An evaluation of the soil-structure interaction in the non-linear dynamic analysis of masonry towers

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SUMMARY:
The object of the paper is the influence the soil-structure interaction on the dynamical response of a masonry tower, for which a high level of stress is involved already in the static field. The relevant deformations and displacements at the base of the tower suggest that a significant volume of ground is engaged into the overall dynamic response, both as a participating mass and as a potential carrier of energy dissipation. In order to investigate this aspect and assess the sensitivity of the dynamic response of the soil-structure system to different soil characteristics, the non linear dynamic response of a case study is analyzed, by including in the model a significant volume of foundation soil and considering two different ground types. The numerical model is based on a specific Rigid Body and Springs approach, able to model the significant inelastic aspects of the constitutive behaviour and the meso-scale damage mechanisms with a moderate computational effort.

Keywords: Nonlinear dynamic analysis; Masonry Towers, Soil-structure Interaction

1. INTRODUCTION

The paper deals with the non linear dynamic modelling and analysis of slender masonry bell-towers with a specific reference to the problem of the soil-structure interaction. In the literature, there are several research studies dealing with the seismic assessment and the vulnerability analysis of masonry towers, with regard to different aspects: mechanical and numerical analysis by computational or simplified approaches (Casolo 1998; Riva et al. 1998; Bernardeschi et al. 2004; Carpinteri et al. 2006; Curti et al., 2006; Peña et al. 2010; 2001; Milani et al, 2012); experimental testing methods and structural identification (Binda et al. 2005; Ivorra and Pallares 2006; Russo et al. 2010; Anzani et al. 2010). A significant case is that of the Civic Tower of Pavia, Italy (about 900 years old), suddenly collapsed on 17 March, 1989 (Binda et al., 1992), which has drawn the attention of the scientific community on the high vulnerability of masonry towers also to low-intensity earthquakes, since static vertical loads combine with the dynamic loads induced by the ground motion. The examination of the documentation about the damage caused by the 1976 Friuli earthquake (Doglioni et al., 1994) points out that, in isolated bell towers, damage patterns tend to be distributed all along the height, although it is frequently more severe at the base. This suggests the need of further investigations about the combined effects of flexural and axial actions, as well as the incorporation into the model of the higher vibration modes, which seem to be have a relevant role in the damage of the upper part, especially the tower crown and belfry (Curti et al., 2006). Moreover, during strong earthquakes, shear damages are often observed, and in this case the reduction of the section stiffness (i.e. the loss of validity of the Eulero–Bernoulli hypothesis of plane cross-section) can significantly affect the overall response of the structure. When the tower is not particularly slender, and depending on the frequency content of the forcing actions, a material model which is capable to describe both the axial and the shear response and damage under cyclic loading is required in order to investigate the global shear damage effects (Casolo and Peña, 2007; Peña et al., 2010). Even if great attention has been devoted to the theme (the mentioned references are a limited part of the available literature) the dynamic analysis of masonry structures in the presence of the interaction with the foundation soil is still unexplored. A first approach to the problem is presented in the paper, by proposing a direct modelling of the soil-foundation-structure system.

2. THE NUMERICAL MODEL: RIGID BODY AND SPRING APPROACH

Analyses are performed by means of a specific mechanistic model, made by rigid masses and springs, (RBSM) which considers only the in-plane dynamics. This model is capable of describing higher vibration modes, as well as the combined axial and shear deformation and damage of the material by means of a simplified heuristic approach (Casolo, 2004; 2009). The elements are quadrilateral and have the kinematics of rigid bodies with two linear displacements and one rotation, as shown in Figure 1(b). Three springs devices connect the common side between two rigid elements or the restrained sides, as shown in Figure 1(c). These connections are two axial springs k_P and k_R, placed in the point P and R separated by a distance $2b$, and one shear device k_Q placed in the middle of the side. A volume of pertinence V^P, V^Q and V^R is assigned to each connection point. The elastic characteristics of the connecting devices are assigned with the criterion of approximating the strain energy of the corresponding volumes of pertinence in the cases of simple deformation. The conceptual core of this model is the macroscopic unit cell defined by four quadrilateral rigid elements connected to each other as shown in Figure 1(a). The cell size should be equal or larger than the minimum representative volume (RVE) of the heterogeneous solid material. In particular, the orthotropy of the shear response and the local mean rotation of the blocks, which depend on the different geometric arrangement of the vertical and horizontal material joints as well as the shape and size of the original blocks, are features that can be accounted at the macro-scale.

