac{\tau^K}{\Delta t}\left(1 - \exp\left(-\frac{\Delta t}{\tau^K}\right)\right)\right), \tag{2}$$

(j = 1 or 2).

The inelastic strain accumulated at time t_i is the sum of that stored by the Kelvin element and that stored by the Maxwell-Bingham element: details on the expressions for these strains can be found in [7]. Under constant stress, the damage rate ($\dot{D}^V_j$) is

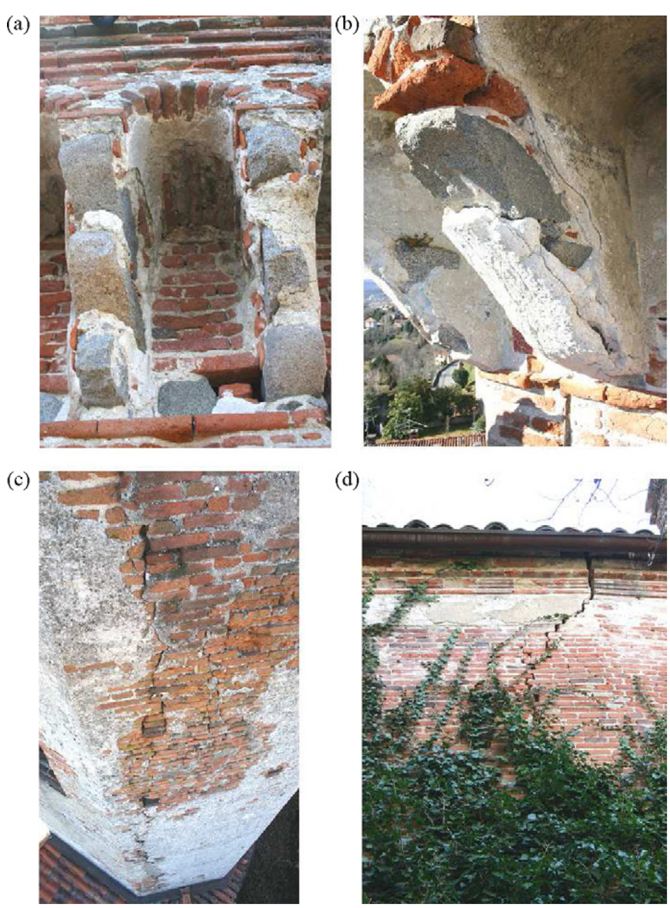

Fig. 8. Crowning of the Tower La Marmora Masserano: (a–b) details of the damage phenomena; (c) Cracks on the North-east wall; (d) Cracks on the North wall of the tower.

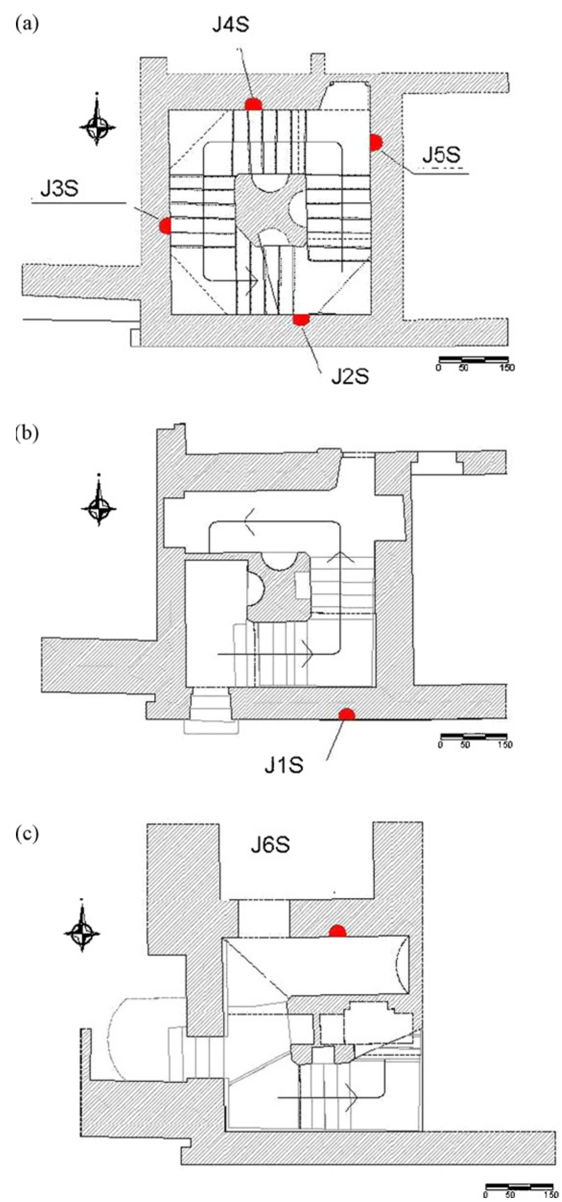

Fig. 9. Position of flat jack tests: (a) at a height of 12 m, (b) at ground floor level, (c) at underground level.

supposed to be given by a law originally proposed by [8] for rock salt:

$$\dot{D}_j^V = \left(Y_j^B\right)^{X_{2H}} X_{1H} D_j \left(\log \frac{1}{D_j}\right)^{\frac{1+X_{1H}}{X_{1H}}} \quad j = 1 \text{ or } 2,\ H = C \text{ or } T. \tag{3}$$

The expression for the 'viscous damage forces' is $Y_j^B = \frac{1}{2}E^M\left(\varepsilon_j^B\right)^2$, similarly to the rigorous definition given by [9] according to thermodynamics, and are associated with the irreversible strain stored in the Maxwell's dashpot (ε_1^B, ε_2^B). If the stress does not exceed σ_0, $\dot{D}_j^V$ is supposed to vanish.

