SAFETY ANALYSIS OF THE BELL TOWER OF S. MARIA MAGGIORE CATHEDRAL IN GUARDIAGRELE (ITALY)

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ABSTRACT
The study of historical building goes beyond the modern principles of earthquake engineering and requires on one side a good knowledge of the building techniques at the time of construction and of the building transformations over the centuries, on the other an in depth survey of the building. While macroscopic analytical methods are key to a first assessment of the structure failure mechanisms, advanced modeling tools are very useful, though they require great care by the analyst as the building materials are highly heterogeneous and brittle in the case of historical masonry structures.
This paper presents the results of the static and seismic safety assessment analysis of an ancient 19-m high bell tower erected in the XIII century and located in Guardiagrele in the Chieti Province, Abruzzo, Italy.
An extensive in situ experimental campaign was conducted to identify the material properties and the exact geometry of the tower as well as the stratigraphy of the supporting soil. The tower natural frequencies and mode shapes are extracted from ambient vibration data using state-of-the-art system identification techniques. The onsite investigation results was used to calibrate a 3D finite element model of the tower developed using the program ABAQUS, HKS inc. In this model, the foundation soil is modeled explicitly in order to study the superficial soil layer filtering of the ground motion input at the base rock and its effects on the structural response. Realistic nonlinear constitutive models for cyclic loading are used for the structural and soil materials. The results indicate that the response of the tower is greatly influenced by the supporting soil.

KEYWORDS: seismic vulnerability assessment, nonlinear analysis, cultural heritage, masonry structures, soil structure interaction

1. INTRODUCTION

One of the most challenging tasks in the seismic safety assessment of an ancient monument is its structural evaluation. The geometry and the composition of the load bearing walls and of the horizontal elements, the mechanical properties of all structural components and the static and dynamic interaction among them need to be investigated and well understood. The challenge in the case of the bell tower of the cathedral of Guardiagrele (Chieti Province, Italy) is further complicated because the monument was built with heterogeneous materials and was erected in several construction phases between the end of the XIII century and the XX century, as shown in Figure 1.
Since the tower was build in different centuries it is reasonable to expect that the data obtained from in situ tests will have a high variability. The masonry tower is 20 meter high above the ground level and is made of stones from the nearby Majella mountain and lime mortar. The floors are made of barrel vaults and the vertical structure, including the foundations, is made of two thick outside stone walls, filled in the middle with an irregular masonry of very poor mechanical properties. The tower is shown in Figure 2.
This paper presents the results of an ongoing study partially presented in 2008 by Camata et al.. The objective of this work is to investigate the influence of several parameters on the structural static and seismic response. In particular this paper focuses on the influence of the soil flexibility and on the effects of the interactions between the bell tower and the cathedral walls that restrain the tower in its lateral displacements.