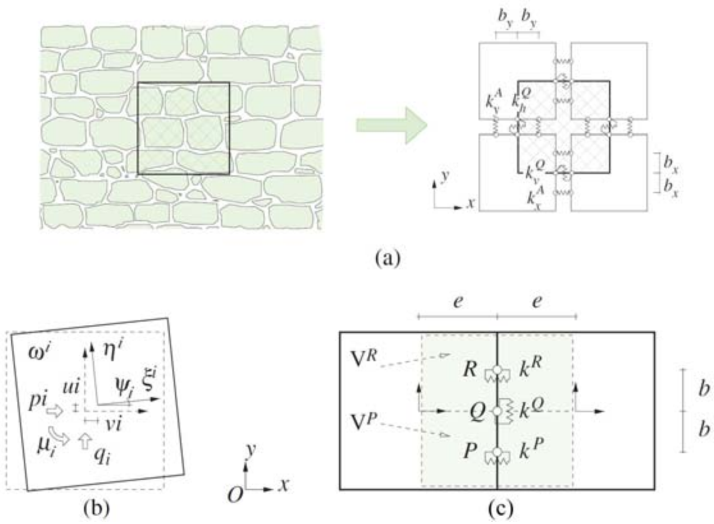

Figure 1. Scheme of the RVE in relation with the unit cell defined by 4 rigid elements (a); kinematics (b); disposition of the connecting spring-devices (c) (Casolo, 2004).

Out-of the linear elastic field, the main macroscopic constitutive aspects are: the very low tensile strength; the significant post-elastic orthotropy combined with the texture effects; the dependence of the shear strength on vertical compression stress; the progressive mechanical degradation during repeated loading; and the energy dissipation capability. To do this, a simplified heuristic approach is proposed, based on the phenomenological consideration of the main in-plane damage mechanisms that can be described at the meso–scale by adopting specific separate hysteretic laws for the axial and shear deformation between the elements. This separation reduces the computational effort, even though a Coulomb-like law is adopted in order to relate the strength of the shear springs to the vertical axial loading. The monotonic and hysteretic constitutive laws are assigned to the connecting devices adopting a phenomenological approach and separate phenomenological descriptions of the hysteresis behaviour of the axial and shear connections, as schematically shown in Figure 2. These laws are based on experimental monotonic and cyclic tests available in literature, and should be assigned to rigid elements whose size is approximately comparable to the test specimens in order to limit the problems with size effect. The plastic response of each axial connection is independent from the behaviour of any other connection, while the shear strength is related to the stresses of the axial connections according with Coulomb criterion. It is worth noting the true discrete character of this model. In fact, during loading, relative motion between two adjacent elements always occurs, with overlapping, separation or sliding between two adjacent rigid elements; numerically, this means compression, tension or shear in the volume of pertinence of the connecting devices. This notwithstanding, the initial contacts do not change during the analysis and the global assemblage maintains the initial connectivity (hypothesis of small displacements) in order to reduce the computational effort.

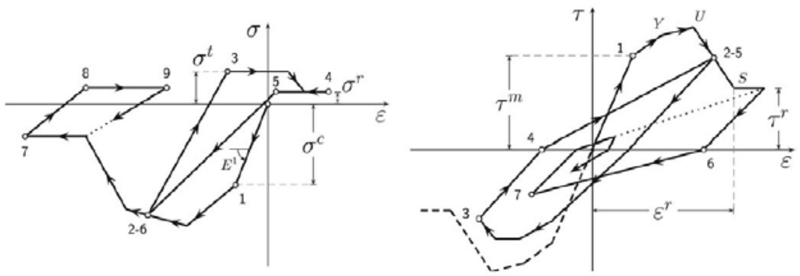

Figure 2. Scheme of the hysteretic rules for the axial (left), and shear springs (right).

3. THE CASE STUDY

A reference tower has been considered, which is supposed to be structurally independent, i.e. with no adjacent interacting construction, and characterized by geometrical regularity both in plan and in elevation. Dimensions were chosen by looking at a number of significant examples (Fig. 3), in order to represent an average masonry bell tower located in seismic zones of Northern Italy (without the intent, of course, to cover all the possible situations). A preliminary assessment of the structural response to seismic actions in the non linear field, both static and dynamic, was performed by the authors in a previous work (Casolo and Uva, 2011) in which, with regard to the boundary conditions at the ground, a cantilever scheme (no soil-structure interaction) was implemented in numerical analyses.

Figure 3 Some typical geometry and damage patterns of Italian bell-towers (Doglioni et al., 1994).

