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COLLAPSE ANALYSIS OF SANTO STEFANO TOWER USING APPLIED ELEMENT METHOD

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Abstract

During the 2009 L'Aquila earthquake, the tower of Santo Stefano di Sessanio, the town's iconic symbol, collapsed to the ground. The tower was believed to have been rendered vulnerable to collapse because of 20th-century renovations to the tower's observation platform, which replaced a wooden deck with one made of reinforced concrete, thus making the tower top-heavy. In the current study, a numerical analysis was used for investigating the collapse behavior of the tower. Doing an initial elastic model of the tower did not allow any realistic prediction of the anomalous seismic behavior showed by the tower; the reason why only a stump of structure actually survived the earthquake remained unexplained. The collapse of the Medici tower is then investigated using a full 3D Applied Element Method ("AEM"). AEM is a discrete crack approach method in which the structure behavior till its collapse is followed. Two models for the structure of the tower were considered in this study. In the first model, the walls were modeled as regular brick elements neglecting the random real shape of the stones of the tower, while in the second model the real random shape of the stones was considered. Fully nonlinear dynamic analysis is carried out including the hysteretic constitutive models of the stones and connecting mortar. Damage obtained from regular elements analysis was somehow overestimated compared to the real damage. On the other hand, the real mesh analysis showed better results closer to reality.

Keywords: Heritage Masonry Structure; 3-D Applied Element Model; Nonlinear Dynamic Analysis.

1. Introduction

L'Aquila's territory (Italy) is rich in medieval walled villages recognized as main historic and artistic sites. In April 6, 2009, a devastating earthquake struck the city of L'Aquila and the surrounding villages, causing more than 300 fatalities and thousands of injuries. Earthquake caused extensive and severe damage to the territory's valuable real estate heritage including eminent churches, important palaces, a variety of monumental buildings and structures, civic and bell towers. The research work presented in this paper focuses its attention on the collapses of the medieval tower of Santo Stefano di Sessanio village, one of the most beautiful in the entire Abruzzo region. In particular, linear and nonlinear seismic analyses of the tower have been carried out to investigate the anomalous seismic behavior of the structure and the reason why only a stump of it survived the earthquake. The insertion of a concrete floor which was performed during the Second World War on top of the tower is supposed to have affected the seismic capacity negatively. Such kinds of interventions are considered to be non-compliant with the conservation criteria according to current standards and ancient masonry structures are currently retrofitted with alternative techniques.

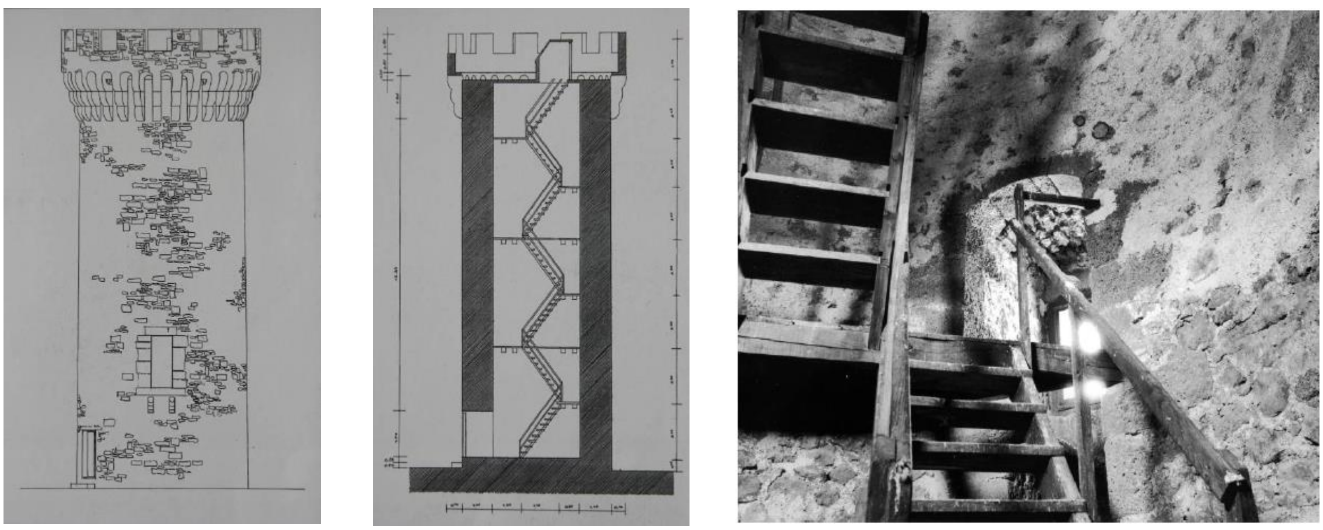
2. Motivations

With a magnitude MI=5.8, April 6, 2009 earthquake represented the main shock of a long lasting seismic sequence including more than 30 minor earthquakes (3.5<MI<5.0) and several thousand of lower magnitude events [1]. Hypocenters were characterized from being relatively close to the surface (depths ranging from 9 to 15 km respectively) and epicenters were located into an ellipse-shaped region measuring about 15 km in length and 5 km in width, centered on the L'Aquila town and parallel to the Apennines mountain chain. Looking back at the historical seismicity of the area, according to CPTI04 catalog several earthquakes comparable in magnitude to the earthquake of April 2009 struck the town in the past, these main events being characterized by an estimated Richter magnitude as high as Me = 6.5, 6.4 and 6.7 in years 1349, 1461 and 1703 respectively [2]. Geologists state this seismic activity to be the result of a normal fault movement on a NW-SE oriented structure which is part of the 800 km long segmented normal fault system running all along the Apennines mountain chain [3, 4, 5, 6]. The same fault system is also considered responsible for previous recent earthquakes, in Umbria-Marche (1997) and Molise (2002). L'Aquila' seismic sequence never ended, and new minor events yet continue to be recorded. Currently, engineers and scientist are concentrating a number of investigations to address the geological characterization of the sites and the assessment of the damage evolution of the survived buildings and structures. In this study, the natural record of the April 6th 2009 main shock has been considered for running dynamic analyses of the tower. Site effects were taken into account with a proper modification of the seismic input.