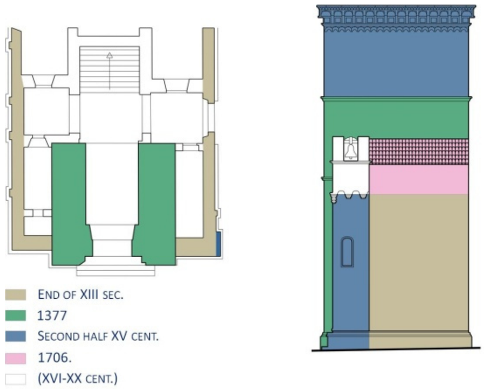


Figure 1 Bell tower construction phases

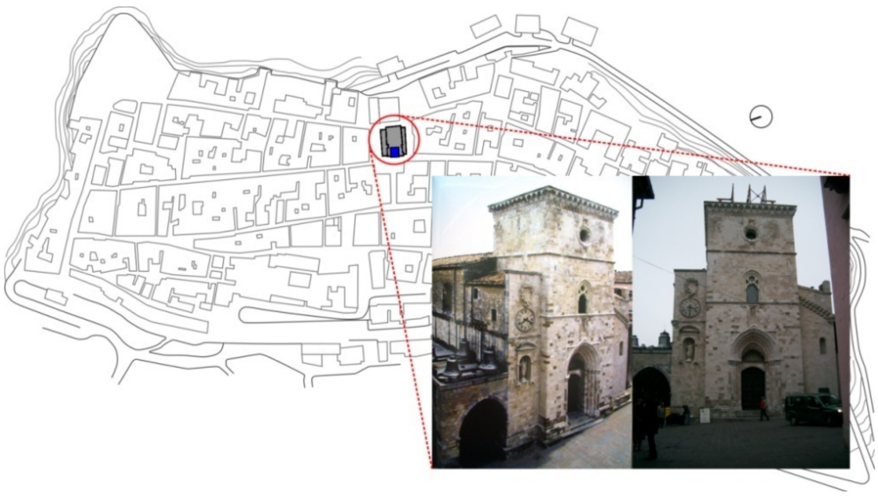


Figure 2 Views of the tower and location

2. EXPERIMENTAL CAMPAIGN

The on-site tests, as well as the structural and architectonic survey, allowed a comprehensive knowledge of the bell tower geometry. The experimental campaign included the determination of the masonry working stress with the use of single flat jacks and several core drills. The latter was used to extract material for laboratory testing as well as to explore the masonry structure through the wall thickness using endoscopy techniques.
The structural walls are made of three-leaf Majella stone masonry and lime mortar. The wall thickness is variable and ranges between 2 meters and 2.80 meters. The two external leaves have an average thickness of 80 cm and are made of compacted regular Majella stone blocks, whereas the infills are made of irregular masonry that includes gravel, cobblestones and mortar. The endoscopy revealed the presence of several voids in the infills.
The tower floors are carried by barrel vaults. The first floor barrel vault structural thickness is 35 cm at the key-stone. The vault consists of regular Majella stone blocks and is filled with 12 cm incoherent material. The second floor vault is similar to that of the first floor, but has a structural thickness of 28 cm. The tower top consists of a reinforced concrete bidirectional slab added in recent years. The floor and the walls below the slab are particularly damaged because of water infiltration and consequent chemical degradation of the materials. In recent centuries the tower top hosted a heavy pitched roof that was destroyed during WWII bombing.
In order to identify the structure dynamic properties, accelerometric data from ambient vibrations were recorded. The results give a good indication of the material quality and damage (e.g. equivalent elastic modulus, cracking in the masonry etc.) and the data obtained were used to calibrate the 3D FEM to accurately predict the dynamic behavior of the monument.
Ambient vibration registrations consist of recording signals induced by ambient perturbation with the use of accelerometers. Sensors were placed at each story. This paper presents the results obtained with the servo-

accelerometers placed on two opposite corners of the tower roof.

The frequencies recorded from the ambient vibration measurements are shown in Table 1. The first frequency range represents the first two modes in the direction east-west and north-south, respectively. The frequencies of these two modes were too close and it was not possible to distinguish them during the test. The second range provides the frequency of the first torsional mode.

Table 1 Frequency recorded

Range	*Frequency (Hz)*
1	**3.7-4.0**
2	**7.2 - 8.4**
3	**10.4 ÷ 10.6**
4	**14.8 ÷ 15.4**
5	**19.4 ÷ 19.9**
6	**31.3**

The ambient vibration data were compared to the results obtained with the modal analysis. The modal analysis was carried out with Midas Gen (vers. 7.21, Midas, 2007) and 50 eigenvalues were extracted. In order to consider the interaction with the external boundaries, the lateral external walls were modeled with 3D 8-node brick elements, whereas for the modal analysis the tower was considered fixed at the base. The elastic modulus considered was E = 3000 MPa. Table 2 shows the values obtained, in the table the participating masses of the tower are indicated in bold (Figure 3) whereas the other modes refer to the boundary external walls. As for the experimental data the first two modes are translational and the third is torsional. The vibration modes obtained confirm the regularity of the tower. The model represents quite well the experimental frequencies obtained from the ambient vibration tests, without any modifications of the geometric or mechanical properties of the model.

