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SEISMIC ANALYSIS OF ANCIENT MASONRY TOWERS

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Abstract

In this paper, numerical analysis of several historical masonry towers located in Italy was conducted. The towers differ in their geometric characteristics in terms of slenderness, thickness of the outer walls, the proportion of openings, while the material properties of all towers are similar.

The purpose of this paper was to analyse the influence of various geometries, soil properties and types of earthquakes on the seismic resistance of masonry structures. The geometries of the towers were taken from the available literature.

The analysis was carried out with the planar numerical model Y-2D, which is based on the finite-discrete element method (FDEM). The towers were discretized at the macro level using triangular three-node finite elements between which contact elements were implemented that take into account material nonlinearity.

The discretization of the towers was carried out at the macro level, taking into account the averaged properties of the mortar and blocks. This is modelled using triangular three-node finite elements between which contact elements are implemented to consider material nonlinearity. In this way, the phenomenon of the initiation and propagation of cracks in the tension and shear was modelled.

An incremental dynamic analysis was performed for three real earthquakes until the complete collapse of the structure. In each increment, the appearance of the first cracks, the propagation of the cracks, as well as the failure mode of the structure were monitored.

The performed numerical analyses highlight the suitability of the FDEM method in the analysis of the seismic resistance of masonry structures.

The conclusions reached in this paper can serve as guidelines for engineers in assessing the seismic resistance of existing masonry structures.

Keywords: historical structures, masonry tower, finite discrete element method, seismic resistance.

1. Introduction

Masonry towers represent an important portion of the built heritage. For that reason, their seismic resistance assessment against earthquakes appears to be of relevant importance for historical reasons. Two existing masonry towers, located in the north Italy are analysed in the presence of seismic excitation (Fig. 1). The geometry of towers is deduced from literature [1]. On the basis of such geometrical data 2D numerical models were built, where different thicknesses are assigned to adjoining elements. The analysis was carried out with the planar numerical model based on the finite-discrete element method (FDEM).

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(a)

(b)

Figure 1. Towers: a) bell tower A located in Vescovana, b) military defence tower B located in Arquà Polesine.

2. Numerical analysis

2.1 Finite-Discrete Element method

Combined Finite-Discrete Element method is intended for the dynamic analysis of a large number of mutually interacting discrete elements, where the elements can fracture and fragment thus increasing the total number of discrete elements [2]. Within FDEM, each discrete element is discretised with its own finite element mesh thus enabling the deformability of discrete elements. Fracture and fragmentation processes are also implemented within the finite element mesh. The mass of discrete elements is lumped into the nodes of finite elements, while the time integration of the motion equation is applied node by node and degree-of-freedom by degree-of-freedom. This is performed in explicit form by using the central difference time integration scheme. The contact forces resulting from the interaction process between two discrete elements are determined by the numerical representation of contact impact, which is executed by employing contact detection and contact interaction procedures [2-4].

The model for fracture and fragmentation, adopted within FDEM is actually a combination of smeared and discrete crack approaches [5]. It was designed with the aim of modelling progressive fracture and failure including fragmentation and of creating a large number of rock fragments. For that purpose, the strain softening which appears in the material after reaching the tensile or shear strength is described in terms of displacement.

In all previous analyses [6-8], this method proved to be very suitable for numerical analyses of masonry structures exposed to dynamic loading. Therefore, it very well describes all the phenomena of the behaviour of the masonry structure exposed to seismic load, such as the formation and propagation of cracks, energy dissipation, the mode of failure, as well as the complete collapse of the structure.

2.2 Description of the numerical model

Historical masonry towers located in Italy presented in this paper (Fig. 1) were made of small clay bricks with low tensile strength [1]. In order to shorten the calculation time, the calculation was made at the macro level, which means that the discretization of each block and mortar was not performed separately, but the discretization of the structure was made by an irregular finite element network with average properties in terms of modulus of elasticity, tensile strength and shear to expect for this type of structure. For the purposes of numerical analysis, the tower A is discretized with 5073, while tower B with 5926 triangular finite elements [9,10].

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The geometry of the models is shown in Fig. 2 (a) and (b), and the finite element mesh used in the numerical analyses in Fig. 2 (c) and (d). Contact elements have been implemented between the finite elements for the purpose of simulating the initiation and propagation of cracks in the structure.