3. Description of the Tower

Santo Stefano village develops all around and below the medieval tower, which is located in elevated, eccentric position with respect to the historic core of the village (Fig. 1a). The tower has a circular cross section characterized from an internal diameter of about 3.89 m and a relevant wall thickness measuring about 1.50 m. From inside, the vertical development of the tower measures about 17.35 m. From outside, the overall height of the tower reached about 20 m. In additions to the tower entrance, which is located at the very bottom of the construction, two small openings are found along the cylindrical body of the tower. One is located at the height of about 3.80 m from the bottom of the tower and the second consists of a narrow window set closer to the tower top end. The lower opening is surrounded by regular stone blocks of average dimension. Two stones stand like cantilevers on the left and right sides of this opening (Fig. 1b) and were probably part of a medieval drawbridge system. The narrow window on top of the tower is located in the opposite direction compared to the entrance opening. On top of the tower, a crown of cantilevers sustains a crenellated parapet. Before April the 6th, 2009, the tower was open to people to could access the tower crown by means of the internal wooden staircase. These two flights of staircase were very narrow flights with a steep slope of about 116% stopping on wooden landings repeating every 4.60m in height (Fig. 2 a, b, c) During the Second World War a concrete floor was built on top of the tower to use it as antiaircraft station. To exit the tower and access the crenellated parapet a small opening was opened in this floor.

Fig. 1 – a) A picture of Santo Stefano village; b) the Tower.



a) b) c)

Fig. 2 – a) Drawing of the tower elevation; b) drawing of the vertical section of the tower; c) picture of the internal volume of the tower and view of the wooden staircase

3.1 After the Earthquake

In April the 6th, 2009, the tower did not resist the strong earthquake and collapsed almost completely. Today, only the bottom part of the construction remains, consisting of a stump of variable height along its perimeter. This stump presents its shortest side where the bottom window was set (about 3.80 m) and the tallest side (measuring about 13 m) right where the top window used to be. To prevent more piece of wall from falling and to reproduce the original profile of the tower, scaffolding and other temporary shoring has been set after the earthquake main shock (Fig. 3).

Fig. 3 – From left to right: the tower before and after the 2009 earthquake; scaffolding and temporary shoring.

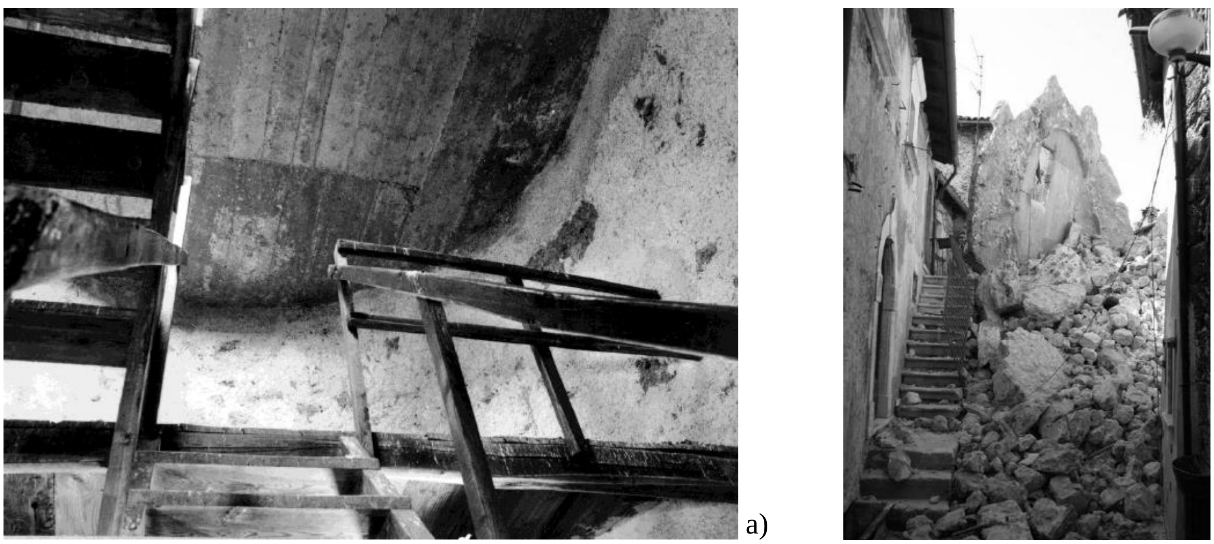
3.2 Historical Notes

Actually, the history of the tower corresponds to that of the village of Santo Stefano di Sessanio, considered to be one of the most beautiful in Abruzzo (Italy). The first settlements of the village date back to the period of the Middle Ages. Actually, the historian Anton Ludovico Antinori (1704-1778) stated he found the words "Santo Stefano" in documents dating back to the year 1195. Originally, the tower was built as a cylindrical body with no crowning on top. The entrance used to be set at an elevated position and probably served by a drawbridge system. Since 1350, due to changes in the military techniques and to disastrous events, some modifications were done to the original tower shape. It is during the Angioins domination's period that the tower is doted of its crenellated parapet to serve as a real defense presidium for the village. Since 1550, regular maintenance was operated on the tower, with restoring interventions carried out as consequence of earthquakes and wars. Starting from 1791, the Santo Stefano village and its tower have been frequently represented in several historical drawings and maps (Fig. 4).

