Le antiche torri e i campanili sono particolarmente sensibili agli scuotimenti sismici tanto da aver caratterizzato nei tempi passati i danni sismici di un territorio. Siano essi di pietra o di mattoni, isolati o compenetrati in costruzioni adiacenti, esibiscono danni tipici o meccanismi incipienti di crollo ormai catalogati e studiati. Le normative sismiche di ogni paese prevedono procedure di verifica convenzionale di queste costruzioni per sisma atteso.

Convenzionalità della procedura di verifica con coefficienti di garanzia che partono da quello di confidenza e finiscono su quello cosiddetto di struttura (il quale abbatte le azioni da spettro del sisma atteso sulla base della riserva di duttilità) sono alla base del calcolo. Questo si poggia su dati di input legati alla conoscenza dei parametri meccanici della muratura il cui reale reperimento non risulta codificato.

Per torri isolate l’analisi va condotta con modelli che contengono la deformabilità del suolo. Una semplice procedura utilizza parametri “condensati” in molle elastiche desumibili dalle formule di Gazetas o di Viggiani che tengono anche in conto la profondità del piano fondale. Le prove cross-hole, fornendo le velocità delle onde di taglio nei pressi della fondazione, consentono una migliore stima dei parametri contenuti nelle predette formule.

Il primo e più semplice procedimento di verifica consiste nel “beam model” in cui la torre, se isolata, si considera come una mensola con vincolo elastico alla base. Questo semplice modello consente ovviamente sia analisi dinamiche sia statiche equivalenti con carichi verticali ed orizzontali provenienti dallo spettro del sisma atteso nel sito. Per quanto riguarda la dinamica questo modello fornisce risultati accettabili invece per quanto riguarda le verifiche in relazione alle azioni orizzontali possono considerarsi accettabili solo se la torre ha modeste aperture, altrimenti le singolarità introdotte da queste rendono illusorie le verifiche anche se si tiene in conto la sezione ridotta per le aperture. Anche aspetti dovuti alla presenza o meno di scale solidali in muratura all’interno rendono questa procedura illusoria.

Una procedura più affinata è quella del push-over considerando il materiale non lineare e la discretizzazione ad elementi finiti. In questo caso i parametri del materiale aumentano e il loro reperimento in situ risulta complesso ed è opinabile la relazione fra prove non distruttrive in situ e parametri di input dei programmi FEM.

Nell’articolo accluso sono esposte alcune procedure commentate in relazione ad un caso specifico riguardante la torre di Reno Centese investita dal sisma Emiliano 2012.
ANALYSIS OF ANCIENT TOWERS UNDER SEISMIC SHOCK: A CHECK FACED TO A COLLAPSING BELL TOWER

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Summary
The towers are symbolic monuments that often represent the identity of a community and it is important to avoid the collapse. In this paper, one case history occurred in Italy is presented. The bell tower of Reno Centese (Italy) has been completed in 1883. The bricks used come from a demolition of previous tower in same place. Structural shape is a void square tube with inside timber staircase. During the first shock of the earthquake (May 2012) received a strong damage. The surrounding zone was evacuated. Now, after provisional structural strengthening intervention, the tower is waiting for the definitive restoring work. The collapse was avoided.

Keywords: masonry, tower, collapse, monitoring, diagnostic, strengthening

1 Introduction
The Emilia 2012 earthquake damaged many towers and bell towers in a territory in which these constructions are numerous. A practical problem occured immediately after the quake to avoid the collapse of many seriously injured towers. The intervention technique was different for every single case, traditional or innovative, but this topic is a chapter that need a particular analysis. Here only a brief mention about it is made in reference to a specific case studied. Particular attention we want to give to the problem of analytical and numerical check of the towers in seismic ambient, facing a case in which the severe damage occurred: we can say that the limit state was reached.

The Emilia 2012 Earthquake is now well known as effects on constructions and towers in particular. In this paper particular reference is made to the tower of Reno Centese belonging to the Municipality of Cento in province of Ferrara (IT), it was prone to collapse and was saved by an innovative technique of quick intervention. This was “provisional” intervention but it still the only one that, after two years, allows to have the life in the square where the tower stands.