3.1. The geometry of the reference bell-tower

The geometry of the model is simplified by disregarding some geometric details, like for example the internal vaults. In Fig. 4 and 5, the 3D drawings and the schematic sections and plans of the tower are shown, and the geometrical characteristics are summarized in Table 3.1.

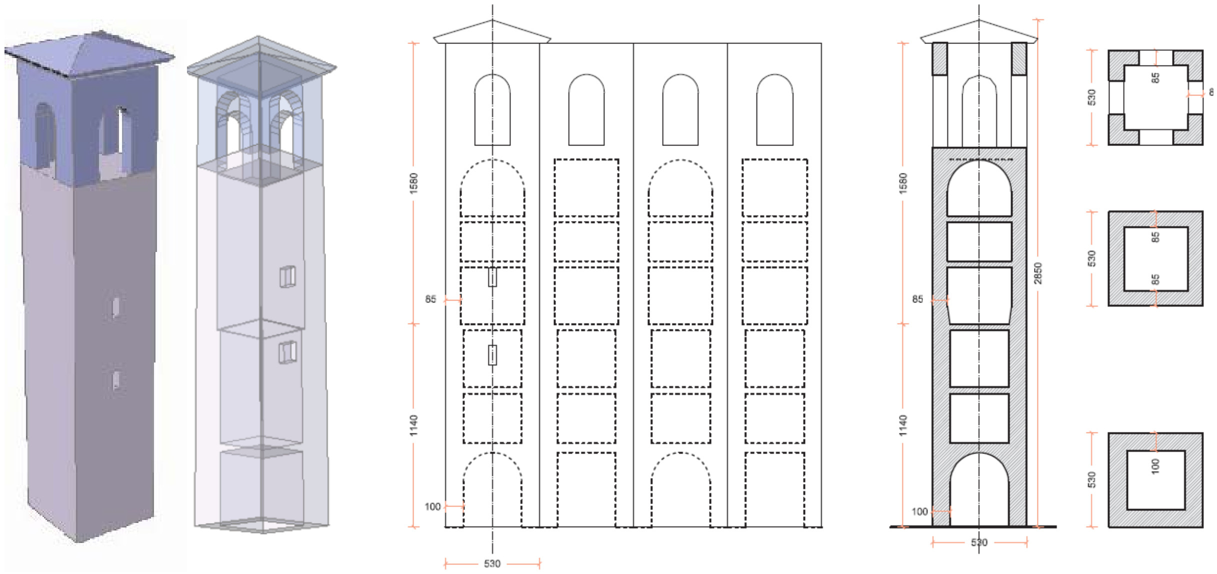

Figure 4. 3D drawing, schematic prospects, section and plan views of the idealized case study.

Table 3.1. Geometrical characteristics of the reference tower.

Total height (H)	28.50 m
Base (LxL)	5.30 m x 5.30 m
Base wall Thickness (t)	1.00 m
Wall mass density (ρ)	1900 kg/m^3
Damping (ξ)	0.05

3.2. Mechanical parameters of the masonry material

According to the constitutive model adopted for the axial and shear springs (Section 2), a set of parameters are needed in order to define the corresponding skeleton curves and hysteretic rules. The values assigned to the relevant parameters to define the masonry material of the reference tower are: compressive stress at the elastic limit: *1 MPa*; peak compressive strength: *2 MPa*; residual compressive strength: *0.2 MPa*; peak tensile strength: *0.2 MPa*; residual tensile strength: *0.02 MPa*; shear value at the elastic limit: *0.088 MPa*; peak shear strength on the horizontal plane: *0.097 MPa*; peak shear strength on the vertical plane: *0.165 MPa*; residual shear strength: *0.02 MPa*; friction coefficient on the horizontal plane: *0.25*; friction coefficient on the vertical plane: *0.05*.

3.3. The model of the foundation soil

3.3.1. Mechanical parameters of the soil

Two types of foundation soil have been considered in order to perform the dynamical analysis accounting for the soil-structure interaction: rock (type A ground) and deposits of compact gravel (type C ground). Since the study is oriented at appraising the interaction effects in the structural dynamic response, only the elastic part of the constitutive behaviour of the soil is actually relevant in the performed analyses. The elastic moduli adopted for the two soils are, respectively: $E_A = 1400$ *MPa*; $G_A = 584$ *MPa*; $E_C = 280$ *MPa*; $G_A = 117$ *MPa*.