Under increasing stress, the damage rate ($\dot{D}_j^S$) is given by the time derivative of the following law, originally proposed for concrete:

$$D_j^S = 1 - \frac{1}{1 + A_H \left\langle Y_j^M - Y_{0H} \right\rangle^{B_H}} \quad j = 1 \text{ or } 2,\quad H = C \text{ or } T. \tag{4}$$

The 'static damage forces' are defined as $Y_j^M = \frac{1}{2}E^M\left(\varepsilon_j^M\right)^2$ and are associated with the elastic strain in the Maxwell's spring:

$$\varepsilon_1^M = \frac{\sigma}{E^M(1-D_1)},\quad \varepsilon_2^M = -\nu\frac{\sigma}{E^M(1-D_2)}. \tag{5}$$

In general, $\dot{D}_j = \dot{D}_j^S + \dot{D}_j^V$.

The model parameters A_H, B_H, X_{1H}, X_{2H} take different values, according to the sign of the relevant strain ($H = T$ for tension, $H = C$ for compression). In Fig. 5 the experimental and numerical results of the pseudocreep tests on the inner leaf and on the belfry are compared. As expected, the prisms of the inner leaf reached failure well before the prisms of the belfry. The numerical simulation can satisfactorily reproduce the experimental diagrams.

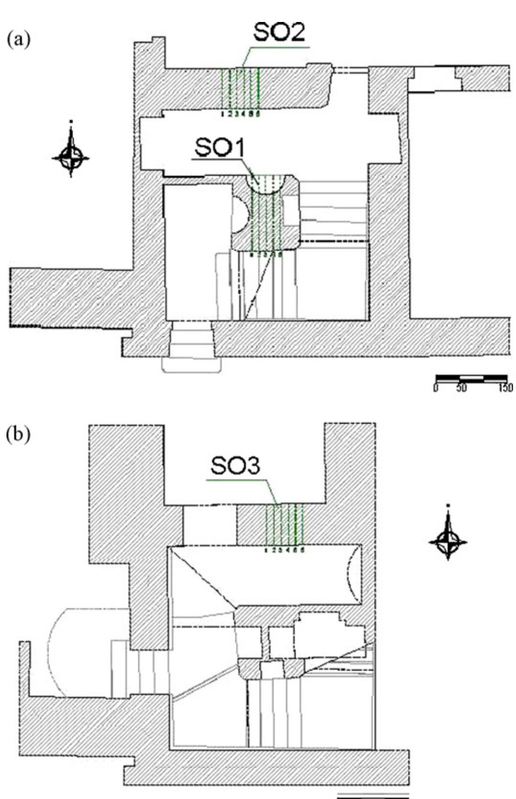

Fig. 10. Position of sonic tests.

2.2. Interpretation by a probabilistic model

The results of the creep and pseudocreep tests have been interpreted also by means of a probabilistic model, based on the definition of a random variable as a significant index of vulnerability, and on the solution of the classic problem of reliability in stochastic conditions. The aim of this approach is to provide a mathematical model able to predict possible failures of heavy masonry structures due to long-term damage, allowing preventive repair interventions [10].

By experimental evidence, the strain evolution connected with a given stress history of a viscous material like a historic masonry can be described through the parameters $\dot{\varepsilon}_v$ and $\dot{\varepsilon}_h$, respectively defined as the vertical and horizontal strain-rate taken on the linear branches of the strain versus time diagrams shown by the specimen at the stress level σ, remaining constant for a certain time interval.

The deformation can be interpreted as a stochastic process of the random variable $\dot{\varepsilon}$, which in turn depends on the stress level σ corresponding to which the deformation is recorded. Therefore, for each stress level σ the strain-rate $\dot{\varepsilon}$ (measured in ε/ sec) can be modelled with a probability density function (pdf) $f_{\dot{E}}(\dot{\varepsilon}, \sigma)$ that results to be dependent on the stress σ and on the strain-rate $\dot{\varepsilon}$.