Fig. 4 – Drawings of Santo Stefano village on historical maps.

Most recently, during the '80s, the Office of the Superintendence to the Monumental Goods documented a diffuse cracks pattern and other damages related to aging. Pictures in Fig. 5 show the severe degradation suffered by the external surface of the wall characterized from loss of stones and cementing material, especially from top of the tower. Specific restoring interventions were carried out. As mentioned earlier, during the Second World War a concrete platform was built on top of the tower to use it as antiaircraft station (Fig. 6a). Surprisingly (or probably not!), this rigid, heavy concrete slab separated from the tower and felt down almost untouched as consequence of the April 2009 earthquake (Fig. 6b).

Fig. 5 – The severe degradation of the tower as recorded during the 80's.



a) b)

Fig. 6 – a) Picture of the concrete slab as seen from inside the tower; b) the concrete slab ruined on the ground as a rigid, whole single piece.

4. Numerical Simulation of the Towers Dynamic Response

Unfortunately, no experimental testing has been ever carried out on the masonry wall of the tower to determine current material properties and strength of the original masonry structure. At the current time, after the tower's collapse, operating double flat jack and other kind of in situ investigation on the remaining stump of the tower has become unfeasible and risky. However, accurate details concerning the tower geometry are available from measurements operated in the past. This made it possible to prepare a numerical computer model of the tower that was used for running both linear and non-linear analyses of the structure. Mechanical properties assigned to masonry are summarized in Table 1. These values are those suggested by Italian Rules and Recommendations with respect of a chaotic masonry typology [7, 8]. In fact, all across the wall structure (measuring about 1.5 m in thickness) the original masonry texture corresponds to irregular, sometimes almost round stone elements used in combination with a poor lime-clay mortar.

Table 1 – Mechanical properties assigned to masonry

Compressive Strength f_m (N/mm^2)	τ_0 N/mm^2	E N/mm^2	G N/mm^2	Unit weigth kN/m^3
1.4	0.026	870	290	19

First, an elastic analysis was carried out to evaluate the modal properties of the tower and to estimate the stress concentration and displacements in the structure when subjected to gravity and service loads and to seismic loads as well (response spectrum analysis) (Fig. 7). Unfortunately, the elastic model of the tower did not allow any realistic prediction of the anomalous seismic behavior showed by the tower so that the reason why only a stump of structure actually survived the earthquake remained unexplained.

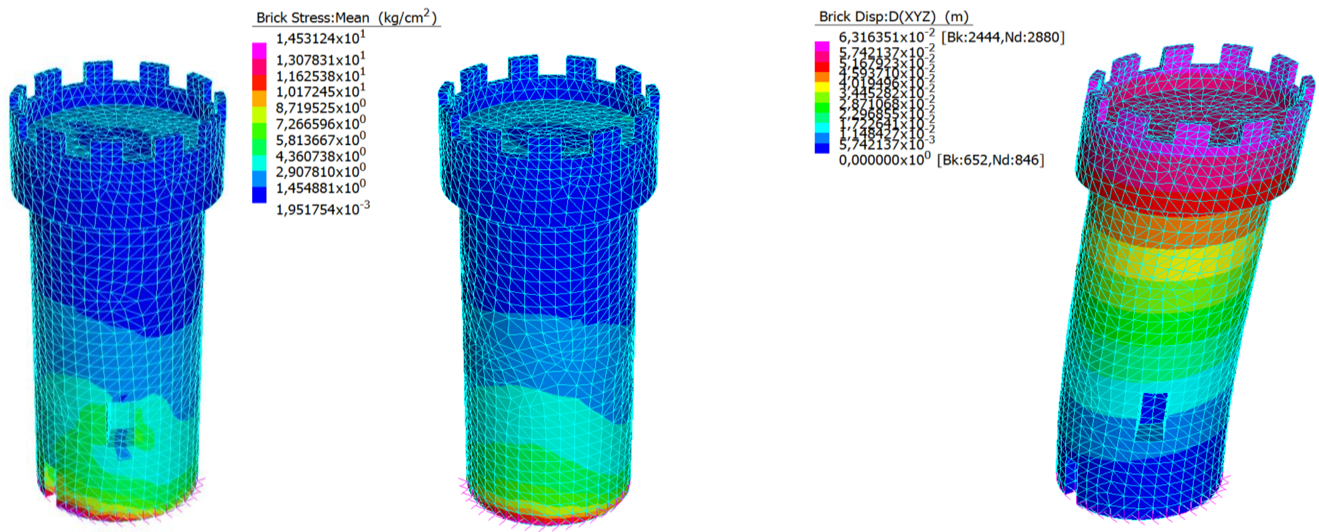


Fig. 7 – Results from the elastic analysis of the tower.