The tensile strength of the contact elements taken was selected in the amount of 0.27 MPa, and the shear strength in the amount of 1.08 MPa. The coefficient of friction was adopted in the amount of 0.7. The modulus of elasticity of finite elements was chosen in the amount of 22500 MPa and the Poisson's ratio in the amount of 0.3. A simplified 2D discretization of the towers is adopted by assuming for the elements, whenever necessary, different thicknesses.

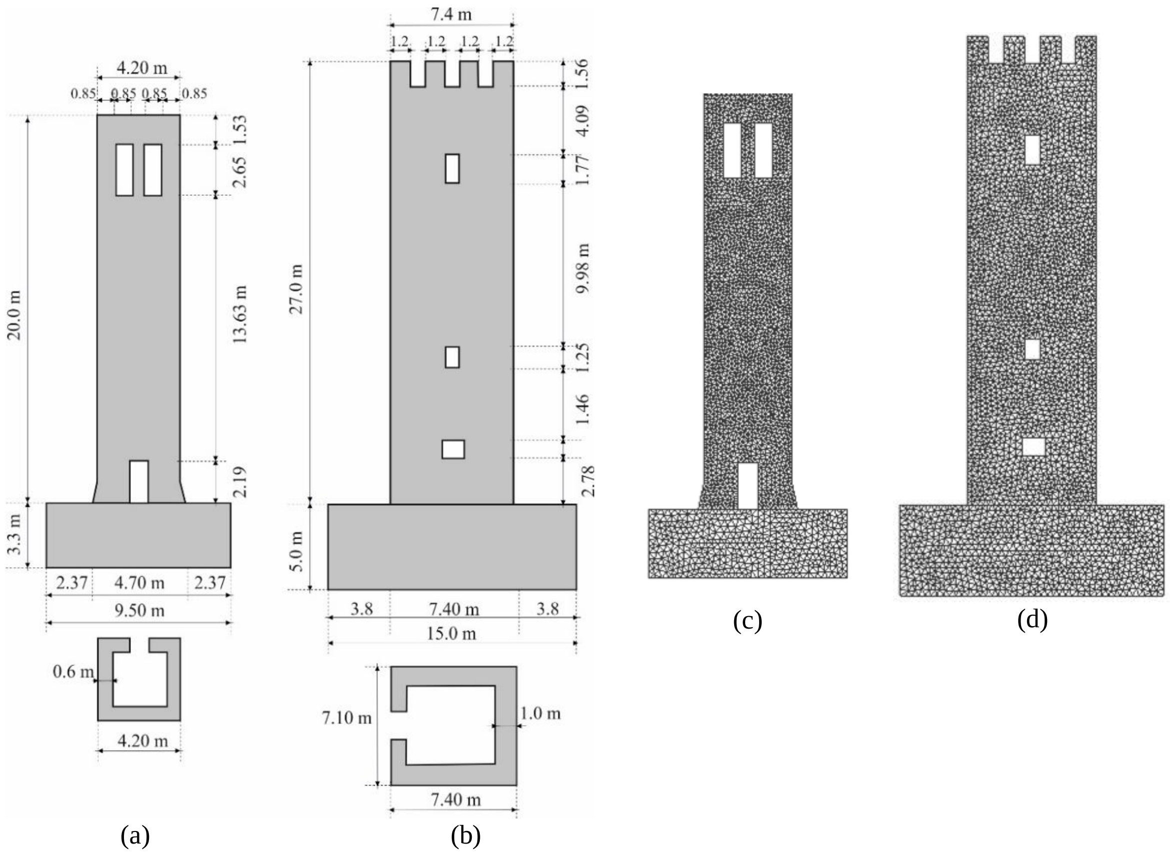

Figure 2. Geometry of tower: a) A; b) B; Discretization of tower: c) A; d) B.

In presented numerical analyses, the structures were exposed to horizontal and vertical ground acceleration (Fig. 3a and Fig. 3b) which was recorded on 15. April 1979. during an earthquake with the epicentre in Petrovac (Montenegro).