3.2.1. Choice of the significant volume of soil

The choice of the significant volume of soil to be introduced in the numerical model is an important question. Clearly, it is necessary to consider a volume large enough to include the pressure bulb under the foundation system. This volume will also represent a mass of soil that participates in the dynamic response of the system. On the other side, the choice cannot be casual, because a possible effect of wave propagation, and in particular of resonance and multiple reflections within the domain, could arise. In order to minimize this eventuality, it was chosen to limit the width of the soil volume under the value of 1/4 of the length λ^{2Hz} of the waves "s", *secundae* corresponding to the frequency of the first mode of the tower (which is about *2 Hz*). Thence, for the two considered ground types, we have:

$$v_s = \sqrt{G/\rho}\ ;\ G_1 = 584\ MPa \rightarrow \lambda^{2Hz} = 285m;\ G = 117\ MPa \rightarrow \lambda^{2Hz} = 127m \qquad (5.1)$$

In Fig. 5, the mesh adopted for the numerical analyses of the case study is shown. The elements coloured in brown represent the foundation of the tower, and the mechanical parameters of masonry are assigned to them. The elements coloured in dark green represent the soil (for each of the two considered ground types, the proper mechanical parameters - *§3.3.1*- are assigned), and are *41 m* deep, in order to model the actual thickness of the participant soil mass. In order to reduce the problem of the wave reflection within the domain, two vertical strips (light green colour) endowed with a properly reduced of the stiffness in the horizontal shear springs (1/10 of the stiffness of vertical shear springs) have been introduced in the mesh. In the same figure, the two control points A and B are also shown.

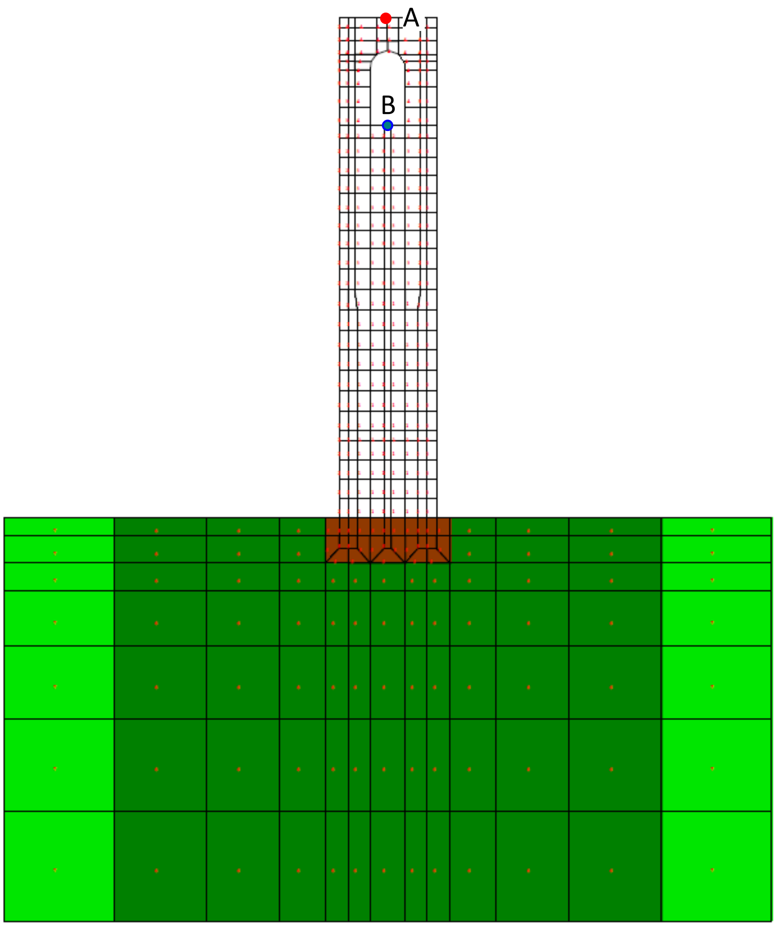

Figure 5 Mesh of the tower with the significant volume of soil included in the model, and location of the control points A and B.