5. The Applied Element Method (AEM)

In this study, the Extreme Loading® for Structures (ELS) software [11], which is based on the AEM, is used. The AEM was initially developed at the University of Tokyo in 1998 by Tagel-Din and Meguro (2000, a,b) solving a two-dimensional plane stress problem. Later on, in beginning of current century, it was expanded to solve three-dimensional problems with the ELS software. The AEM is an innovative modeling method that adopts the concept of discrete cracking. In AEM, structures are modeled with element assembly, as shown in Fig. 8a. The elements are connected along their joint surfaces through a set of normal and shear springs. Those springs are responsible for transfer of normal and shear stresses among adjacent elements. Each spring represents stresses and deformations of a certain volume of the material, as shown in Fig. 8b. Each two adjacent elements can be completely separated once the springs connecting them fail. Fully nonlinear path-dependent constitutive models are adopted in the ELS software, as shown in Fig. 8c. For concrete in compression, an elasto-plastic and fracture model is adopted Maekawa and Okamura (1983) [12]. When concrete is subjected to tension, the linear stress-strain relationship is adopted until cracking, where the stresses degrade to zero. Since AEM adopts a discrete crack approach, the reinforcing bars are modeled as bare bars for the envelope (Okamura and Maekawa, 1991[13]), while the model of Ristic et al. Ristic et al. [14](1986) is used for the interior loops. AEM is a stiffness-based method, in which an overall stiffness matrix is formulated and the equilibrium equations including each of stiffness, mass and damping matrices are nonlinearly solved for the structural deformations (displacements and rotations). The solution for equilibrium equations is an implicit one that adopts a dynamic step-by-step integration (Newmark-beta time integration procedure) (Bathe, 1995 [15], Chopra, 1995[16]).

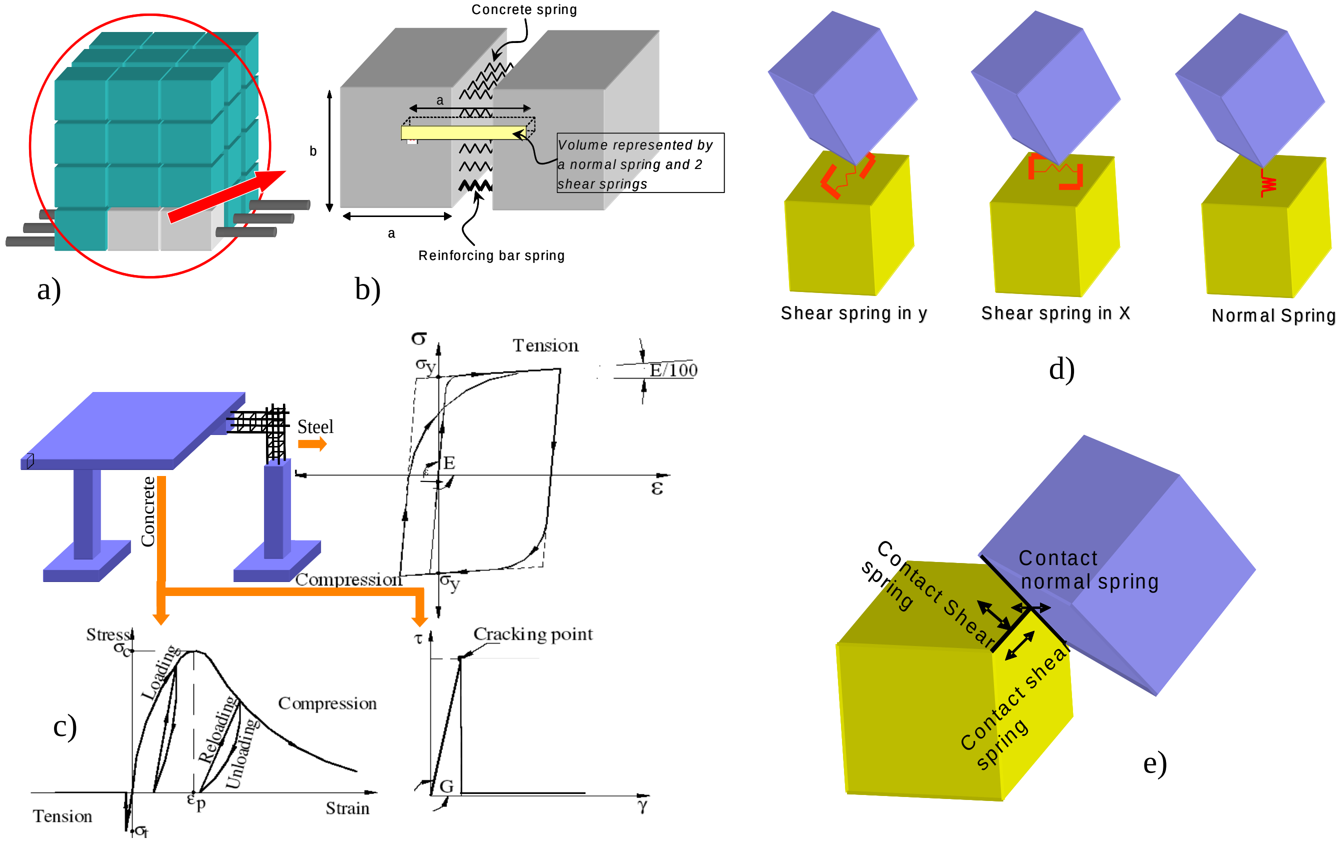


Fig. 8 – Schematic modeling of a structure with the AEM: a) Element generation for AEM; b)Spring distribution and area of influence of each pair of springs; c) Examples of constitutive laws; d) Corner-to-face or corner-to-ground contact; e) Edge-to–edge contact.

In AEM, two neighboring elements are separated from each other if the matrix springs connecting them are ruptured. Elements may automatically separate, re-contact or contact other elements. Figures 8d) and e) illustrates the different types of element contact, where contact springs are generated at contact points.