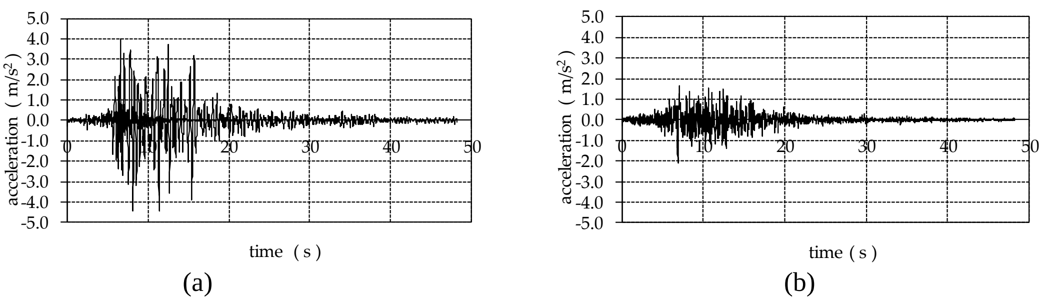

Figure 3. Accelerograms of the earthquake in Petrovac recorded on 15 April 1979.: (a) horizontal north-south direction; (b) vertical direction.

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3. Results and conclusion

In this paper, an incremental dynamic analysis of masonry towers for two different support conditions, elastic and rigid base, was performed.

The displacement of the top of the tower A on elastic base under different peak ground accelerations are shown in Fig. 4, while the initiation and propagation of the cracks that appeared in this numerical model during the action of the given accelerogram (with the largest ordinate 0.7g) are shown in Fig. 5.

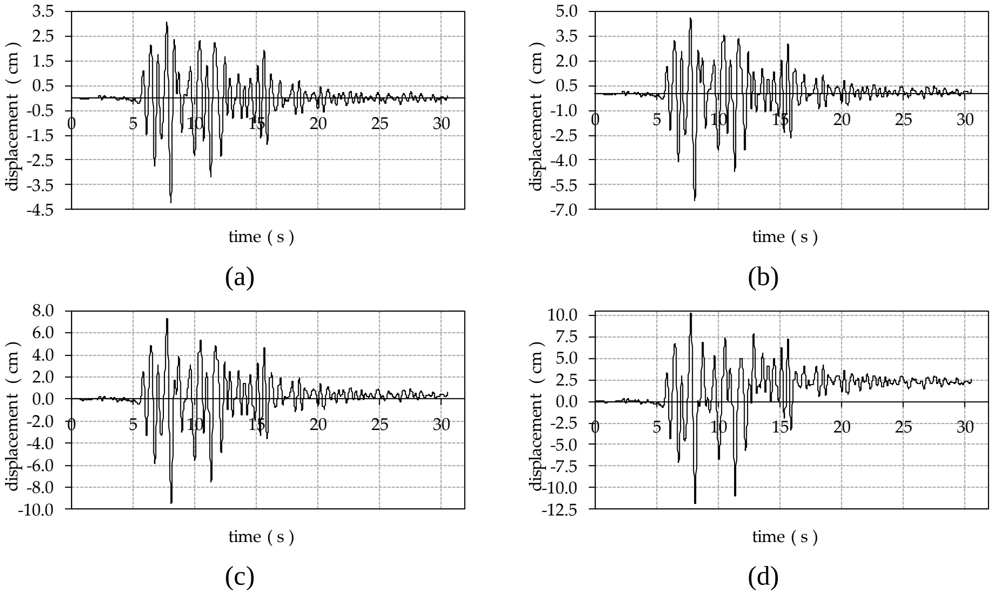

Figure 4. Displacement of the top of the tower A on elastic base under the peak ground acceleration of: (a) a_g=0.15 g; (b) a_g=0.30 g; (c) a_g=0.45 g; (d) a_g=0.60 g.