In view of the preservation of historic buildings from major damage or even failure, it would be very convenient to indicate a critical value of the strain-rate under which the residual life of the building is conveniently greater than the required service life. Here, a conventional value of $\dot{\varepsilon}$ may be assumed as a critical value indicating a safety limit. Consequently, for a given stress level σ^*, the probability to record the critical strain-rate connected with the secondary creep safety limit increases if the strain-rate $\dot{\varepsilon}$ increases. The calculated values allow to plot an experimental "fragility curve" connected to each chosen strain-rate levels (Fig. 6) [11]. Since the experimentally measured strain-rates only refer to a discrete number of stress levels, it would be quite convenient to have a suitable tool capable to predict, though in probabilistic terms, the system behaviour at any stress level. Following this approach, the deterioration process can be treated as a reliability problem; the reliability $R(t)$ concerns the performance of a system over time and it is defined as the probability that the system does not fail during the time t. Here, this definition is extended and $\bar{R}(\sigma)$ is assumed as the probability that a system exceeds a given significant strain-rate $\bar{\dot{\varepsilon}}$ with a stress σ. The random variable used to quantify reliability is $\bar{\Sigma}$ which is the stress to exceed the strain-rate $\bar{\dot{\varepsilon}}$. Thus, from this point of view, the reliability function is given by [12].

$$\bar{R}(\sigma) = Pr(\bar{\Sigma} > \sigma) = 1 - F_{\bar{\Sigma}}(\sigma), \tag{6}$$

where $F_{\bar{\Sigma}}(\sigma)$ is the distribution function for $\bar{\Sigma}$ and represents the theoretical modeling of the experimental fragility curves. In order

Table 1
Results of flat-jack tests.

Test *n*	[N/mm^2]
J1S	0.61
J2S	0.64
J3S	0.11
J4S	0.82
J5S	1.03
J6S	0.54

Table 2
Sonic velocity.

Test *n*	[m/s]
S01	1029.6
S02	771.3
S03	1262.0

to model the experimental fragility curves and to evaluate $F_{\bar{\Sigma}}(\sigma)$, a Weibull distribution has been chosen [11–12].

This type of prediction may allow to evaluate the results of a monitoring campaign on a massive historic building subjected to persistent load and to judge whether the creep strain indicates a critical condition in terms of safety assessment. Of course, the precocious recognition of a critical state will allow to design a strengthening intervention to prevent total or partial failure of the construction.

3. The Tower Masserano of palace Ferrero - La Marmora at Biella

The Masserano Tower at Biella is an octagonal building ending with a crowning decorated with stone brackets; it is included within Palace Ferrero - La Marmora (Fig. 7) and presents a central pillar having a squared base 1.56 m wide and an octagonal upper part of about 1.21 m radius (Fig. 7c). In addition to a superficial decay at the crowning (Fig. 8), the tower shows a diffuse damage on the South, South-East, East and North-East fronts together with vertical cracks on the lower portion of the central pillar. On the North-East front, corresponding to a long vertical crack (Fig. 8c) continuing on the squared lower portion of the tower, an out-of-plane relative displacement can be observed. On the North external wall of the squared base two cracks are present having maximum opening on the upper tip (Fig. 8d), denoting a probable soil settlement on the North and East side of the tower. A simple calculation, based on the effect of the dead load, gives a uniformly distributed state of stress of about 0.8 N/mm^2 at the base of the tower. After a visual inspection by means of a mobile platform, sonic and flat jack tests (Figs. 9 and 10) have been carried out to characterize the masonry from a mechanical point of view [13–19]. The results of flat jack tests (Table 1) show higher stress values on the North and East side of the tower, which seems compatible with the hypothesis of