2 The bell tower of Reno Centese
The isolated bell tower of Reno Centese is located in the square of the village belonging to municipality of Cento, province of Ferrara, Italy (FE). Starting from the bottom, the tower is made by 1) a main prismatic body (square shape) with small circular openings, 2) a belfry and then 3) a spire (Figure 1). The total height of the tower is 29.25m with a constant thickness of the walls (in the trunk) equal to 0.5mm.

The main materials used are: bot piles as foundation, structure made by bricks that come from a demolition of previous tower in same place, external thin plaster as “sagramatura” that is a traditional technique in this geographic area, internal plaster made by traditional lime, stone decorations in the upper part of the tower and timber structures for staircase and internal decks.
3 Main damages after 2012 Emilia’s earthquake

During the night of 20 May 2012, an earthquake with a magnitude of 5.9 occurred in the provinces of Ferrara, Modena and Bologna causing severe damages in particular on masonry towers and bell towers. In the following two months, other seismic events occurred and, in particular, other six events with a magnitude greater than 5.

The tower of Reno Centese, Cento, during earthquake in 2012, was under “cosmetic” restoration so a scaffolding was present at the moment of the shock. An inclined severe crack due to an out-of-plane rotation of the upper part of the bell tower occurred with successive sliding, and also two dislocations appeared in two directions [1]. Other cracks occurred also on north and east sides and the tower was considered by the experts prone to collapse (Figure 2). For this reason a “red area” was identified around, inside which all activities were blocked.

It is important to observe that all the flexural and shear cracks occurred in two separate zones: one located 1,5m above the basement and the other one on the spire.
Figure 2. Main damages after Emilia’s 2012 earthquake: a) diagonal crack (west side), b) dislocation A (20cm), c) dislocation B (6cm), d) cracks (north side)

4 Emergency strengthening intervention avoiding collapse

When the tower was defined by the experts close to collapse, one of the authors proposed a “hard” and “unconventional” emergency intervention, based on the use of both spritz FRC and FRP wrapping. This was an invasive technique, but the tower did not fall down also after other shocks. The final aim of the strengthening interventions was to achieve strength and stability of the bell tower higher than the ones before the earthquake, in order to prevent the structure to a possible successive seismic event.

The starting point was the application of fibre-reinforced projected cement mortar in order to close cracks. A truck with an arm of 52m was used to execute the work in safety conditions. The new mortar produced also a confinement effect on the underlying masonry, because was characterized by higher compressive strength, tensile strength and fracture energy.

Then a double layer of fibers were applied horizontally and vertically, in particular the first layer was made by glass and the second by carbon, to remove the “red area” and to start the restoration works in the internal part of the bell tower (Figure 3).

Figure 3. Strengthening interventions: a) application of spritz FRC, b) FRP wrapping
After the strengthening works, the tower was monitored by Acoustic Emission Technique (AE), and no warnings were perceived during the successive seismic events. Sensors were installed close to cracks. In fact, during crack propagation, the elastic energy is released and produce some waves that are captured by the sensors. In this way, it is possible to monitor the damage process.

5 Seismic analysis after shocks

The analytical-numerical study is “on progress” and, as final task, has to assess if the conventional methods of analysis according to the Italian Codes are able to forecast the damages on a structure for an expected earthquake. The dynamic behaviour of the bell tower of Reno Centese has been investigated by using a dynamic modal analysis with software Strauss7. Mechanical properties chosen are [2]:

- mean masonry compressive strength: $f_{m} = 2.4 \text{MPa}$;
- elastic modulus of masonry: $E_{m} = 3500 \text{MPa}$;
- friction coefficient: $\mu_{m} = 0.4$;
- mean shear strength of masonry in absence of axial force: $\tau_{0} = 0.07 \text{MPa}$;
- masonry density: $\gamma_{m} = 18 \text{kN/m}^3$;
- confidence coefficient: $FC = 1.35$.

In order to perform the computational seismic verification according to the Italian Codes, it is necessary to take into account soil-structure interaction. The analysis are carried out in both fixed and elastic constraint (rotational spring) at ground level.