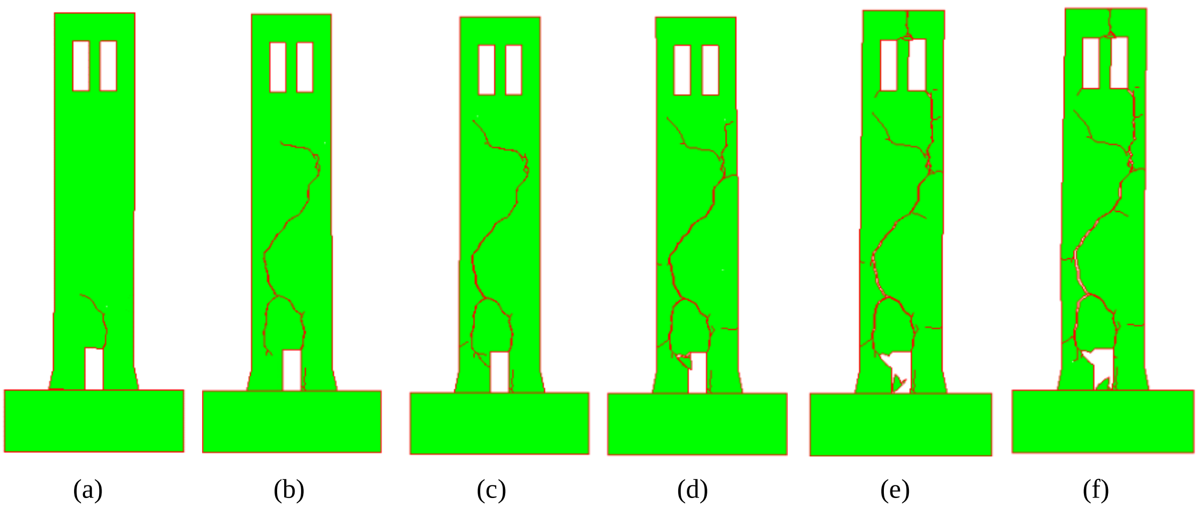

Figure 5. Crack pattern of the tower A on elastic base for acceleration ($a_{g.max.}$= 0.70 g) in time: (a) t = 7.9 s; (b) t = 8.5 s; (c) t = 9.5 s; (d) t = 14.5 s; (e) t = 32.0 s; (f) t = 48.2 s.

The displacement of the top of the tower A on rigid base under different peak ground accelerations are shown in Fig. 6, and the cracks of observed tower during the action of the given accelerogram (with the largest ordinate 0.5g) are shown in Fig. 7.

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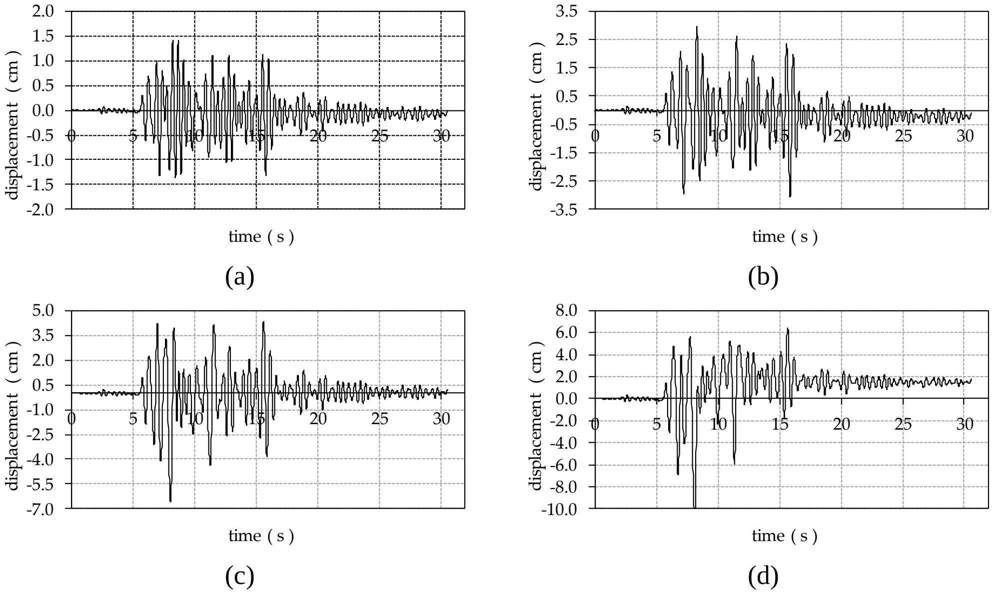

Figure 6. Displacement of the top of the tower A on rigid base under the peak ground acceleration of: (a) a_g=0.10 g; (b) a_g=0.20 g; (c) a_g=0.30 g; (d) a_g=0.45 g.