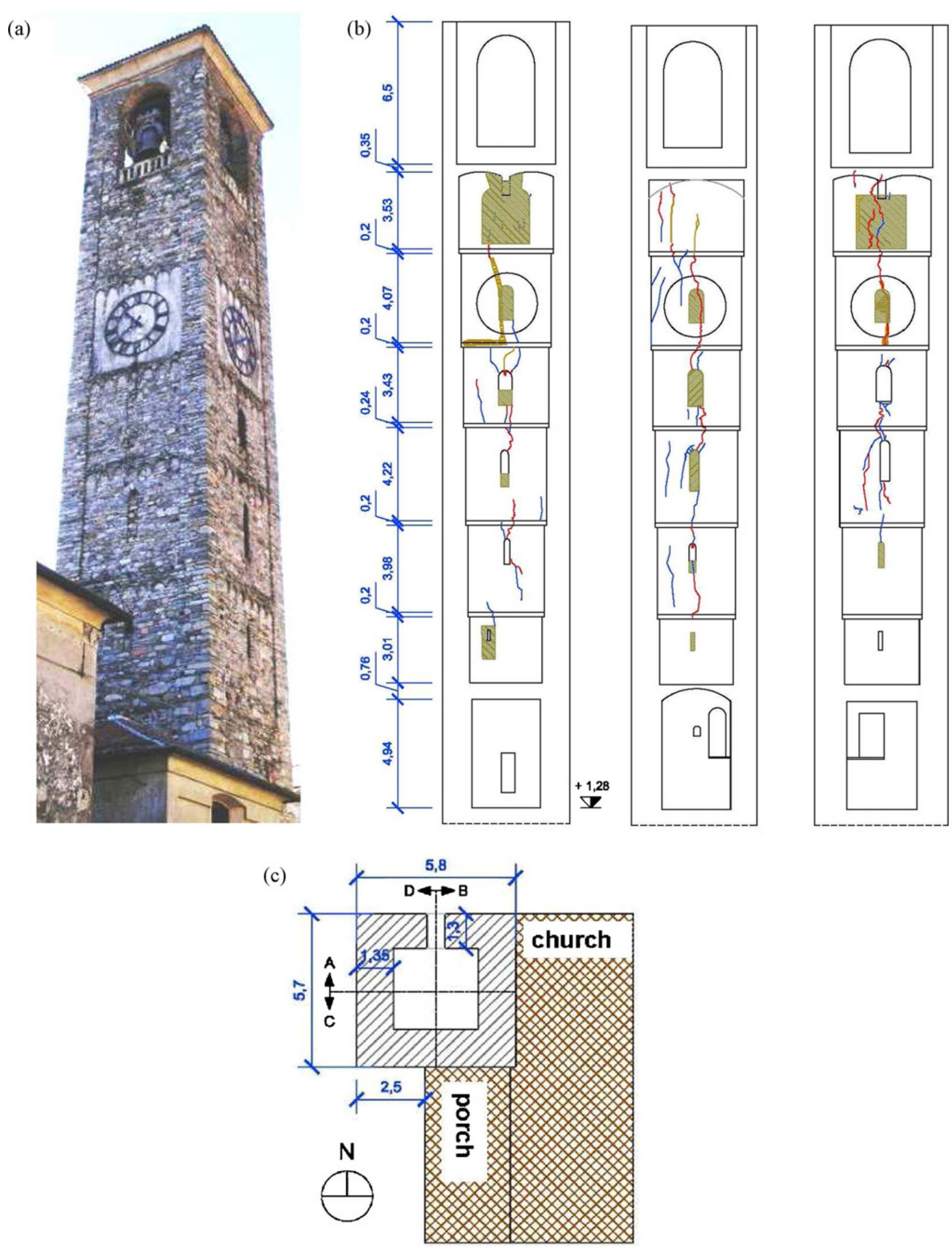

Fig. 11. (a) Bell tower of Arcisate, Crack pattern on the tower sections (b) and plane (c).

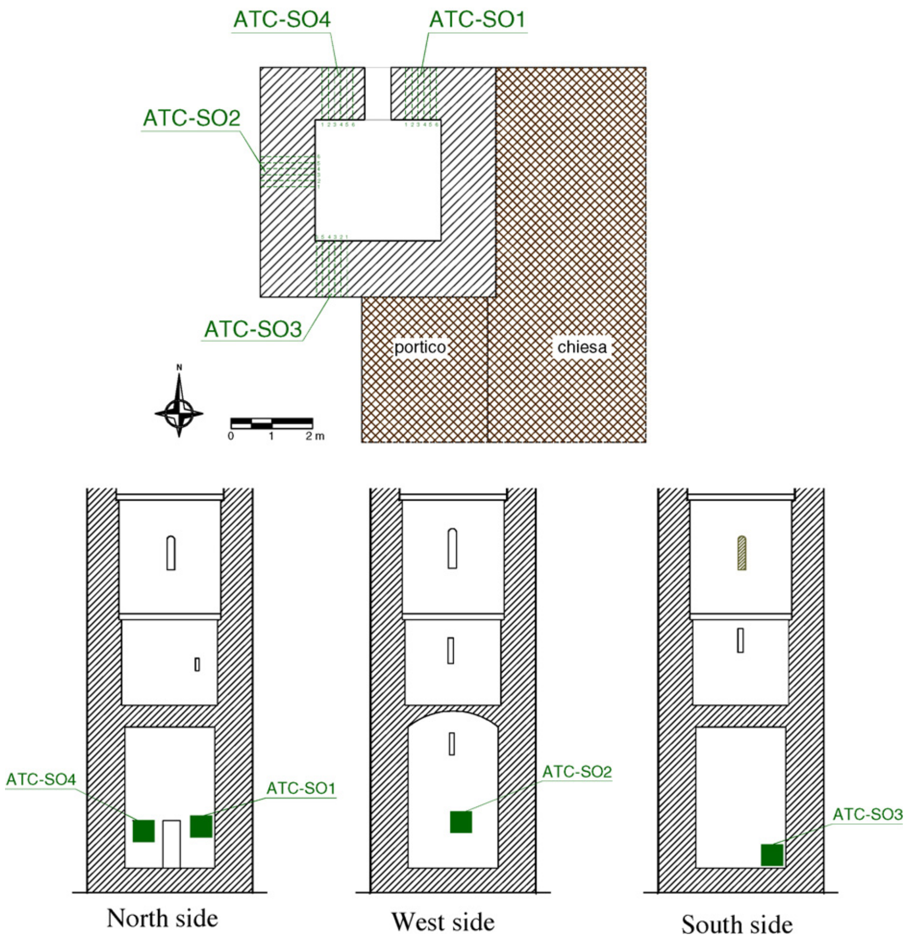

Fig. 12. Tower of Arcisate, position of the sonic tests.

soil settlement above mentioned. The results of sonic tests (Table 2) indicated a particularly low value corresponding to a testing point on the North side. A monitoring program has been designed to record the possible evolution of the opening of the main cracks over a first period of 18 months to achieve an accurate evaluation of the state of damage and a safety assessment of the building [20].

