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NON-LINEAR DYNAMIC ANALYSIS OF MASONRY TOWERS UNDER NATURAL ACCELEROGRAMS ACCOUNTING FOR SOIL-STRUCTURE INTERACTION

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Abstract. *The object of the paper is the influence of the soil-structure interaction on the dynamic response of masonry towers, 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 analysed, 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, in which the structure is idealized as a mechanism made of rigid elements connected each to the other by axial and shear springs. The nonlinear behaviour is lumped into the springs assigning proper constitutive laws able to model the significant inelastic aspects of the constitutive behaviour and the meso-scale damage mechanisms with a moderate computational effort. Two types of foundation soil have been considered in order to perform the dynamical analysis accounting for the soil-structure interaction: rock and deposits of compact gravel. For both models, non-linear dynamic analyses have been performed adopting natural records having different characteristics (with regard to the frequency content; distance from the epicentre and type of soil). Some interesting considerations are derived from this comparative study about a problem that is very actual for those who deal with non linear dynamics of structures, but yet is not much explored.*

1 INTRODUCTION

The paper deals with the non linear dynamic modelling and analysis of slender masonry bell-towers with a specific focus on two aspects which have a fundamental influence over the structural seismic response of this specific typology: investigation of the non-linear dynamic response under natural accelerograms, including the vertical component; effect of the soil-structure interaction.

A main issue in the seismic behaviour of slender masonry towers is the influence of the axial stresses induced by gravity loads, which are often close to the compression strength limit of the masonry material. Considering that historical masonry is typically characterized by complex geometry, irregularities and a high degree of inhomogeneity, stress concentrations can easily occur, which could even trigger a local collapse. Thence, the structural failure can be driven even by a moderate increase in the stress level, which can occur under seismic events or under long term loads. It is clear, therefore, that masonry towers are vulnerable also to low-intensity earthquakes, since static vertical loads combine with the dynamic loads induced by the ground motion.

In Italy, bell towers are a quite widespread architectural element, with peculiar morpho-typological characters, according to the geographic area. The examination of the documentation about the damage caused by the 1976 Friuli earthquake [1] 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 [2].

Moreover, during strong earthquakes shear damages are also often observed, and in this case the loss of validity of the Eulero–Bernoulli hypothesis of plane cross-section can significantly affect the overall response of the structure. Specific approaches are needed on order to deal with these aspects, and in particular it is paramount to model the non-symmetry in tension and compression that characterise the masonry material of which these structures are composed.

Finally, the investigation of the response to very strong earthquakes requires to describe the effects of the damages that cause a remarkable reduction of the material stiffness, as well as the energy dissipation through plastic deformation [3]. 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 also required in order to investigate the global shear damage effects [4, 5].

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 [2, 6, 7, 8, 9]; experimental testing methods and structural identification [10, 11] A significant case is that of the Civic Tower of Pavia, Italy (about 900 years old), suddenly collapsed on 17 March, 1989 [12], 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 [1] has pointed 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 [2, 7]. 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 EuleroBernoulli 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 [4, 6]. 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, which has been developed both for the out-of-plane behaviour [5, 13, 14] and for the in-plane behaviour [4, 5] here applied. The application of the herein described RBSM model is particularly effective since it allows to considerably reduce the computational burden while keeping, at the same time, the information about the meso-scale effects of the masonry behaviour. It has been , in fact, fruitfully employed both for the analyses of complex, real buildings [15, 16, 17, 18] and when it is used to perform a large number of analyses for calibrating simplified macro-models of masonry panels [19, 20].

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 [21].

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.