The AEM was proven to be capable of following the deformations of a structure subjected to extreme loads to its total collapse[17-32] Therefore, since the goal of the current study is to investigate the behavior of the historical building under severe seismic action, it was decided that the AEM is the most appropriate numerical tool for such an investigation

6. Structural Model of the Tower

Two models for the structure of the tower were considered in this study. In the first model, the walls were modeled as regular brick elements neglecting the random real shape of the stones of the tower, as shown in Fig. 9, while in the second model the real random shape of the stones were considered as shown in Fig. 10. The top slab was modeled as a reinforced concrete slab. The total number of elements in the model is 50,000 elements with 300,000 degrees of freedom.

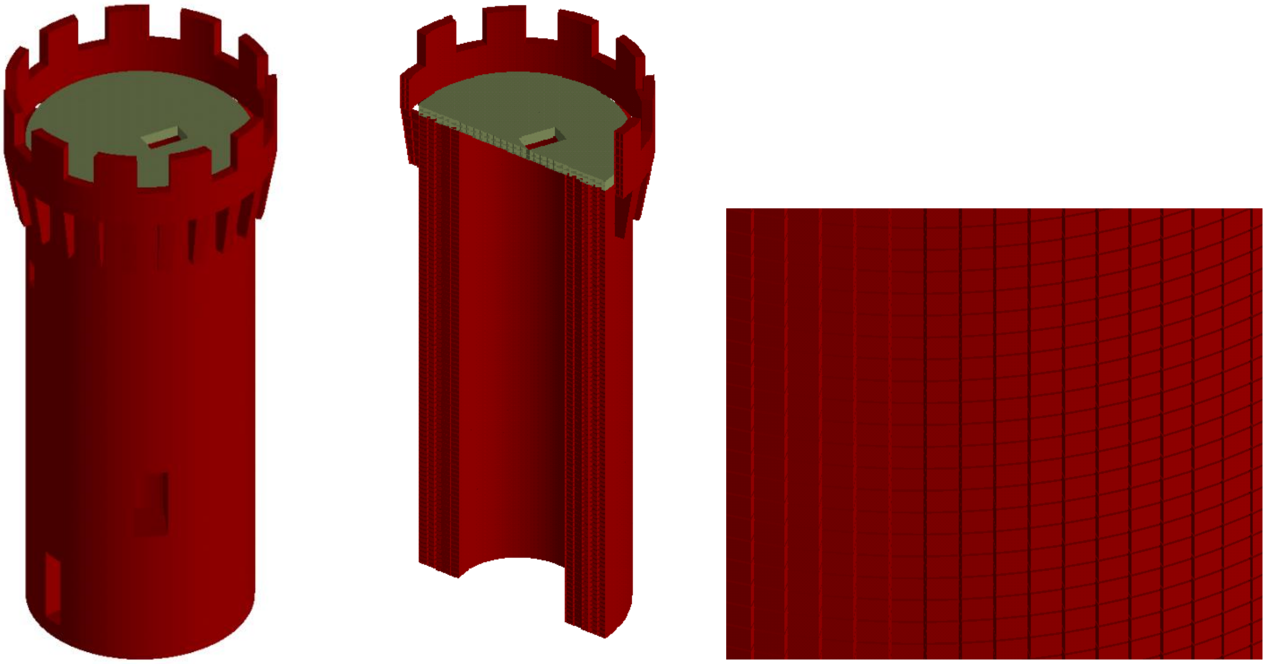


Fig. 9 – Simplified structural model of the tower with regular brick elements

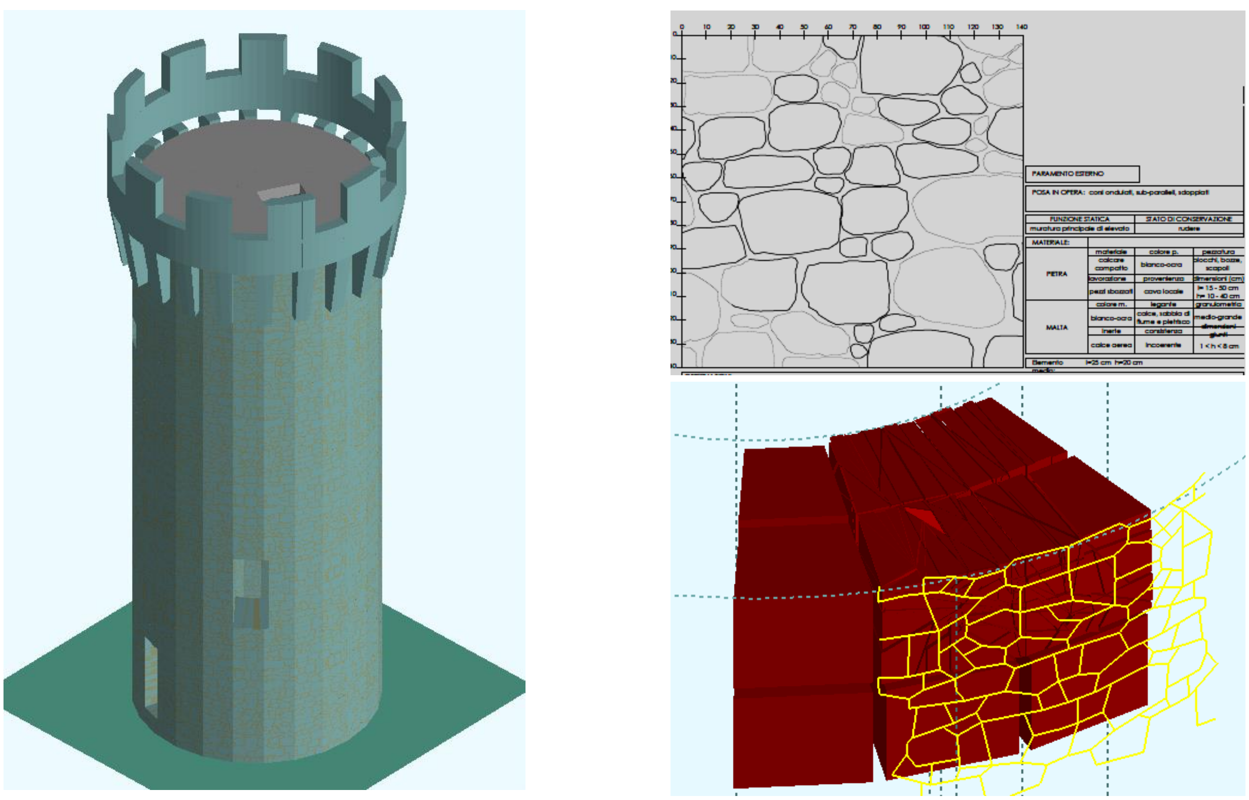


Fig. 10 Advanced structural model of the tower with irregular brick elements