4. The bell tower of the Collegiata of San Vittore at Arcisate

The tower, having roman origins, is a construction about 36 m high, built in stonework masonry on a square base, connected on the East side and partly on the South side to the body of the church (Fig. 11). The survey of the external walls has been useful to reveal the interventions carried out in different periods, including super elevations, closing and opening of windows, damage repairs and repointing. Till a height of 10 m the tower is characterized by an irregular stonework, with no decorations, followed by six orders of floors, five of which defined by masonry offsets at the corners and by corresponding sequences of small hanging arches marking the floor levels. The sixth and highest order corresponds to the addition carried out in the XVIIIth century to host the bell trusses. The crack pattern has been accurately examined also through the help of an aerial platform which allowed to closely inspecting the external wall surface (Fig. 11b). Along the four sides, the tower shows long vertical cracks, most of them cutting the whole wall thickness and passing through the keystones of the arch window openings. They are present between the third order of the tower and the base of the belfry and show a maximum opening corresponding to the upper tip. Many superficial cracks are also diffused, particularly on the North and West front not adjacent to the church. A simple calculation of the state of stress induced by the dead load indicates a value of about 0.86 N/mm^2 [21].

The masonry has been also characterized through sonic tests, as shown in Figs. 12 and 13 and in Table 3. Apart from those corresponding to test S04 on the North front, the results of sonic velocity indicate a relatively compact masonry, of fairly good execution. In view of the design of repair interventions to guarantee the building safety, more information need be collected for a complete diagnosis. In particular, flat jack tests, a structural monitoring and a structural analysis have been planned [22].

5. The medieval towers of Alba

These masonry buildings from the XIIIth century are the tallest and mightiest medieval towers preserved in Alba (Fig. 14). Torre Sineo is square, 39 m high, and leans to a side by about 1% (Fig. 15). Wall thickness ranges from 2 m at the foundation level to 0.8 m at the top. The bearing walls are *a sacco*, i.e., consist of brick faces

Table 3
Sonic velocity m/s.

Test *n*	[m/s]
S01	2184
S03	2053
S03	2239
S04	1623

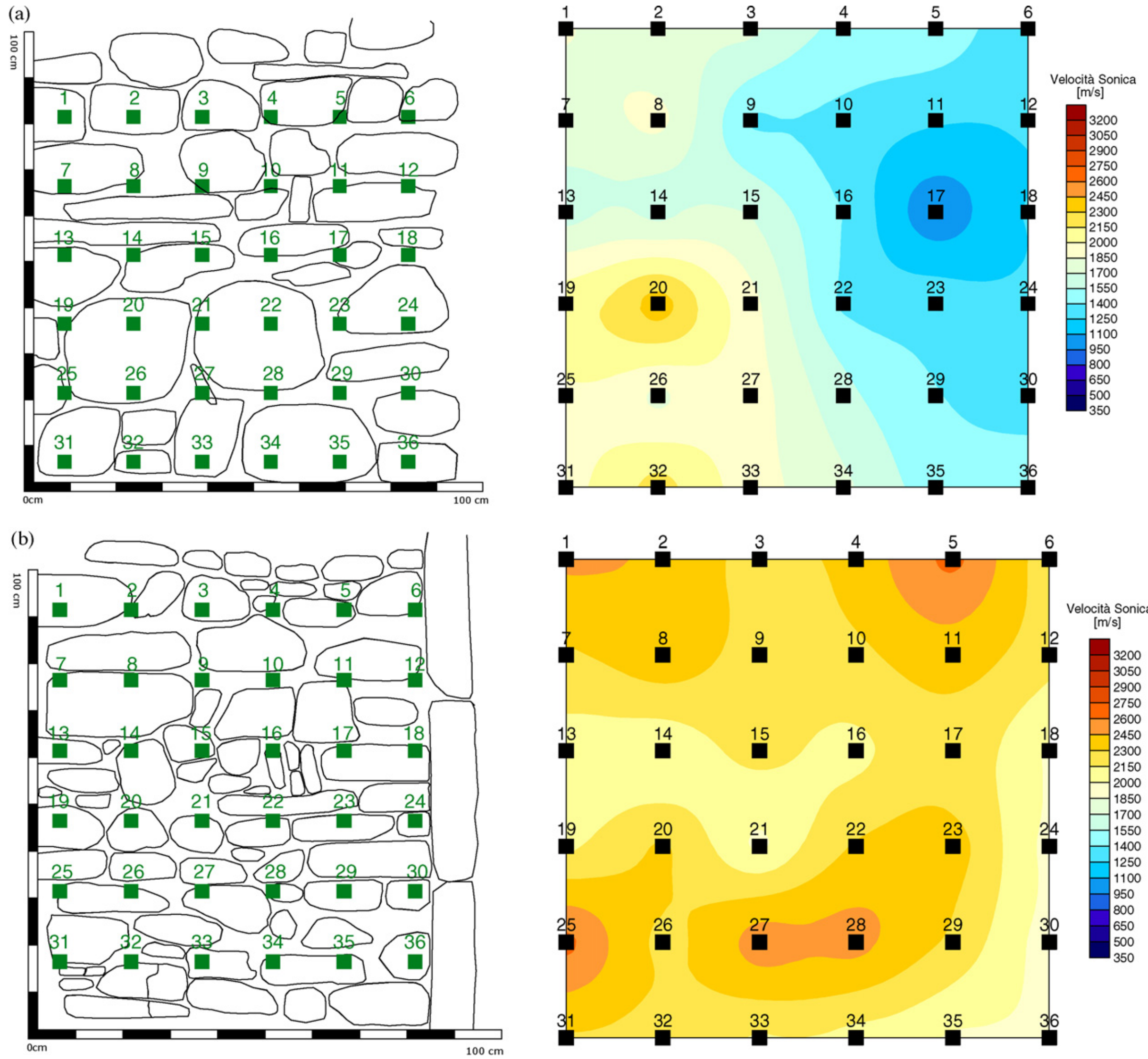

Fig. 13. Tower of Arcisate: (a) Sonic test S04 on the North side; (b) Sonic test S03 on the South side.

enclosing a mixture of rubble and bricks bonded with lime and mortar. Till a height of 15 m, the tower is incorporated in a later building.