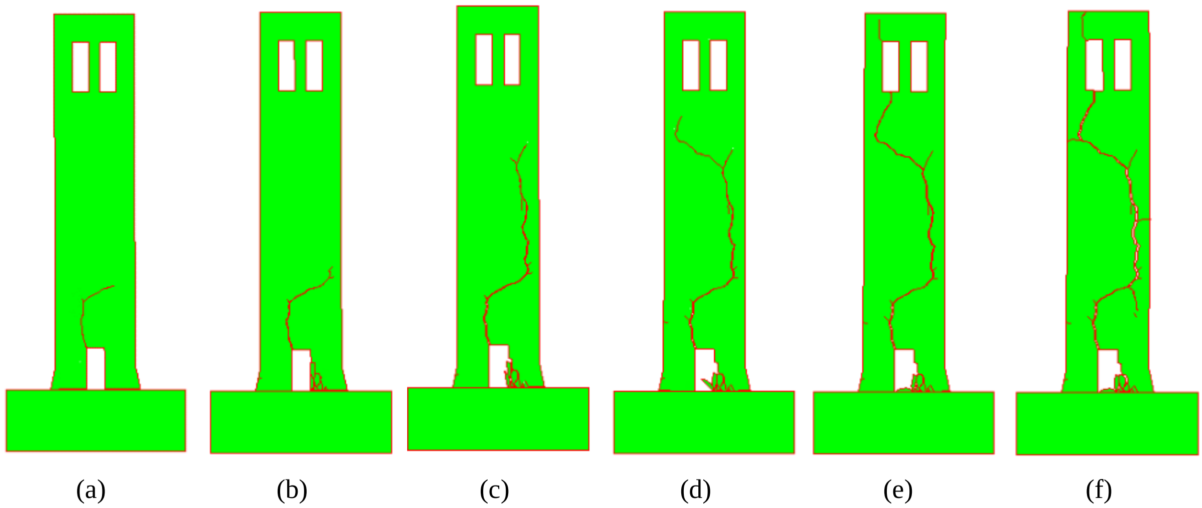

Figure 7. Crack pattern of the tower A on rigid base for acceleration ($a_{g.max.}$= 0.50 g) in time: (a) t = 8.0 s; (b) t = 8.3 s; (c) t = 8.5 s; (d) t = 9.5 s; (e) t = 9.8 s; (f) t = 48.2 s.

In Fig. 8, the displacement of the top of the tower B on elastic base under different peak ground acceleration are shown.

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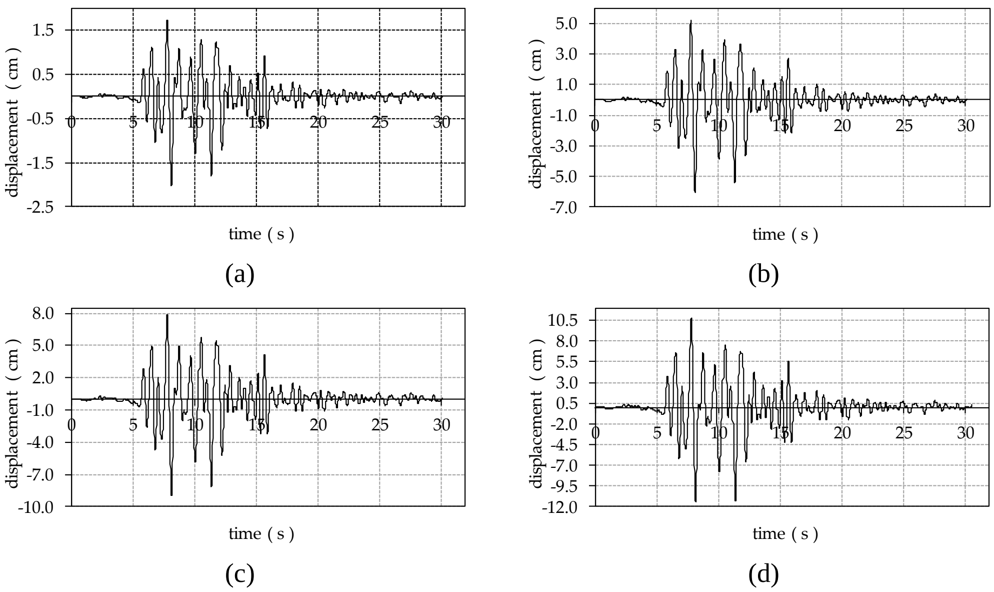

Figure 8. Displacement of the top of the tower B on elastic base under the peak ground acceleration of: (a) a_g=0.10 g; (b) a_g=0.30 g; (c) a_g=0.45 g; (d) a_g=0.60 g.