The mechanical properties of constituting materials of the tower are shown in Table 2. The earthquake record used in analysis is shown in Fig. 11. The accelerograms used were selected among a group of 7 different records found to be in agreement with the response spectrum compatibility criterion.

Table 2 – Mechanical properties of constituting materials

Material	Brick	Concrete	Steel
Yield stress (MPa)	----	----	360
Tensile strength (MPa)	0.3	2.0	504
Compressive strength (MPa)	3	25	504
Young’s modulus (MPa)	2500	22130	200000
Shear Modulus (MPa)	1000	10500	81556

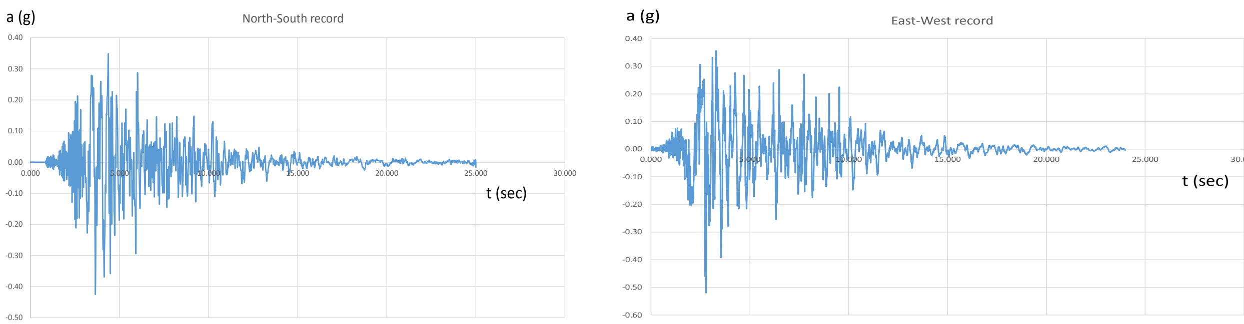
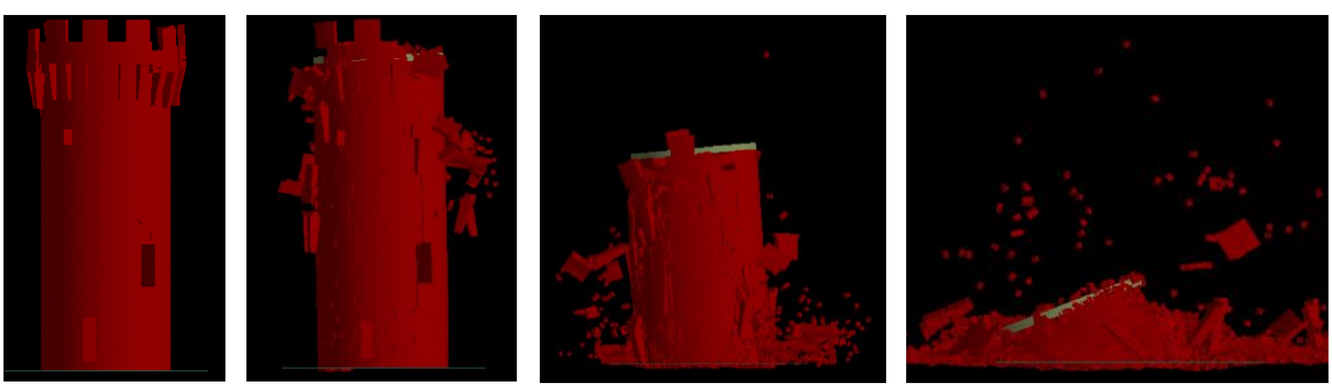


Fig. 11 – Earthquake records used in dynamic analysis.

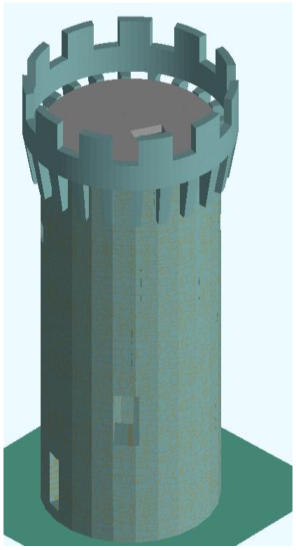
7. Analysis, Results, and Discussion

The application of the earthquake loading to the tower showed a total collapse of the tower as shown in Figs. 12 and 13, for both regular and real mesh patterns, respectively. Failure was initiated by inclined cracks starting at the corners of the openings as shown in Fig. 14. The major cracks that caused the collapse are very close to the ones observed in reality as shown in Fig. 14.