Torre Astesiano has a similar structure, but has a rectangular base. The filling material is more organised, with brick courses arranged in an almost regular fashion, however not connected with the outer wall faces. In this case, too, the total thickness of the masonry ranges from 2 m at the bottom to 0.8 m at the top. Total height is about 36 m and the tower does not lean on any side. It is also incorporated in a later building, approximately 15 m high, built when the tower had been completed.

Fig. 14. Overall view of the medieval towers of Alba.

Torre Bonino, just under 35 m high, is the least imposing of the towers analysed. The bearing walls are similar to those of Torre Astesiano, save that the lower storeys are coated with a thick layer of plaster of over 10 cm. The thickness of the walls ranges from 1.8 m at the base to 0.6 m at the top. The square shaped structure has been incorporated in a valuable building from the Italian Art Nouveau period.

The geometry of the towers and the buildings they are embedded in was fully acquired and organised within a CAD system. The positions of the openings and the variations in the thickness of the tower walls were carefully recorded, together with the positions of the main cracks observed in the structures. In particular, the deviation from verticality of the Sineo Tower was evaluated with an optical instrument. One side of the tower leans to the north. Maximum eccentricity, of 39 cm to the north and 3 cm to the west, was measured at the top.

Complete three-dimensional FEM models of the three towers have been built using the CAD drawings meshed with 20-node

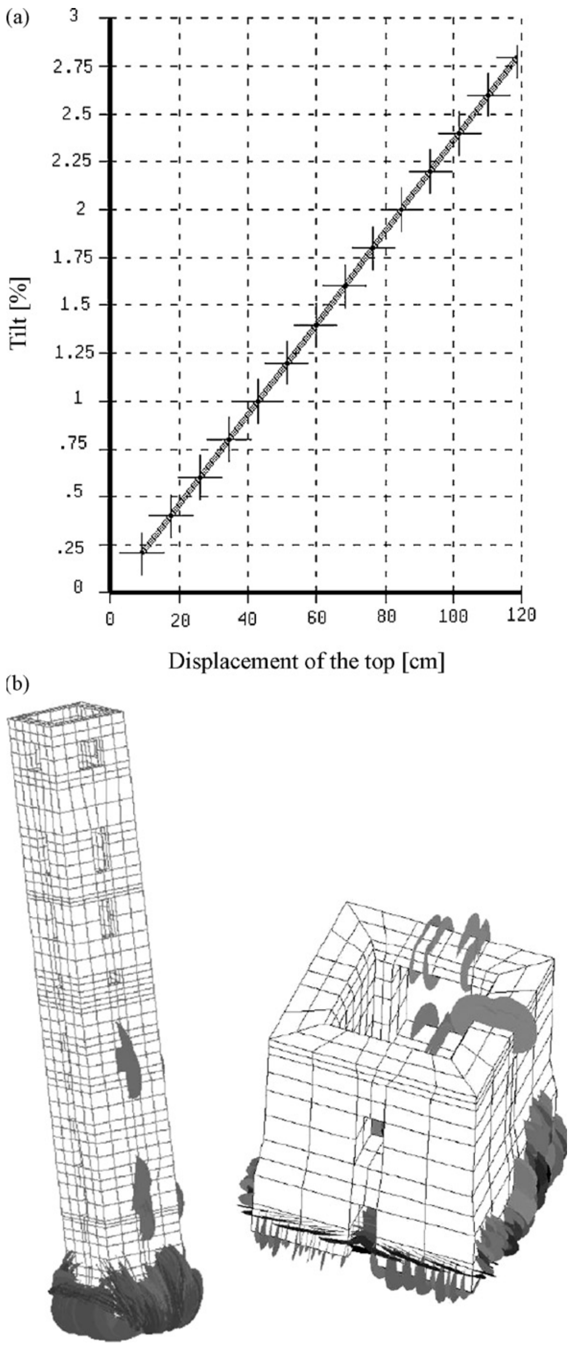

Fig. 15. (a) Evolution of the Torre Sineo top displacement with respect to the tilt; (b) Torre Sineo crack pattern relative to the 3% tilt of the foundation and detail of the crack pattern in the foundation zone.

isoparametric solid brick elements, in order to perform the analysis with the commercial code DIANA. At least five nodes are present in the thickness of the towers wall. The models take into account the presence of openings and the variation of the wall thickness at different levels. On the other hand, the presence of wood floors has been disregarded. Each structure is mainly subjected to its dead load. As far as Torre Sineo is concerned, also the effect of an increasing tilt of the foundation has been considered, combined to the load provided by the wind action exerted to the upper region of the tower (Fig. 15b). The main mechanical parameters of the models have been directly determined by single and double flat-jack tests. Details of the experimental procedure can be found in [23–25].