The initiation and propagation of the cracks that appeared in presented numerical model for tower B, during the action of the given accelerogram (with the largest ordinate 0.65g) is shown in Fig. 9.

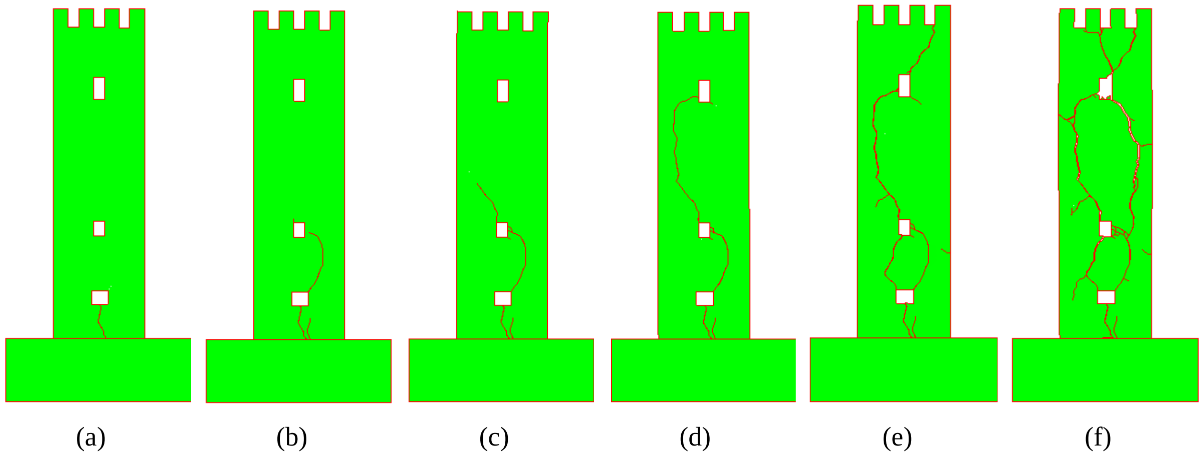

Figure 9. Crack pattern of the tower B on elastic base for acceleration ($a_{g.max.}$= 0.65 g) in time: (a) t = 9.0 s; (b) t = 11.8 s; (c) t = 12.0 s; (d) t = 12.3 s; (e) t = 13.0 s; (f) t = 17.0 s.

In Fig. 10, the displacement of the top of the tower B on rigid base under different peak ground acceleration are shown, while the cracks that appeared for the observed tower, during the action of the given accelerogram (with the largest ordinate 0.45g) is shown in Fig. 11.

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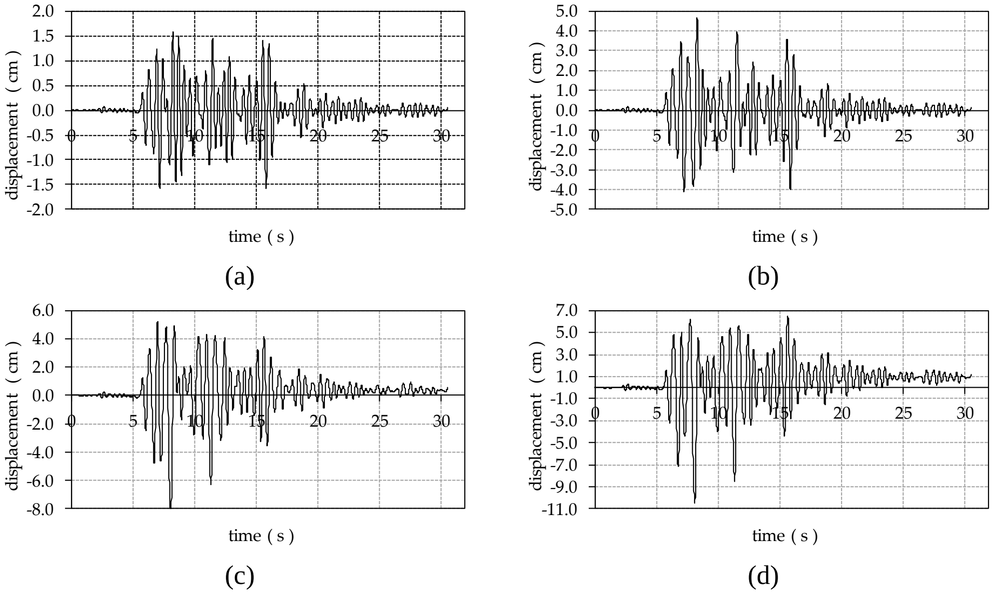