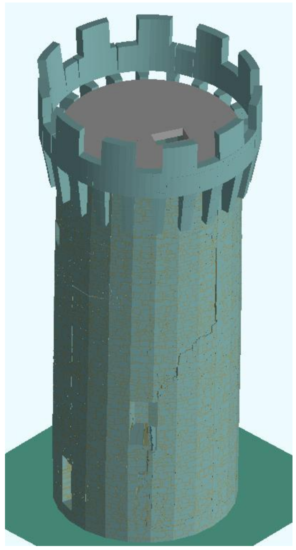


Time 1.9 sec Time 2.9 sec Time 3.9 sec Time 5.9 sec

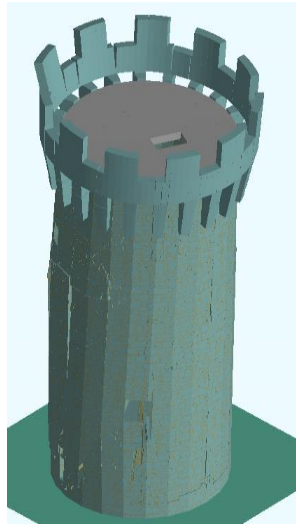
Fig. 12 – Analytical results using the AEM regular elements.



Time 1.0 sec



Time 3.4 sec



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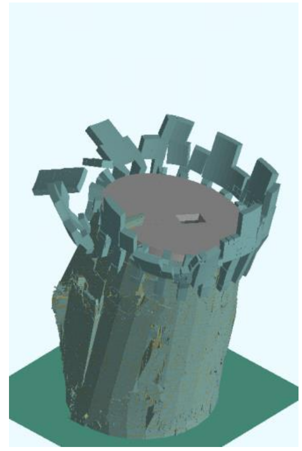
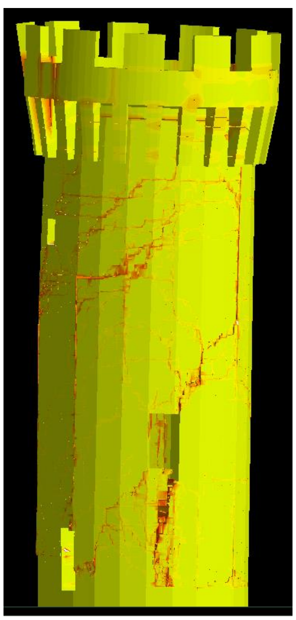
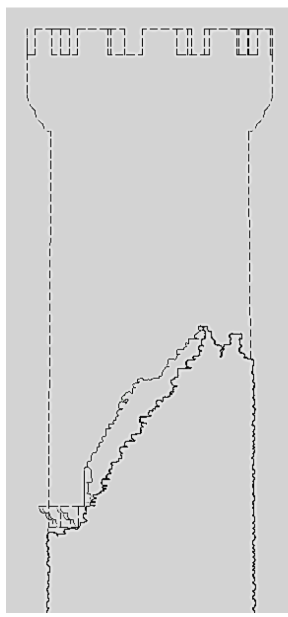


Time 5.0 sec

Fig. 13 – Analytical results using the AEM irregular elements.



Predicted cracks



Observed cracks

Fig. 14 – Major inclined cracks that initiated the collapse of the tower (comparison of real observed cracks and cracks predicted by AEM analysis).

Damage obtained from both element configurations was somehow overestimated compared to the real damage, especially for the regular mesh. In the current study, the reasons for such a discrepancy are not known, but it could be due to many uncertainties in the input data. For example, the real shape of stones especially along the thickness of the wall was assumed regular in the current analysis. Also, the soil could have a significant effect in dissipating part of the earthquake energy. This effect is not considered in the current study and the walls of the towers are assumed totally fixed to the ground, which is not true. A third possibility could be the random nature of material strength of the brick and the mortar of the walls. A further study to refine the analytical results is necessary and would be carried out in future to account for the above mentioned factors.

8. Summary and Conclusions

The collapse of the tower of Santo Stefano di Sessanio is investigated using the Applied Element Method. A fully nonlinear time-domain dynamic analysis is carried out including the hysteretic constitutive models of the stones and connecting mortar. The analytical results were able to accurately model the initiation of the major cracks that caused the collapse of the structure. The analytical results showed a complete collapse of the tower due to the seismic loads. Damage obtained from regular elements analysis was somehow overestimated compared to the real damage. On the other hand, the irregular mesh analysis showed better results closer to reality. The discrepancy could be due to some material and geometrical uncertainties as well as neglecting the soil-structure interaction which could be a source of seismic energy dissipation. A further study would be carried out to refine the model.

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References

[1] Ameri G, Augliera P, Bindi D, D'Alema E, Ladina C, Lovati S, Luzi L, Marzorati S, Massa M, Pacor F, Puglia R (2009): Strong-motion parameters of the Mw=6.3 Abruzzo (Central Italy) earthquake. INGV sez. di Milano-Pavia, Italy Report. http://esse4.mi.ingv.it.