4. NON LINEAR DYNAMIC ANALYSES ACCOUNTING FOR SOIL-STRUCTURE INTERACTION

Two numerical models of the reference tower have been analyzed (see Fig. 5) by using the two soil types defined in §3.2.1. The classical, simpler cantilever scheme, in which the interaction with the ground is neglected, has also been considered (Casolo and Uva, 2011). With regard to the cantilever model, the comparison with the scheme 1 (rock) indicates that no significant difference can be

observed in the dynamic response of the elevation structure and in the distribution of the damage, as it could actually be expected (for the detailed results about the cantilever model in terms of modal shapes, stress and strain maps, deformed shapes, see the reference Casolo and Uva 2011). In the following paragraphs, the discussion of the case study will be made with reference to the models with the soil volume: rock (actually equivalent to the full base clamping) and compact gravel. It should be remarked that, in the context of the present paper, attention is focused on the response of the tower and on the alteration induced by the interaction with the soil on the natural vibration modes and damage mechanisms, whereas the strictly geotechnical aspects (advanced modelling of the non linear constitutive behaviour and failure of the ground) are not treated.
For the dynamic analyses, a moderate level of hazard has been considered, by using artificially generated accelerograms which are consistent with the design spectrum of the Italian zone 3 (PGA = $0.15\ a_g$). This hazard level is representative of the regional areas where slender masonry bell-towers are mostly diffuse, and allow the appraisal of the structural performance in a frequent load condition. It should be also pointed out that only the horizontal components of the accelerograms have been considered, whereas the analysis in the presence of a vertical component has been at the moment left out (this is a very interesting and important question, that should be faced in the continuation of the research work). A final observation concerns the choice of the accelerograms, that are design consistent, artificially generated. It would be surely necessary, with regard to this aspect, to perform more extensive analyses, including natural accelerograms.

4.1 Effects on the natural vibration modes

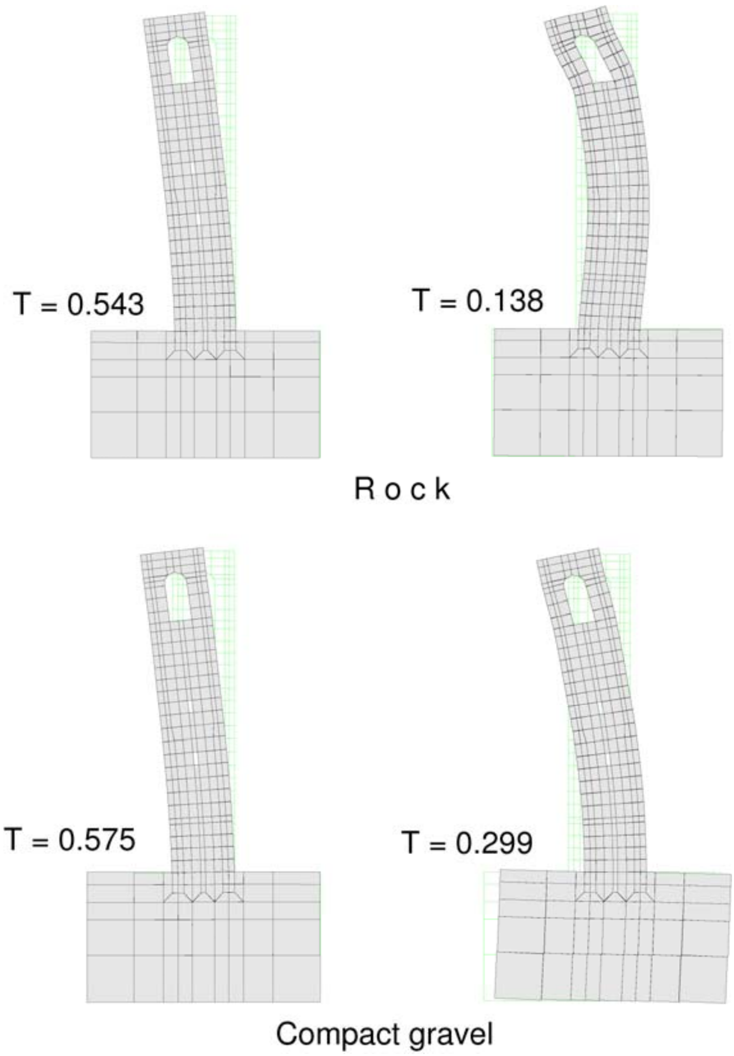

Figure 6 The first 2 mode shapes in the presence of a rock soil and compact gravel soil.

First of all, the natural vibration modes for the numerical models of the bell–tower with the two different foundation soils have been determined. In Fig. 6, the first two natural vibration modes (which are flexural modes) with the correspondent periods are shown. It can be observed that in the scheme 2 (compact gravel) there is an increase in the natural vibration period, especially for the second mode. By looking at the modal shape, it can be seen that the higher deformability of the soil has the effect of “increasing” the effective height of the tower, with a consequent effect also on the expected deflections.