Table 2 Vibration modes of the tower (X: direction North-South, Y: direction East-West)

Mode	*Freq.*	*Period*	*Participating Mass*		*Mode*	*Freq.*	*Period*	*Participating mass*	
	(Hz)	*(sec)*	*M_X(%)*	*M_Y(%)*		*(Hz)*	*(sec)*	*M_X(%)*	*M_Y(%)*
1	**3.78**	**0.26**	**2.25**	**70.29**	21	21.99	0.05	0.14	0.00
2	**3.93**	**0.25**	**54.76**	**3.08**	22	22.98	0.04	0.00	0.20
7	**8.53**	**0.12**	**0.03**	**0.28**	27	25.09	0.04	0.00	1.52
10	**11.85**	**0.08**	**0.14**	**0.04**	30	26.74	0.04	0.03	0.01
12	**13.32**	**0.08**	**0.00**	**6.55**	32	28.34	0.04	0.00	0.00
16	**14.86**	**0.07**	**0.01**	**1.28**	36	30.01	0.03	0.01	0.20
18	**18.09**	**0.06**	**13.02**	**0.08**					
38	**31.50**	**0.03**	**0.03**	**0.28**	39	32.08	0.03	0.01	1.07

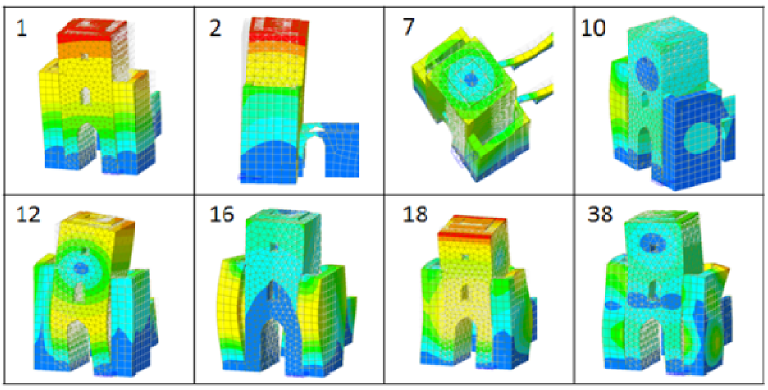


Figure 3 Principal vibration modes

3. BELL TOWER NUMERICAL MODEL

A three-dimensional nonlinear FE model of the bell tower, including the supporting soil, was developed using the version 6.5.1 of the commercial code Abaqus (Abaqus Inc., 2005) as shown in Figure 4.
The ground is modeled with linear base condition using 21320 nodes and 18720 8-node hexahedral elements (C3D8R) to study the influence of ground flexibility on the tower response. The tower is modeled using 11040 nodes, 26251 4-node linear tetrahedral elements (C3D4), 4488 linear hexahedral elements (C3D8R) and 72 4-node linear quadrilateral elements (S4R). The tower is connected to the soil using the “tie” option (Abaqus Inc., 2005), which allows to “fuse” together two surfaces with different meshes. The constrains exerted by the cathedral were modeled with “hard” contact interactions which do not allow penetration in compression, but allows the contact to open freely in tension.
The ground is modeled with linear base conditions to assess the ground flexibility influence on the ground motion (input at the base rock) and on the tower response. The mechanical parameters used in the analyses are reported in Table 3.