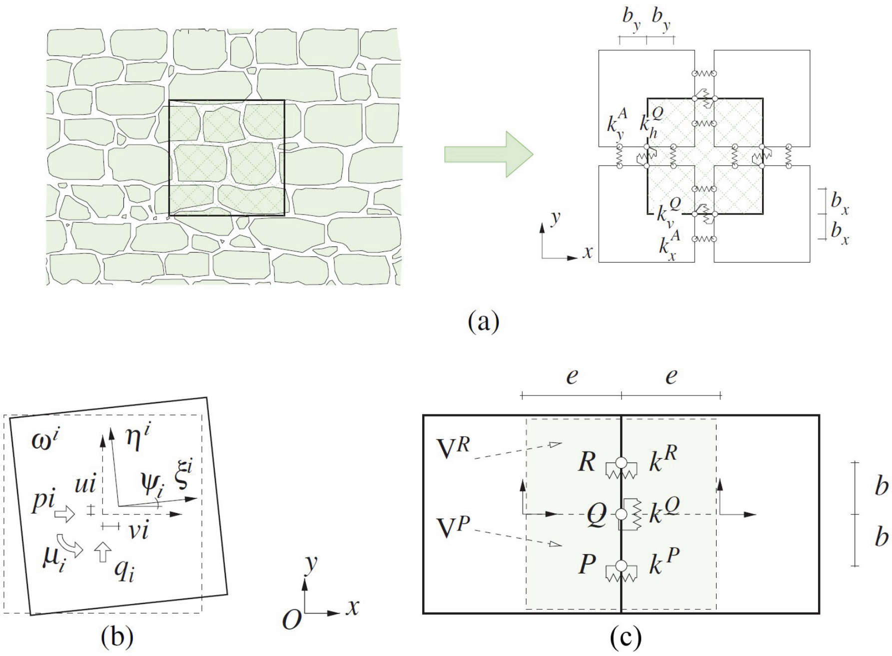

Figure 1: Scheme of a unit cell defined by four rigid elements(a), kinematics (b), disposition of the connecting spring-devices between a couple of rigid elements (c) for macroscale modelling of a representative volume of masonry [5, 21].

Two axial, one symmetric shear, and two in-plane flexural loadings are considered for the parameter identification. 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 mechanism maintains the initial connectivity in order to reduce the computational effort.

3 THE REFERENCE TOWER

An idealized case study has been considered [7], in order to assess some basic and common features of the structural response to seismic actions and to appraise the performance of the approach proposed for the modelling. Such a reference model has been supposed to be structurally independent, i.e. with no adjacent interacting construction, and characterized by geometrical regularity both in plan and in elevation (Fig. 3, 4). The dimensions (see Table 1) were chosen by looking at a number of significant examples [1, 9, 8], in order to represent an average masonry bell tower located in seismic zones of Northern Italy, without the intent,

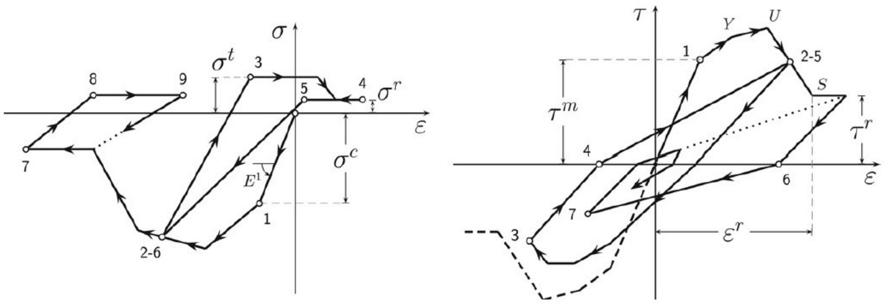

Figure 2: Schematic representation of the hysteretic rules for the axial (on the left), and the shear connecting springs (on the right).

of course, to cover all the possible situations. The geometry of the model was simplified by disregarding the typical structural details, like – for example – the presence of internal vaults.

On the basis of these observations, it was designed an ideal tower having a 5.30 x 5.30 m square base, with a wall thickness varying from 1.00 m at the base to 0.85 m at the top, and having a total height of 28.50 m. In Figures 3 and 4, the 3D drawings and the schematic sections and plans of the tower are shown, and the geometrical characteristics are summarized in Table 1.

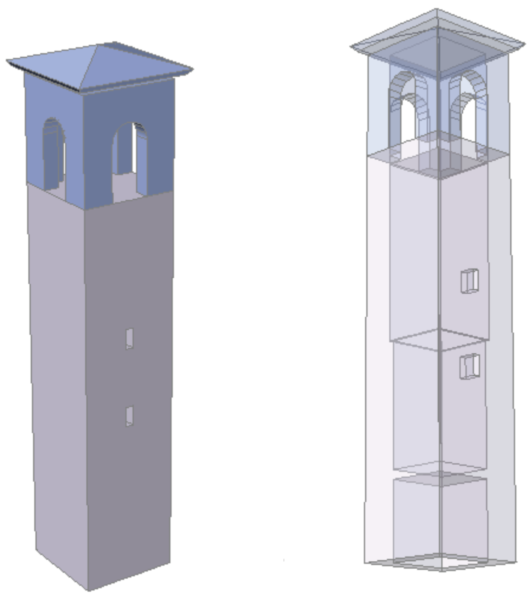

Figure 3: 3D drawing of the reference tower.