4.2. The damage mechanisms

4.2.1. Preliminary analysis under gravity loads

Preliminarily, the analysis of the two models under the only gravity loads was performed (non structural permanent loads and service loads were not considered, since they represent a negligible quote of the total loads). In Fig. 7, the map of the stress component S_{22} is shown for the two ground types. It can be observed that with respect to the gravity loads no significant effect is induced on the structural response of the elevation structure, as it could be expected (the soil volume, for which a specific evaluation of the failure is outside the scope of the present study, is not reported in the figure).

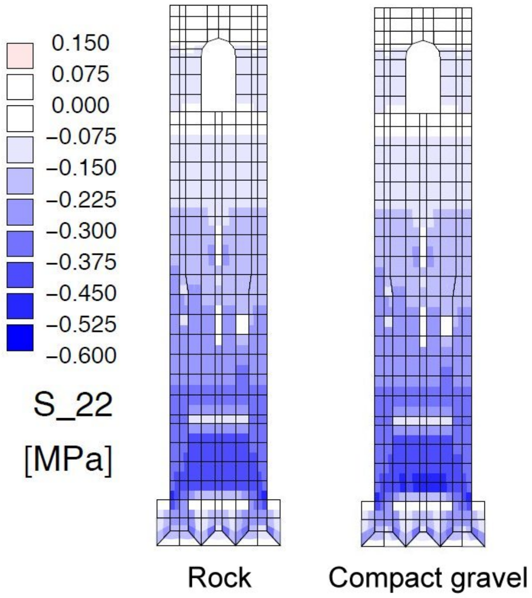

Figure 7 Map of the vertical axial stress component S_{22} for the two schemes.

4.2.2. Time history response and damage modes

The displacement time history and the kinetic energy for the two ground types reveal some interesting aspects. Displacement time histories have been recorded at points A and B shown in Fig. 5 (respectively, top and base of the belfry). By looking at the plot of the point B displacement, it can be observed that in scheme 2 – compact gravel soil, the oscillations have a higher frequency and higher peaks, as it could be predicted as an effect of the increased natural period observed in § 4.1. This kind of effect is also confirmed by the diagram of the kinetic energy (Fig. 8, right) that is considerably greater in scheme 2. With regard to the time history of the point A displacement, it should be remarked that it is strongly sensitive to the specific position along the *x-axis*, and could not be assumed as a representative parameter of the partial belfry mechanisms. In fact, in the case of the scheme 1 (where the collapse mode engages a crumbling of the right-upper portion of the belfry’s arch), the history of the chosen point is actually related to the spalling of the particular element (Fig. 9, final deformed shape). Instead, in scheme 2, there is a uniform shear-sliding mechanisms of the belfry (Fig. 10, final deformed shape) that is well represented by any of the points of the upper portion.

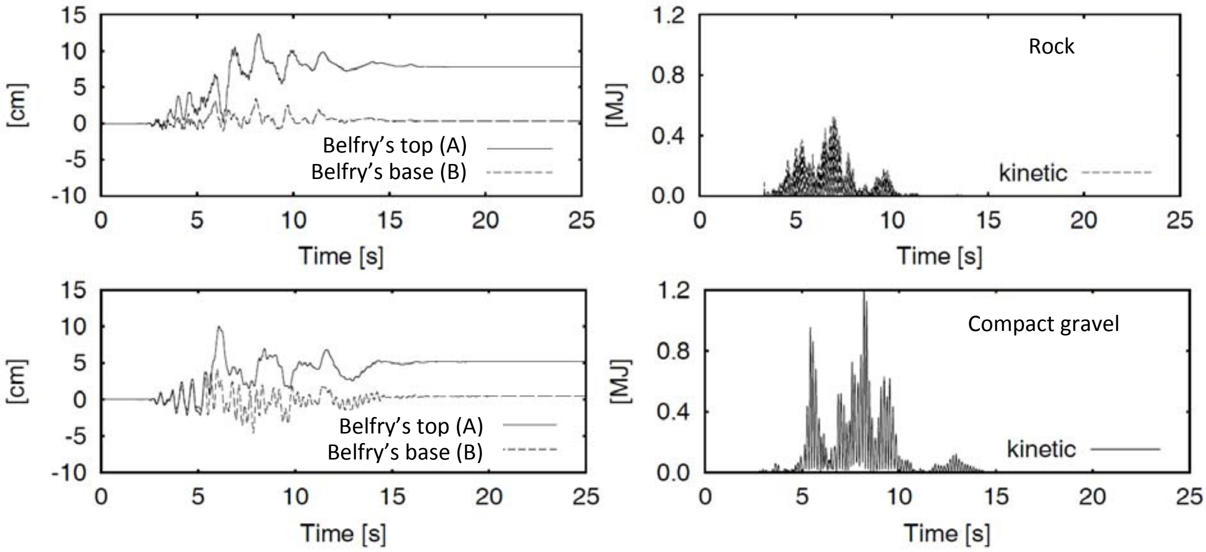

Figure 8 Displacement time history (left) for the control points (see Fig. 5) and kinetic energy (right).