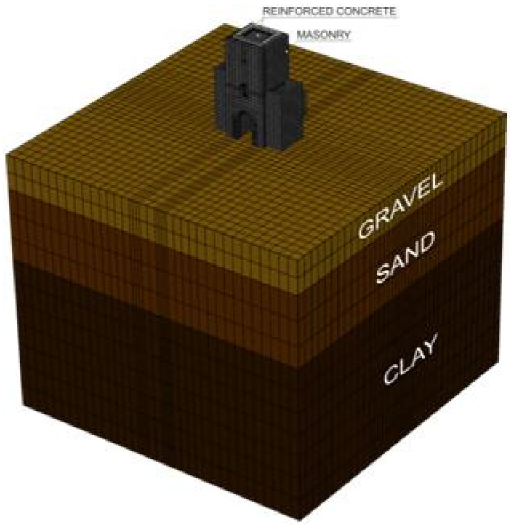


Figure 4 Soil domain stratigraphy and FE model

The monument sits on a thin layer of compact gravel. Beneath this layer, the soil is mainly composed of very dense sand and stiff clay layers. Two boreholes presented in Sciarra and Rainone (1999) were drilled within 200 meters from the tower and provide a wave velocity profile down to a depth of 60 meters.
Table 3 shows shear wave velocities V_s, p-wave velocities V_p, specific weight γ_s, elastic E_0 and shear G moduli obtained with the downholes.

Table 3 Dynamic values obtained with the down-holes

Layer	V_s *m/sec*	V_p *m/sec*	γ_s *kN/m³*	E_0 *MPa*	G *MPa*
1	125	1820	21.0	984	33
2	350	1820	21.0	7714	262
3	350	1340	19.5	7153	244

The nonlinear constitutive model used for the masonry of the tower is based on classical damage plasticity. The evolution of the failure surface is controlled by two hardening parameters which are linked to failure mechanisms under compression and tension loading (Abaqus Inc., 2005). The stress-strain response follows a linear elastic relationship until the value of the failure/yield stress is reached. In compression, beyond the yield stress the plastic behavior is represented by stress hardening and strain softening as shown Figure 5(a).
In the case of tension loading, the failure stress corresponds to the onset of micro-cracking in the masonry material. The crack formation is modeled with a softening stress-displacement response, as shown for the uniaxial case in Figure 5(b).

The damage in the material is represented through a scalar parameter *d*. The stress-strain relations for the general three-dimensional multiaxial condition are given by the scalar damage elasticity equation:

$$\boldsymbol{\sigma} = (1-d)\mathbf{D}_0^{el} : (\boldsymbol{\varepsilon} - \boldsymbol{\varepsilon}^{pl}) \tag{3.1}$$

where   is the initial (undamaged) elasticity matrix.

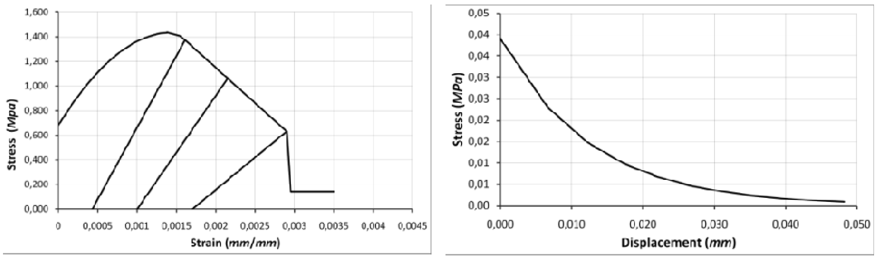
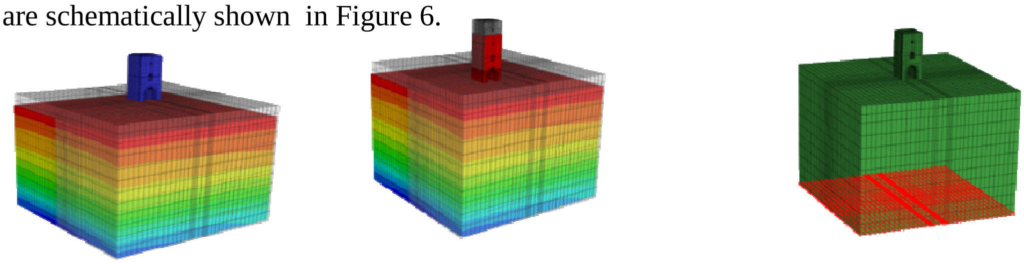