3.1 Mechanical parameters of the masonry material

According to the constitutive model adopted for the axial and shear springs (Par. 2), a set of parameters are needed in order to define the corresponding skeleton curves and hysteretic rules

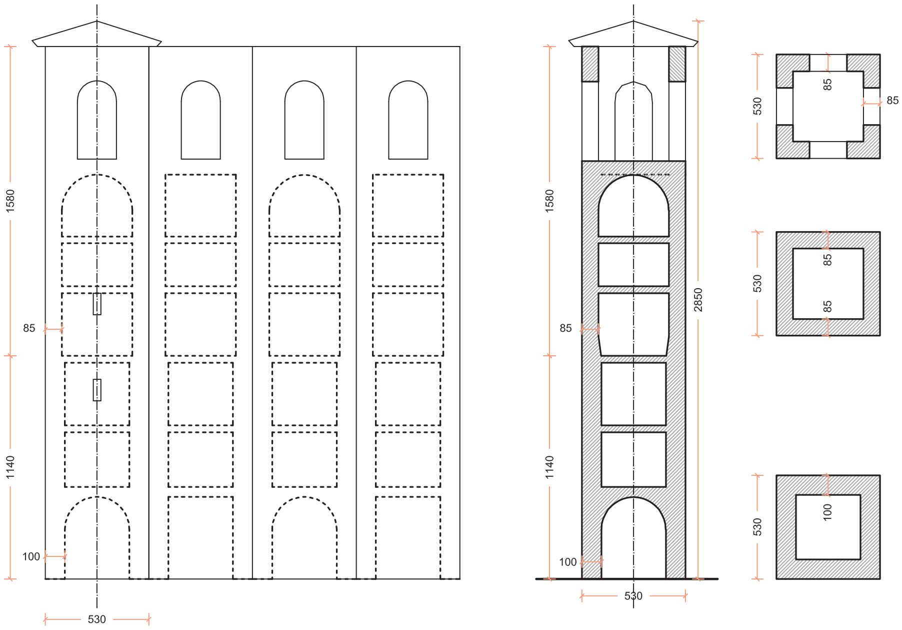

Figure 4: Schematic prospects, section and plan views of the idealized case study.

Table 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

[4].

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: 0.88 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.

3.2 The model of the foundation soil

3.2.1 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 a type B 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: $E_A = 1400\ MPa$; $G_A = 584\ MPa$; $E_B = 700\ MPa$; $G_B = 292\ MPa$.

3.2.2 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 $\lambda^{2\ Hz}$ 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:

$$\nu_s = \sqrt{\frac{G}{\rho}}; \quad G_A = 584\ MPa \rightarrow \lambda^{2\ Hz} = 285\,m; \quad G_B = 292\ MPa \rightarrow \lambda^{2\ Hz} = 198\,m; \quad (1)$$

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 - see Par. 3.2 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) with a reduced stiffness of the vertical 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.

4 Dynamic analyses with natural accelerograms accounting for soil-structure interaction

Two numerical models of the reference tower have been analysed (see Fig. 5) by using the two soil types defined in Par. 3.2. In the following paragraphs, the discussion of the case study will be made with reference to the models with the soil volume: Type A soil (rock) and B type soil (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 set of natural accelerograms has been chosen in order to represent the ground motions. Actually, the aim of the paper is not to perform an engineering safety assessment of the tower, nor to perform a statistical analysis on the base of a large number of analyses. The objective is rather to perform a preliminary investigation for improving the current framework about the seismic assessment of historical towers, by analysing two problems: use of natural accelerograms, including the vertical component; effect of the soil-structure interaction.