[2] Gasperini P, Camassi R, Mirto C, Stucchi M (2004): Catalogo Parametrico dei Terremoti italiani. INGV report. http://emidius.mi.ingv.it.

[3] INGV 2009a. Istituto Nazionale di Geofisica e Vulcanologia - L'Aquila seismic sequence - April 2009. http://www.ingv.it.

[4] INGV 2009b. Rilievi geologici di terreno effettuati nell'area epicentrale della sequenza sismica dell'Aquilano del 6 aprile 2009. http://www.ingv.it.

[5] Chioccarelli E, Iervolino I (2009): Direttività ed azione sismica: discussione per l'evento de L'Aquila. *Proc. XIII ANIDIS Congress, Bologna (Italy), June 2009*.

[6] Chioccarelli, E.; De Luca, F. & Iervolino, I. 2009b. Preliminary study of L'Aquila Earthquake ground motion records v.5.20. *ReLuis Report, http://www.reluis.it.*

[7] Italian Standards and Rules: D.M. 14 Gennaio 2008, Nuove Norme Tecniche per le Costruzioni .

[8] Italian Recomandations: Circolare 02 febbraio 2009 n° 617/C.S.LL.PP..

[9] Applied Science International, LLC (ASI) www.appliedscienceint.com.

[10] Maekawa K, Okamura H (1983): The deformational behavior and constitutive equation of concrete using the Elasto-Plastic and Fracture model. *Journal of the Faculty of Engineering, The University of Tokyo (B)*, **37**(2): 253-328

[11] Okamura H, Maekawa K (1991): Nonlinear analysis and constitutive models of reinforced concrete. Gihodo, Tokyo.

[12] Ristic D, Yamada Y, Iemura H (1986): Stress-strain based modeling of hysteretic structures under earthquake induced bending and varying axial loads, *Research report No. 86-ST-01, School of Civil Engineering, Kyoto University*, Kyoto, Japan.

[13] Bathe K (1995): *Solution of equilibrium equations in dynamic analysis*. Prentice Hall, Englewoods Cliffs, N.J..

[14] Chopra A (1995): *Dynamics of structures: Theory and applications to earthquake engineering*. Prentice Hall, Englewoods Cliffs, N.J.

[15] Tagel-Din H, Meguro K (2000 a): Applied Element Method for dynamic large deformation analysis of structures, *Structural Engineering and Earthquake Engineering. Journal of the Japan Society of Civil Engineers (JSCE)*, **17**(2), 215-224.

[16] Meguro K, Tagel-Din H. (2001): Applied Element Simulation of RC Structures under Cyclic Loading, *ASCE*, **127**(11), 1295-1305.

[17] Tagel-Din H (2002): Collision of structures during earthquakes. *Proceedings of the 12th European Conference on Earthquake Engineering*, London, UK.

[18] Meguro K, Tagel-Din H (2003): AEM used for Large Displacement Structure Analysis, *Journal of Natural Disaster Science*, **24**(1), 25-34.

[19] Tagel-Din H, Meguro K (2000, b): Applied Element Method for simulation of nonlinear materials: theory and application for RC structures, *Structural Eng./Earthquake Eng., International Journal of the Japan Society of Civil Engineers(JSCE)*, **17**(2), 123s-148s.

[20] Tagel-Din H, Meguro K (2000, c): Applied Element Method for dynamic large deformations analysis of structures, *Structural Eng./Earthquake Eng., International Journal of the Japan Society of Civil Engineers (JSCE)*, **17** (2), 215s-224s.

[21] Sasani M, Sagiroglu S. (2008): Progressive collapse resistance of hotel San Diego, *Journal of Structural Engineering*, **134** (3), 478-488

[22] Salem H, El-Fouly A, Tagel-Din H (2011): Toward an economic design of reinforced concrete structures against progressive collapse, *Engineering Structures*, **33**, 3341-3350.

[23] Park H, Suk C, Kim S (2009): Collapse modeling of model RC structures using the Applied Element Method, *Journal of Korean Society for Roc Mechanics, Tunnel& Underground Space*, **19** (1), 43-51.

[24] Helmy H, Salem H, Tageldin H (2009): Numerical simulation of Charlotte Coliseum demolition using the Applied Element Method, *USNCCM-10 Conference-Ohio-USA.*

[25] Helmy H, Salem H, Mourad S (2012): Progressive collapse assessment of framed reinforced concrete structures according to UFC guidelines for alternative path method, *Engineering Structures*, **42**, 127–141.

[26] Helmy H, Salem H, Mourad S (2013): Computer-Aided Assessment of Progressive Collapse of Reinforced Concrete Structures according to GSA Code, *J. Perform. Constr. Facil.*, 529-539.

[27] Sasani M (2008): Response of a reinforced concrete infilled-frame structure to removal of two adjacent columns, *Engineering Structures*, **30**, 2478–2491.

[28] Wibowo H, Reshotkina S, Lau D (2009): Modelling progressive collapse of RC bridges during earthquakes, *CSCE Annual General Conference*, GC-176-1-11.

[29] Salem H, (2011): Computer-aided design of framed reinforced concrete structures subjected to flood scouring, *Journal of American Science*, **7**(10), 191–200.

[30] Salem H, Helmy H (2014): Numerical investigation of collapse of the Minnesota I-35W Bridge, *Engineering Structures*, **59**, 635–645.

[31] Salem H, Mohssen S, Kosa K, Hosoda A (2014): "Collapse Analysis of Utatsu Ohashi Bridge Damaged by Tohoku Tsunami using Applied Element Method, *Journal of Advanced Concrete Technology*, **12**(10), 388-402.