Figure 5 (a) inelastic compressive behavior, (b) tensile behavior

The value of the scalar damage under cyclic loading is unknown. Three damage conditions are described in Camata et al. (2008). A single damage model is described in this paper. In the low damage range, when the plastic deformation ε_p = 0.001, the scalar damage parameter d = 0.01, when ε_p = 0.022, the scalar damage parameter d = 0.40, whereas in the high damage range, when the plastic deformation ε_p = 0.001, d = 0.01. When ε_p = 0.022, d = 0.55, when ε_p = 0.00295, d = 0.80 and finally when ε_p = 0.003, d = 0.94.

The analyses consisted of three steps. In the first step only the soil domain was loaded with its gravity load (*step 1*). In the second strep the bell tower was loaded with its self weight (*step 2*). Finally, the total ground acceleration was applied at the base of the soil domain in two horizontal orthogonal directions (*step 3*), The three steps are schematically shown in Figure 6.



Step 1 *Step 2* *Step 3*

Figure 6 Loading steps

Two natural accelerograms were selected among the main events recorded from the accelerometric station GRD, located on rock soil (Type A according to EC8, CEN 2004) in Bocca di Valle in the Guardiagrele county, which is part of the Italian national accelerometric network RAN (Department of the National Civil Protection). The seismic input selection was made based on:

- the historical seismic activity in the Guardiagrele area (described in Boschi et al. 2007), with particular attention to the seismogenic zones which caused major damages in the past;
- the input frequency content, which relates to the different epicentral areas and to the geodynamic evolution of the Appenini Mountain range.

One dimensional numerical modeling with an equivalent linear analysis was carried out in the frequency domain (Schnabel et al., 1972) to calculate the local seismic effects at the basement of the tower as well as at the basement of the soil domain. This was performed considering the vulnerability of the monument in respect to the events originated from different epicentral areas. The geological model and the moduli degrading curves used to perform the analysis were obtained using the geological and geophysical (down hole) data available at the site.

The recorded accelerograms, the input frequency content and elastic spectra derived from the accelerograms at the base tower are shown in Figure 7. It is worth noting that the two earthquakes have the same peak ground

acceleration but quite different frequency contents. More specifically, the Fucino earthquake has an important frequency content at 4 Hz, which is the natural period of the tower.
Figure 8 shows the elastic spectra obtained deconvoluting the recorded accelerograms to the base of the solid domain. The figures show that the peak ground acceleration at the tower base is amplified by a factor 3 with respect to the soil base.