6. Damage detection in the Alba towers

The cracking processes taking place in some portions of the masonry towers were monitored using the Acoustic Emission (AE) technique. Crack advancement, in fact, is accompanied by the emission of elastic waves which propagate within the bulk of the material. These waves can be captured and recorded by transducers applied to the surface of the structural elements [26–27].

The measurement system used for the AE monitoring consists of eight piezoelectric (PZT) transducers, calibrated on inclusive frequencies between 50 and 500 kHz, and eight control units. The threshold level for the signals recorded by the equipment, fixed at 100 μV, is amplified up to 100 mV. The oscillation counting limit was fixed at 255 oscillations every 120 s [27]. This procedure is referred to as Ring-Down Counting (RDC), where the number of counts is proportional to crack advancement. The number of counts (N) is obtained by determining the number of times that the signal crosses the threshold voltage. In any case, utilising the RDC method and neglecting the material attenuation properties, the AE counting number (N) can be assumed to be proportional to the quantity of energy released in the masonry volumes during the loading process [28–29].

For the Sineo Tower, through AE monitoring, two cracks were detected in the inner masonry layer at seventh floor level (Fig. 16a). The monitoring process revealed an ongoing damaging process, characterised by slow crack propagation inside the brick walls. In the most damaged zone, crack spreading had come to a halt, the cracks having achieved a new condition of stability, leading towards compressed zones of the masonry. In this particular case, it can be seen that, in the monitored zone, each appreciable crack advancement is often correlated to a seismic event. In the diagram shown in Fig. 16a, the cumulative AE function relating to the area monitored is overlaid with the seismic events recorded in the Alba region during the same time period; the relative intensity of the events is also shown [30–31].

A similar behaviour was observed for Torre Astesiano. This structure was monitored by means of two transducers applied to the inner masonry layer of the tower, at the fourth floor level near the tip of the large vertical crack. The results obtained during the monitoring period are summarised in the diagram in Fig. 16b. It can be seen how the damage to the masonry and the propagation of the crack, as reflected by the cumulative number of AE counts, evolved progressively over time. A seismic event of magnitude 4.7 on the Richter scale occurred during the monitoring period: from the diagram we can see how the cumulative function of AE counts grew rapidly immediately after the earthquake.

The monitoring of Torre Bonino was performed the at first floor level, where the effects of restructuring works have affected the masonry most adversely. Under constant loading, a progressive release of energy is observed, due to a pseudocreep phenomenon in the material. A seismic event of magnitude 3 on the Richter scale occurred during the monitoring period (Fig. 16c).

During the observation period, the towers behaved as sensitive earthquake receptors. Thus, as can be seen, the AE technique is able to analyse state variations in a certain physical system and to identify, well in advance, the premonitory signals that precede a "catastrophic" event [32].

The time dependence of the structural damage observed during the monitoring period can also be correlated to the rate of propagation of the microcracks. If we express the ratio of the cumulative number of AE counts recorded during the monitoring process, N, to the number obtained at the end of the observation period, N_d, as a function of time, t, we get:

$$\eta = \frac{E}{E_\mathrm{d}} = \frac{N}{N_\mathrm{d}} = \left(\frac{t}{t_\mathrm{d}}\right)^{\beta_\mathrm{t}}. \quad (7)$$

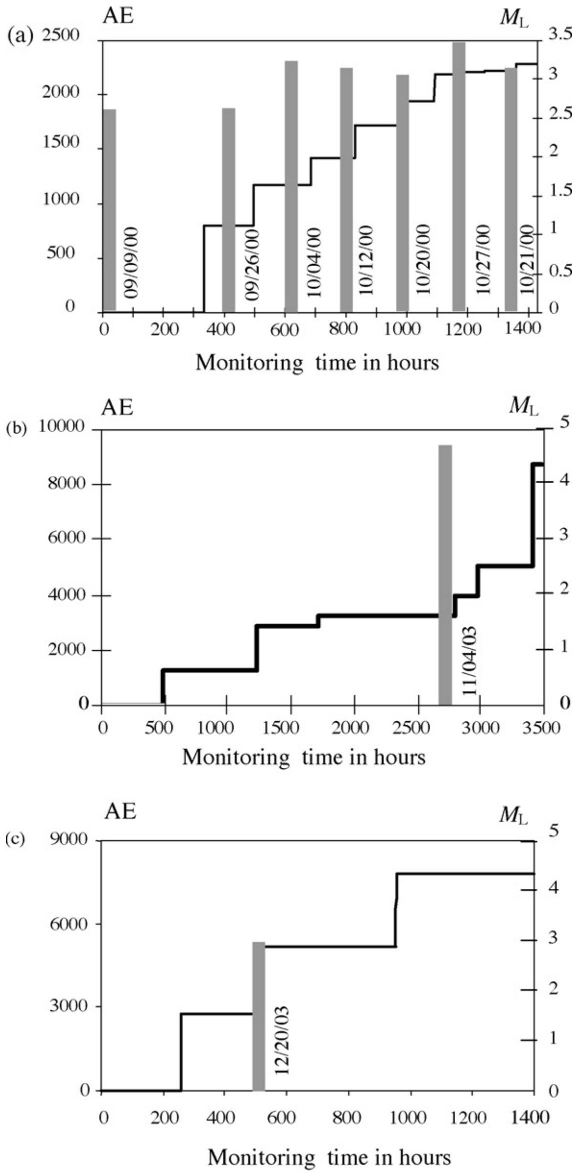

Fig. 16. AE counting number and seismic events in local Richter scale magnitude (M_L): (a) Torre Sineo, (b) Torre Astesiano, (c) Torre Bonino.