Starting from the idea that non linear dynamics is the most complete and realistic method for the structural analysis under seismic actions, particularly for special types of buildings like these, it is necessary to overcome a number of drawbacks, that actually make the application

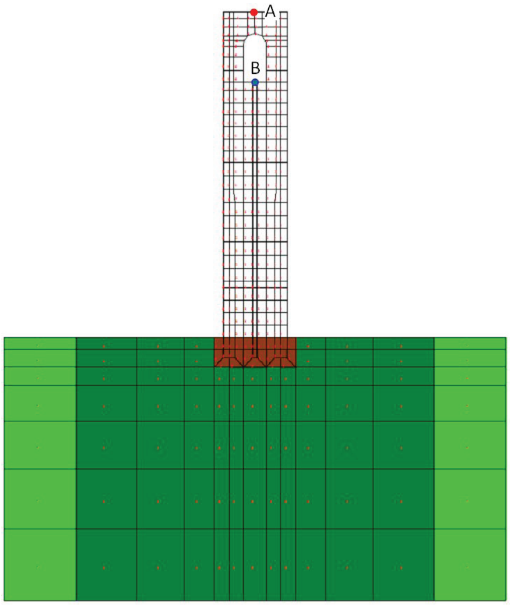

Figure 5: Mesh of the tower with the significant volume of soil included in the model, and location of the control points A and B.

of the approach quite limited. The two above mentioned points (which are not much investigated) are indeed critical, since concerns some of the major criticisms raised against non linear dynamic analyses: proper choice of the accelerograms in order to be truly representative of the seismic hazard (in this sense, the use of artificial accelerograms is often considered to be a purely conventional "domesticated" option); sensitivity of the method to the presence of large soil masses participating to the motion. With regard to the second point, it has been shown in [22] that the different characteristics of the ground have important effects on the structural response of the tower under the dynamic seismic loads both on the distribution of the damage and on the related failure mechanisms. Thence, the incorporation of the soil volume in the dynamic numerical model has a great importance with regard to the structural performance and should has been considered in the models assumed in the present study.

4.1 Choice of the natural records

On the basis of the above mentioned remarks, a relatively limited groups of accelerograms has been used (6), leaving apart the specific requirements of the Building Codes with respect to the deviation from the design spectrum. The idea, instead, was of considering a set of case studies representing a real, coherent seismic event. The records used in the analyses are listed in Table 2.

4.2 Numerical results

For each of the two models, the registrations of three seismic events at three stations have been considered, as shown in Table 2. For each of them, three records are available: horizontal component in the North-South direction (NS); horizontal component in the West-East direction (WE); vertical component (UP).

Since the problem is analyzed by means of a plane model, a group of accelerogram shall be composed of two components, a horizontal one plus the vertical one. Thence, the available

Table 2: Selected station and recordings [23, 24].

Event station/EC8 soil	component	Date / UTC time	Event code	R_{epi} [km]	PGA [cm/s^2][cm/s]
Friuli 3^{rd} shock	NS				258
FRC (Forgaria)	WE	1976-09-15 / 03:15:18	FR2-76b	17.3	211
soil B	UP				96
Umbria-Marche $1^{st} shock$	NS				332
CLF (Colfiorito)	WE	1997-09-26 / 00:33:12	UMM-97a	2.8	284
soil D	UP				236
Umbria-Marche $1^{st} shock$	NS				486
NCR (Nocera Umbra)	WE	1997-09-26 / 00:33:12	UMM-97a	13.1	263
soil E	UP				143

registrations have been combined in order to obtain 6 different groups of accelerograms to be applied to each of the two computational models of the tower (Type A soil - “Mat3b”; Type B soil - “Mat4b”). In order to appraise the effects of the presence of the vertical component of the accelerogram, all the analyses have been also performed by considering the horizontal component only. A total number of 12 nonlinear dynamic analyses has been therefore performed for each of the two soil cases.

In this paragraph, due to space limitations, the most interesting and representative results have been selected. In the Figures 6, 8, 10, 7, 9, 11, results are reported in terms of deformed shape, map of the shear deformation E_{12} and E_{21}, map of the S_{22} stress component (all plotted at the end of the time history) for the two cases: combination of the horizontal and vertical components simultaneously applied (bottom of the figure); horizontal component alone (top of the figure). In the center, the two displacement histories are plotted and compared in the same graph.

5 DISCUSSION OF THE RESULTS AND FINAL REMARKS

Among the whole set of analyses, the results relative to three groups of accelerograms have been selected, all belonging to 2 distinct events (Friuli Earthquake, 1976, third shock, Forgaria Cornino Station; UmbriaMarche Earthquake, 1997, first shock, Station of Nocera Umbra):

C1. Fr2-76b FRC NS (with and without the vertical component “UP”);

C2. Fr2-76b FRC WE (with and without the vertical component “UP”);

C3. UMM-97a NCR NS (with and without the vertical component “UP”).