Figure 10. Displacement of the top of the tower B on rigid base under the peak ground acceleration of: (a) a_g=0.10 g; (b) a_g=0.25 g; (c) a_g=0.35 g; (d) a_g=0.44 g.

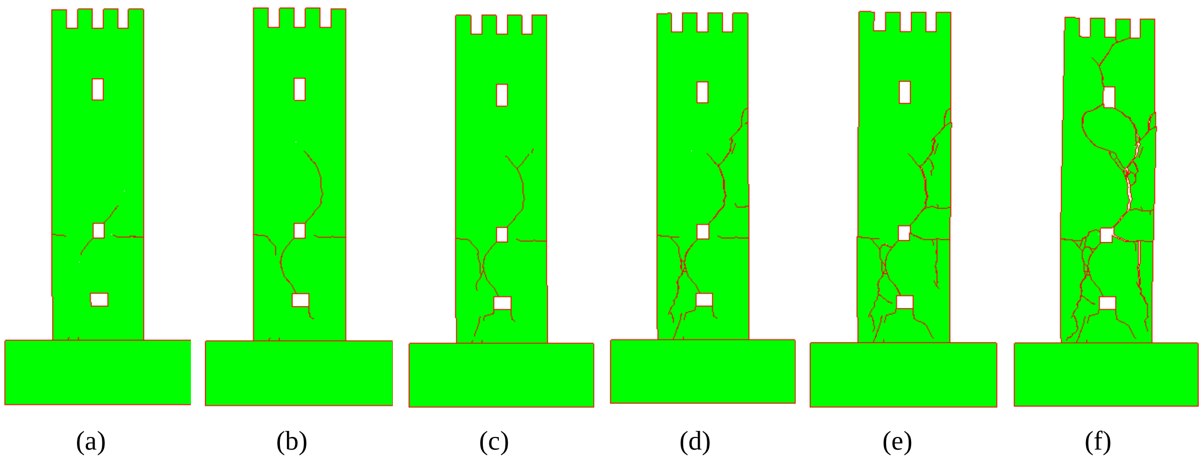

Figure 11. Crack pattern of the tower B on rigid base for acceleration ($a_{g.max.}$= 0.45 g) in time: (a) t = 8.3 s; (b) t = 8.5 s; (c) t = 10.2 s; (d) t = 11.5 s; (e) t = 12.0 s; (f) t = 17.0 s.

Figure 12 presents the comparison of the displacements for different peak ground acceleration on elastic and rigid base.

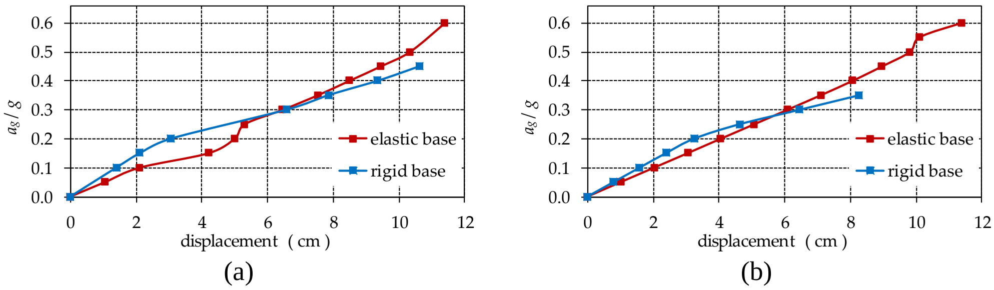

Figure 12. Displacement for different peak ground accelerations on rigid and elastic base for tower: (a) A; (b) B.

From the presented results, it can be seen that the tower on the elastic base has a higher seismic resistance in comparison to the tower on the rigid base. Also, the displacements achieved at the top of the tower are greater for the type of support on an elastic base.

Acknowledgements

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