An interesting aspect concerns the alteration of the damage mechanisms activated by the seismic history in the two cases. The diagrams of Fig. 9 and 10 illustrate the final deformed shapes, the maps of S_{22} stress component and the map of E_{12}-E_{21} strain components, which well represent the damage mechanism that is triggered. For the scheme 1-rock (Fig. 9), there is an evident local mechanism involving the belfry, with a concentration of damage in the lintel, whereas the development of a global shear mechanism is almost negligible. The global shear mode becomes very evident in the scheme 2 – compact gravel, where the tower tends to be split into two separate vertical portions (see Fig. 10, E_{21} map). This is also visible in the final deformed shape, where the consistent vertical displacements indicate the failure of the central strip. Also in this case the belfry is engaged in a local damage mode, that is anyway quite different from the previous one. Now, the base of the belfry is uniformly damaged (Fig. 10, E_{12} map), and the sliding of the whole belfry occurs (Fig. 10, final deformed shape).

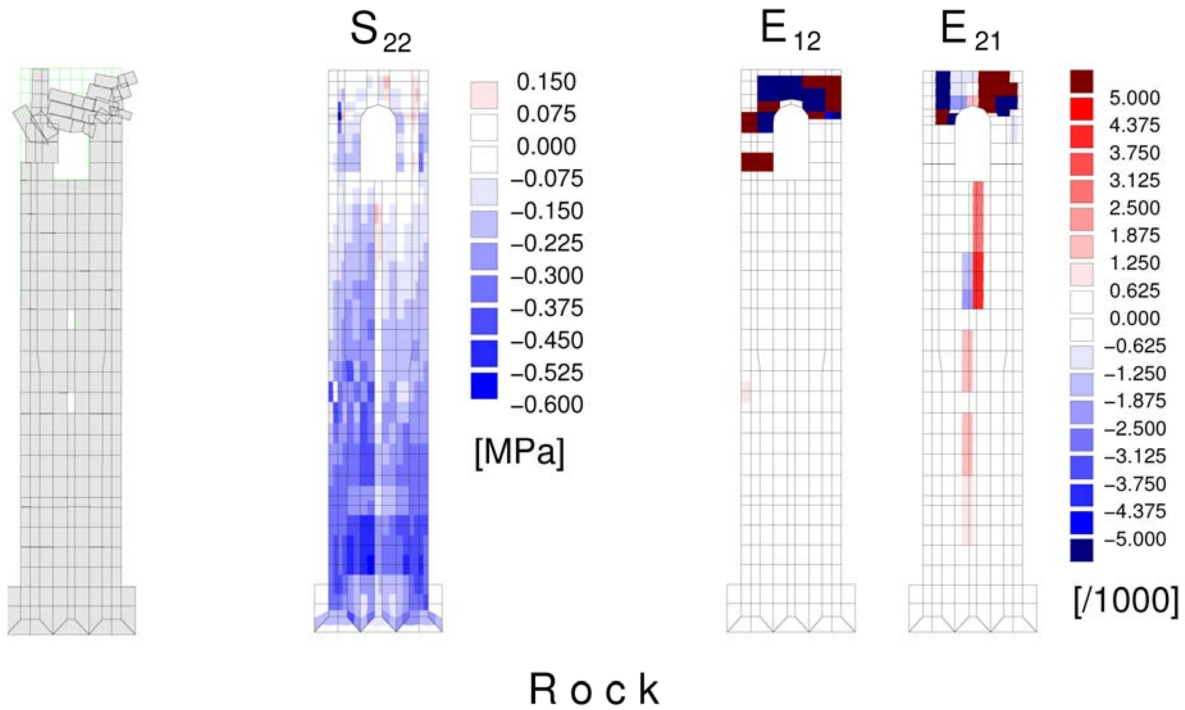

Figure 9 Results of the non linear dynamic analyses for the scheme 1– rock soil. From left to right: final deformed shape; map of the shear deformation E_{12} and E_{21}; map of the S_{22} stress component.