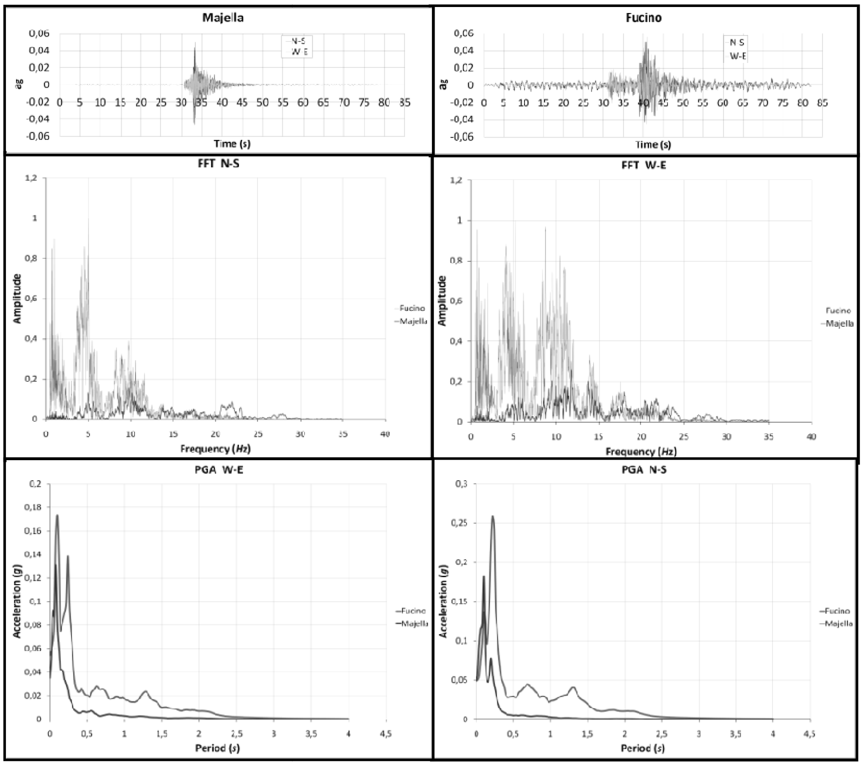


Figure 7 Majella and Fucino earthquakes at the base of the bell tower

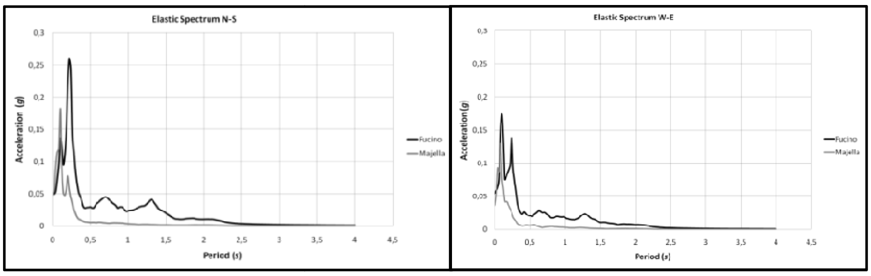
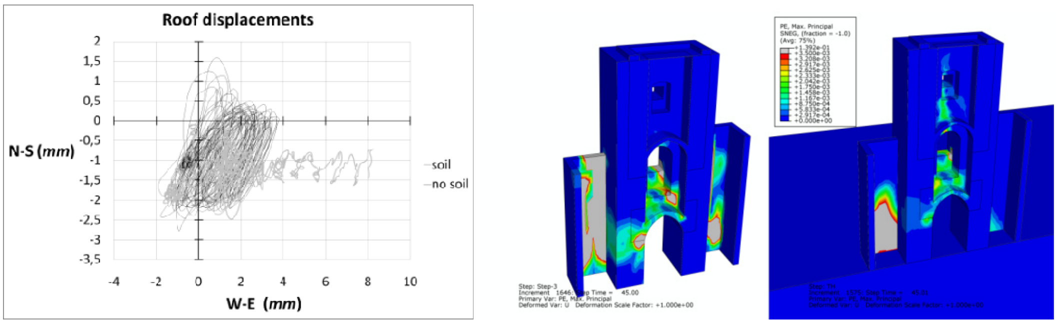


Figure 8 Majella and Fucino earthquake spectra obtained from the accelerograms deconvoluted to the base of the soil domain

4. RESULTS

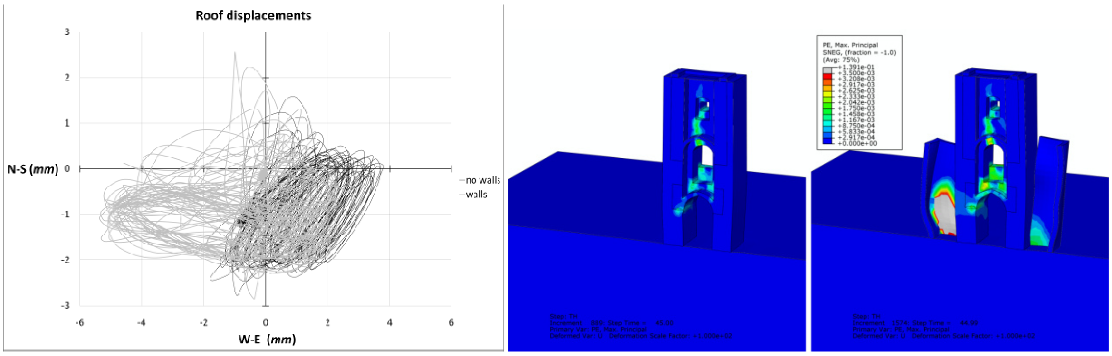
The bell tower response is first studied using the Majella earthquake accelerogram.
Figure 9 shows the results obtained with two different models: one that considers the tower alone with fixed base conditions, the second one (described in the previous paragraph) that explicitly models the base soil. At the initial stage of the time history analysis, the tower response obtained with the fixed boundary conditions is more rigid than that obtained including the soil. However, the fixed conditions tend to cause larger inelastic plastic strains and damage compared to the analysis with the soil domain as shown by the maximum principal inelastic strains. For this reason, at the end of the time history, the roof tower displacements obtained with the fixed based are more than twice those obtained with the flexible soil.