It is possible at this point to perform some interesting observations for the cases of analysis of mentioned, comparing for each of them the results obtained with or without the presence of the vertical component of the recording. A further element of comparison is then represented by the results for the same series of analyses made for the second category of soil, through which are highlighted specific aspects of soil structure interaction analysis in nonlinear dynamics.

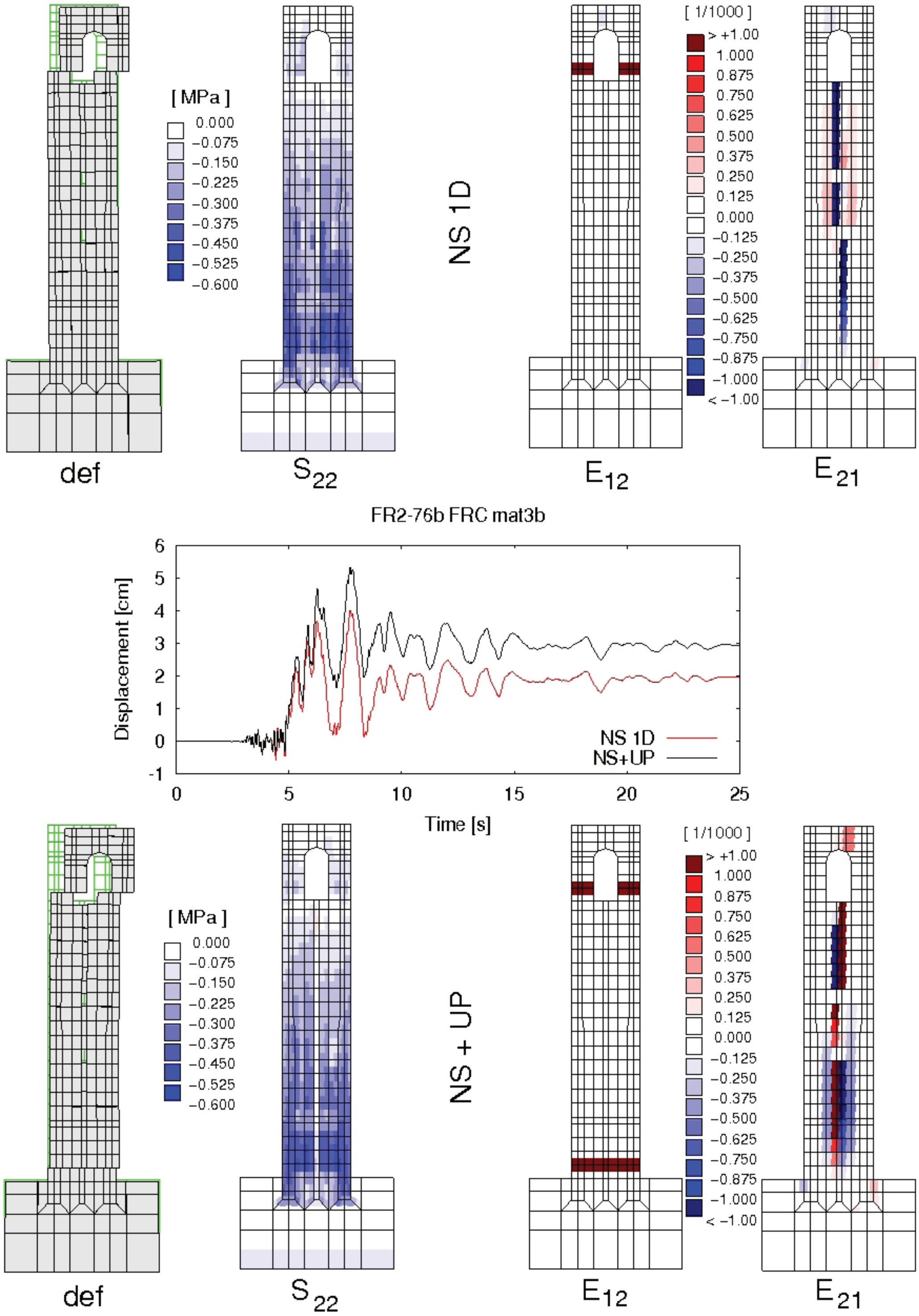

Figure 6: Results of the non linear dynamic analyses for the model: soil A rock; accelerogram Friuli, Forgaria Cornino, 3^{rd} shock; NS horizontal component (mat3b FR2-76b FRC NS).