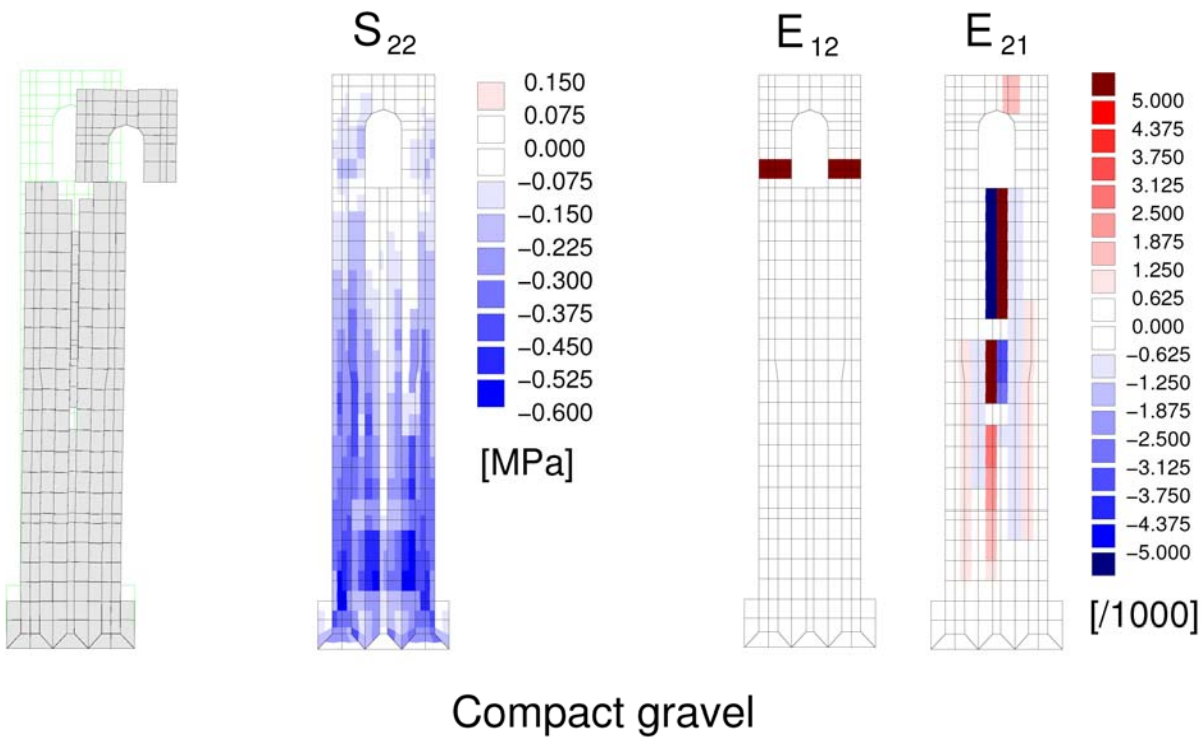

Figure 10 Results of the non linear dynamic analyses for the scheme 2 – compact gravel soil. From left to right: final deformed shape; map of the shear deformation E_{12} and E_{21}; map of the S_{22} stress component.

5. FINAL REMARKS

The objective of the study is to investigate how important is to explicitly model the volume of the foundation soil in the non linear dynamic analyses of masonry towers. To this aim, a numerical model explicitly including a significant volume of soil has been implemented, and two different ground types considered: rock and compact gravel, performing the non linear dynamic analyses for artificial design consistent accelerograms corresponding to a $PGA = 0.15\ a_g$. The results have provided some interesting preliminary indications. After checking that in the scheme with the rock soil the response of the elevation structure corresponds to that of the cantilever model, a softer soil (compact gravel) has been considered. By comparing the results in terms of final deformed shape, map of the stress and strain components, it has been observed that the different characteristics of the ground have important effects on the structural response of the tower under the dynamic seismic loads. A first effect is the increase of the natural vibration period (the first two flexural modes have been compared) in the presence of softer soils, as was actually to be expected. The most important observation, anyhow, regards the distribution of the damage after the dynamic history and the related failure mechanisms, that are significantly different for the two schemes. In particular, in the presence of the softer soil (compact gravel), the development of a global shear failure of the tower is observed in addition to the local damage in the belfry, which was instead the only present in the scheme with the rock soil. Also the specific features of the two belfry’s mechanisms are significantly different. After this preliminary investigation, it seems that the choice of considering the presence of the soil volume in the dynamic numerical model has a great importance with regard to the structural performanceand should not be neglected when performing a seismic assessment of a historical masonry structure.

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