In the case of elastic constraint, to determine the rotational stiffness of the spring, Gazetas formula has been used [3]. Two different values are considered:

- rotational stiffness of the static spring:
  \[
  K_{a} = \frac{3.6 \cdot G \cdot B^3}{(1 - \nu)} \text{[kNm]} \tag{1}
  \]
  where: $G =$ shear modulus, $B =$ side of the squared foundation, $\nu =$ Poisson coefficient. Under the basis of minimum and maximum values of $G$, two values of $K_{a}$ are obtained and represent the lower and upper bound of the stiffness; these two values are then multiplied by $f_{D}$ coefficient that takes into account the depth of the foundation. At this stage, a mean value is computed that represents the static or maximum value of the stiffness.

- rotational stiffness of the dynamic spring:
  \[
  K_{\text{dynamic}} = K_{\text{static}} \cdot K_{a}(\omega) \text{[kNm]} \tag{2}
  \]
  where: $K_{a}(\omega)$ is a coefficient evaluated starting from the vibration periods $T_{B}$ and $T_{c}$.

Also in this case, a mean value between the two values of dynamic stiffness is considered that represents the dynamic or minimum value of the stiffness.

In the analysis, the following values are used:

\[
K_{\text{static}} = K_{a,\text{max}} = 0.97 \cdot 10^7 \text{kNm}
\]
\[
K_{\text{dynamic}} = K_{a,\text{min}} = 0.76 \cdot 10^7 \text{kNm}
\]

According to the Italian Codes, the behaviour factor $q$ for such masonry structures is 2.8; in the following analysis also $q = 1$ has been considered as reference value.

At the end two different model for the bell tower is studied: a beam model and a plate model (Figure 4). In the beam model, due to the fact that the openings are neglected, only the earthquake in one direction (x-direction) is considered and the behaviour is perfectly symmetric; in the plate model, the earthquake is considered in two directions (x and z directions) because the openings are taking into account and the model is not symmetric yet.
6 Seismic verification and simulation

6.1 Beam model

In order to perform the seismic verification for the beam model, a ratio between capacity and demand is evaluated along the height of the tower. The worst case, as expected, is the one characterized by fixed constraint (\( K_\alpha \to \infty \)) and behaviour factor q equal to 1 (Figure 5).

![Figure 5. Capacity/Demand for the case \( K_\alpha \to \infty \): a) bending moment, b) shear force](image)

![Figure 6. Capacity/Demand for the case \( K_{\alpha_{\text{min}}} \): a) bending moment, b) shear force](image)
In Figure 7, the two areas (the first 1.5m above the basement and the second on the spire) in which all flexural and shear cracks occurred, are underlined. It can be seen that these areas are not so far to the ones predicted by using beam model.

Figure 7. Damages on the bell tower: a) due to bending moment, b) due to shear

In order to perform the seismic verification for the plate model, the Mohr-Coulomb capped and cut-off criterion is used. Considering the cohesion \( c = 0.75 \text{MPa} \) and an internal friction angle \( \varphi = 16.17^\circ \), and knowing the following relations

\[
f_c = \frac{2c \cos \varphi}{1 - \sin \varphi}; \quad f_s = \frac{2c \cos \varphi}{1 + \sin \varphi}
\]

the Mohr-Coulomb circles are drawn for each plate in terms of stresses in principal directions.

The worst case, as expected, is the one characterized by fixed constraint (\( K_x \to \infty \)) and behaviour factor \( q \) equal to 1 (Figure 8, Figure 9).

Figure 8. Mohr-Coulomb circles for earthquake in x-direction, \( K_x \to \infty \), \( q=1 \)
At the end the envelopes of the Mohr-Coulomb circles for the two limit cases \( K_{\alpha} \to \infty \) and \( K_{\alpha,\min} \) are reported (Figure 10).

\[
K_{\alpha} \to \infty
\]

\[
K_{\alpha,\min}
\]

7 Conclusions

In this paper the Authors want to underline the significant contribution given by the emergency strengthening interventions to Reno Centese tower proposed by one author. The tower was defined by a group of experts prone to collapse; applying spritz FRC and FRP wrapping, the failure was avoided. Another advantage was the possibility to start the restoration works in the internal part in a safety conditions. However, the invasive nature of the technique represents its main shortcoming. From the computational point of view, a crack pattern coming from the numerical simulations is very close to the one occurred on the bell tower after the Emilia’s 2012 earthquake.

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