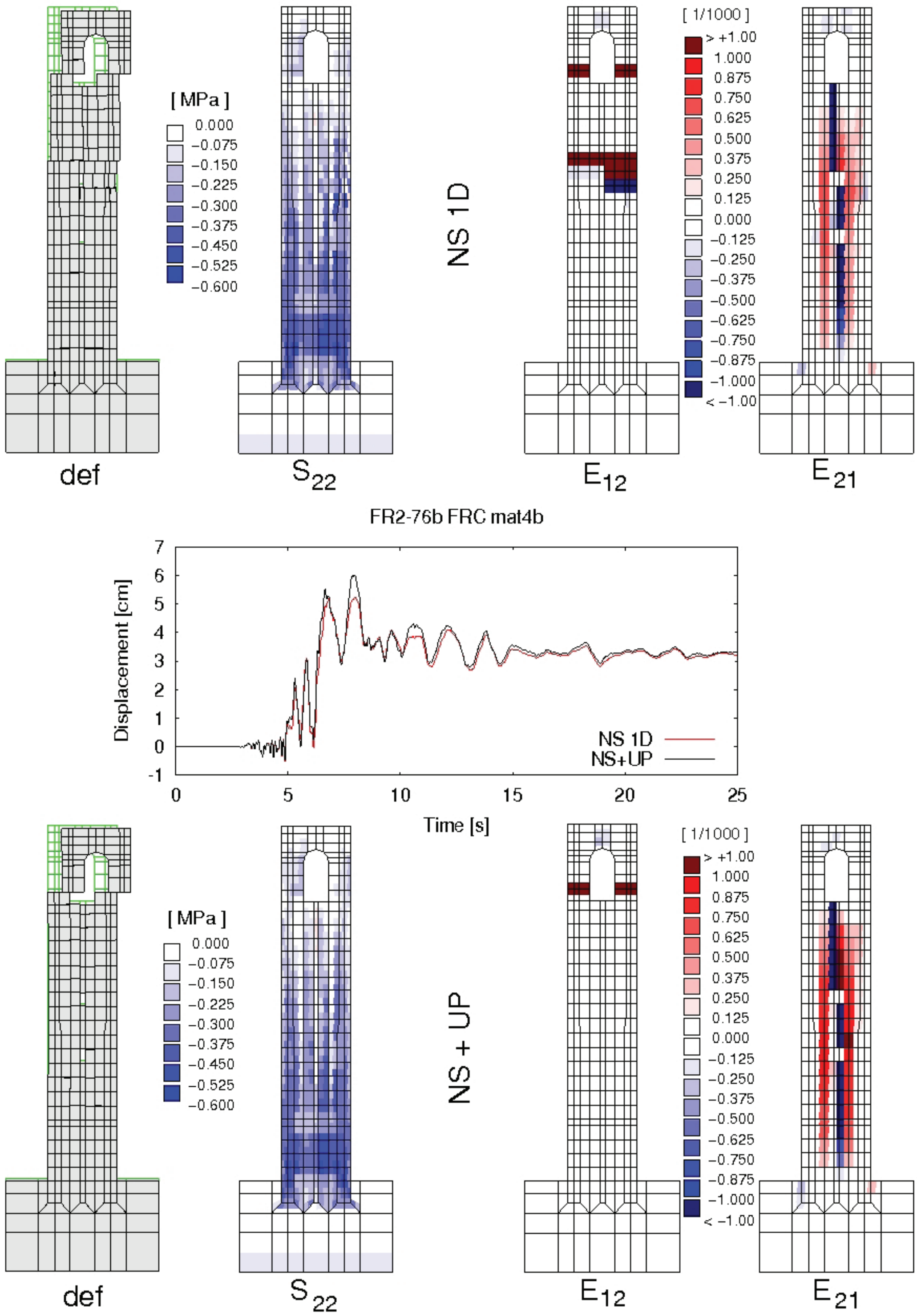

Figure 7: Results of the non linear dynamic analyses for the model: soil B; accelerogram Friuli, Forgaria Cornino, 3^{rd} shock; NS horizontal component (mat4b FR2-76b FRC NS).