(a) roof displacement (b) maximum principal inelastic strains

Figure 9 Fixed vs. flexible supporting soil responses

The lateral constrains influence the displacement response of the tower as shown in Figure 10. In fact, the tower restrained by the lateral walls exhibits smaller roof displacements and larger inelastic plastic strains.



(a) roof displacement (b) maximum principal strains

Figure 10 Fixed vs. flexible supporting soil responses

Figure 11 shows the principal inelastic strain components at 80 seconds for the case with high damage. The results indicate that the Fucino earthquake is far more demanding than the Majella earthquake.

At 80 seconds large inelastic principal components in tension and compression are present in the tower indicating that the tower would collapse under the Fucino earthquake and survive without large plastic deformations to the Majella earthquake.

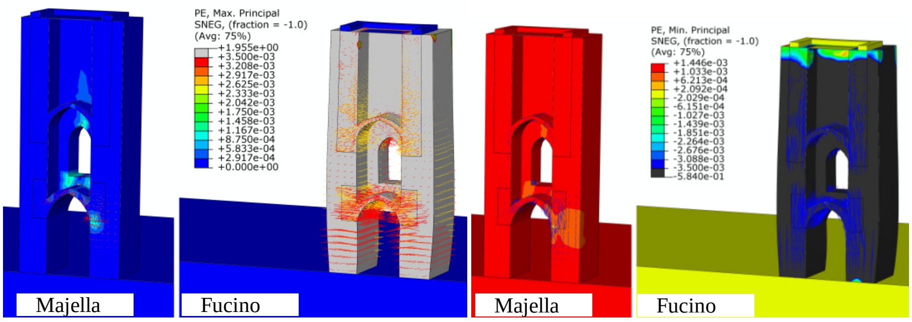


Figure 11 Majella vs. Fucino earthquake tower response

This underlines the importance of the frequency content of the input. In fact, although the peak ground acceleration of the two earthquakes is the same, the Fucino earthquake has large frequency amplitudes at 4 Hz, a frequency that is very close to that of the first two fundamental modes of the tower (Table 1). It is worth noting that while the Majella earthquake had its epicenter near the town of Guardiagrele, the epicentral distance of the Fucino earthquake from the bell tower is around 50 km.

5. CONCLUSIONS

This paper presents partial results of an ongoing study on the seismic vulnerability assessment of the bell tower of the Cathedral of S. Maria Maggiore in Guardiagrele – (Chieti province, Italy).
The following conclusions can be drawn from this study:

- the selection of the input ground motion records for time history analyses is very important. In particular, the results indicate that the structural response is greatly influenced by the frequency content of the input and that the PGA alone is not a sufficient indication. In order to correctly evaluate the vulnerability it is important to consider the historical seismic activity as well as the input frequency content;
- the flexibility of the soil as well as the influence of the external constrains greatly influence the response of the monument and therefore cannot be neglected.

This paper reports the results of an ongoing study. Currently, the importance of several other parameters are investigated. In particular, the influence of the nonlinear soil behavior shows to be an important parameter that needs to be further investigated.

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