In Eq. (7), the values of E_d and N_d do not necessarily correspond to critical conditions ($E_d \le E_{\max}$; $N_d \le N_{\max}$) and the t_d parameter must be construed as the time during which the structure has been monitored. By working out the β_t exponent from the data obtained during the observation period, we can make a prediction as to the structure's stability conditions. If $\beta_t < 1$, the damaging process slows down and the structure evolves towards stability conditions, in as much as energy dissipation tends to decrease; if $\beta_t > 1$ the process becomes unstable, and if $\beta_t \cong 1$ the process is metastable, i.e., though it evolves linearly over time, it can reach indifferently either stability or instability conditions [31–32].

In order to obtain indications on the rate of growth of the damage process in the towers, as given in Eq. (7), the data obtained with the AE technique were subjected to best-fitting in the bilogarithmic plane. This yielded a slope $\beta_t \cong 0.648$ for the Sineo Tower, $\beta_t \cong 1.041$ for the Astesiano Tower and $\beta_t \cong 0.491$ for the Bonino Tower (Fig. 17). These results confirm how the damage process stabilised in the Sineo and Bonino Towers during the monitoring period, whereas for the Astesiano Tower, it evolved towards a metastable condition.

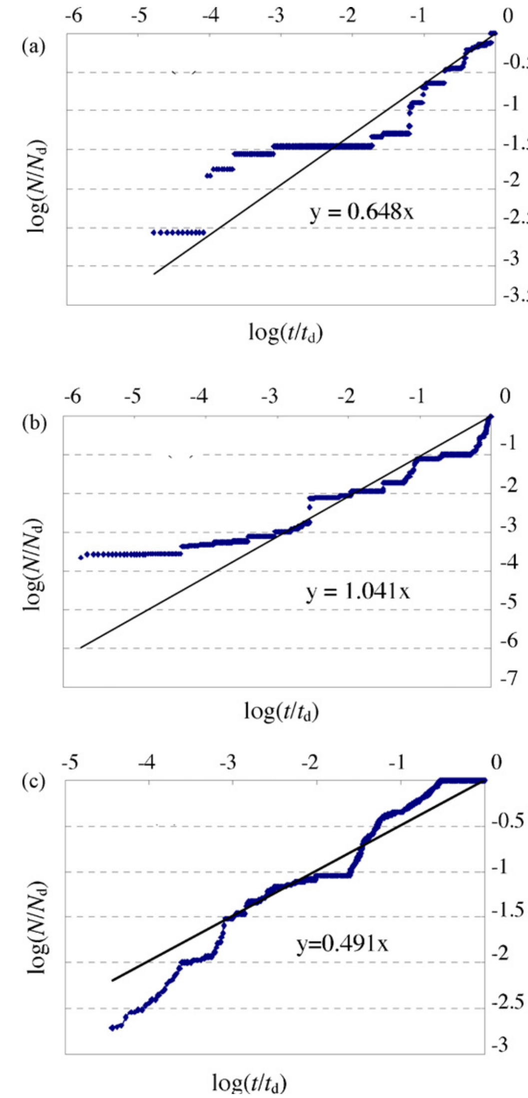

Fig. 17. Evolution of damage: (a) Torre Sineo, (b) Torre Astesiano, (c) Torre Bonino.

7. Conclusions

The research has shown the importance of a global approach in the safety assessment of historic towers. The available information on historic evolution, masonry characterisation and on-site survey can help in the damage interpretation but also give information on the structural analysis.

The failure of monumental buildings is fortunately an exceptional event; it happens generally very rapidly, but it may be avoided if the damage symptoms as cracks, deformations, etc. are carefully taken into account as early as possible. When the safety assessment of towers is required, any risk factor that may affect their integrity has to be taken into account. The following procedure is suggested as a preliminary investigation which may be adopted by public administrations: (i) accurate geometrical survey highlighting the irregular features: lack of verticality, distortions, discontinuities; (ii) survey of the crack pattern and of the damage phenomena visually detected; (iii) interpretation of the crack patterns and recognition of its causes: settlements, effect of dead load, others; (iv) survey of the masonry texture and of the morphology of the wall sections; (v) on-site characterisation of the masonry walls through sonic, flat-jack and radar tests.

The above preliminary analyses can indicate the presence of damage in the structure, and suggest the possible need of more detailed investigations and/or of periodically repeated surveys and finalize future interventions to directly repair the occurred damage and to remove the vulnerability sources. Different approaches are available for the safety assessment, including the Acoustic Emission monitoring technique, rheological and probabilistic models for the interpretation of pseudocreep tests and case studies to predict the remaining lifetime of the highly sustained structural elements. The presented multilevel approach may contribute to sketch guidelines for effective assessment and intervention in the case of masonry towers and other typologies of monumental buildings [33–34].

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