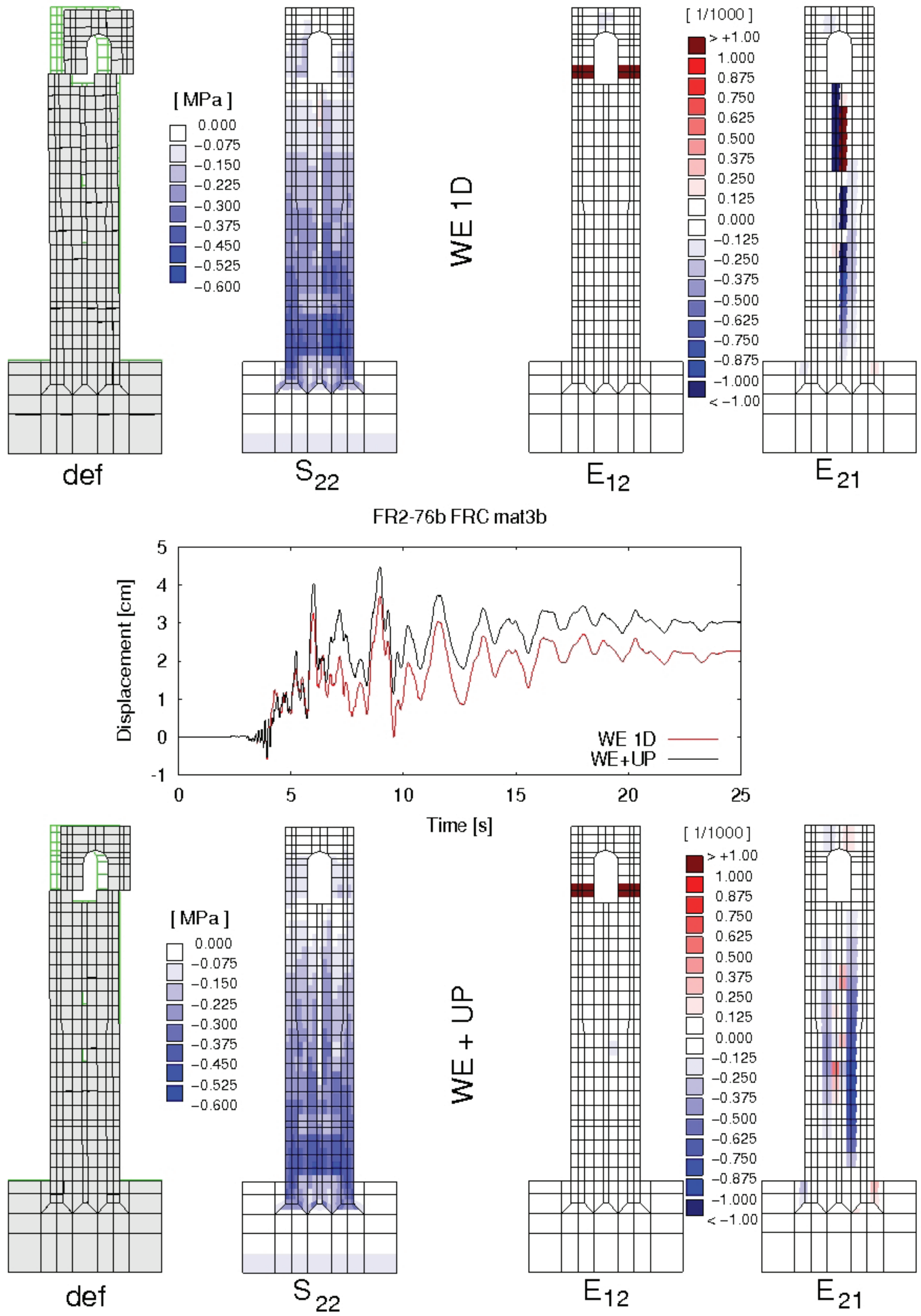

Figure 8: Results of the non linear dynamic analyses for the model: soil A; accelerogram Friuli, Forgaria Cornino, 3^{rd} shock; WE horizontal component (mat3b FR2-76b FRC WE).

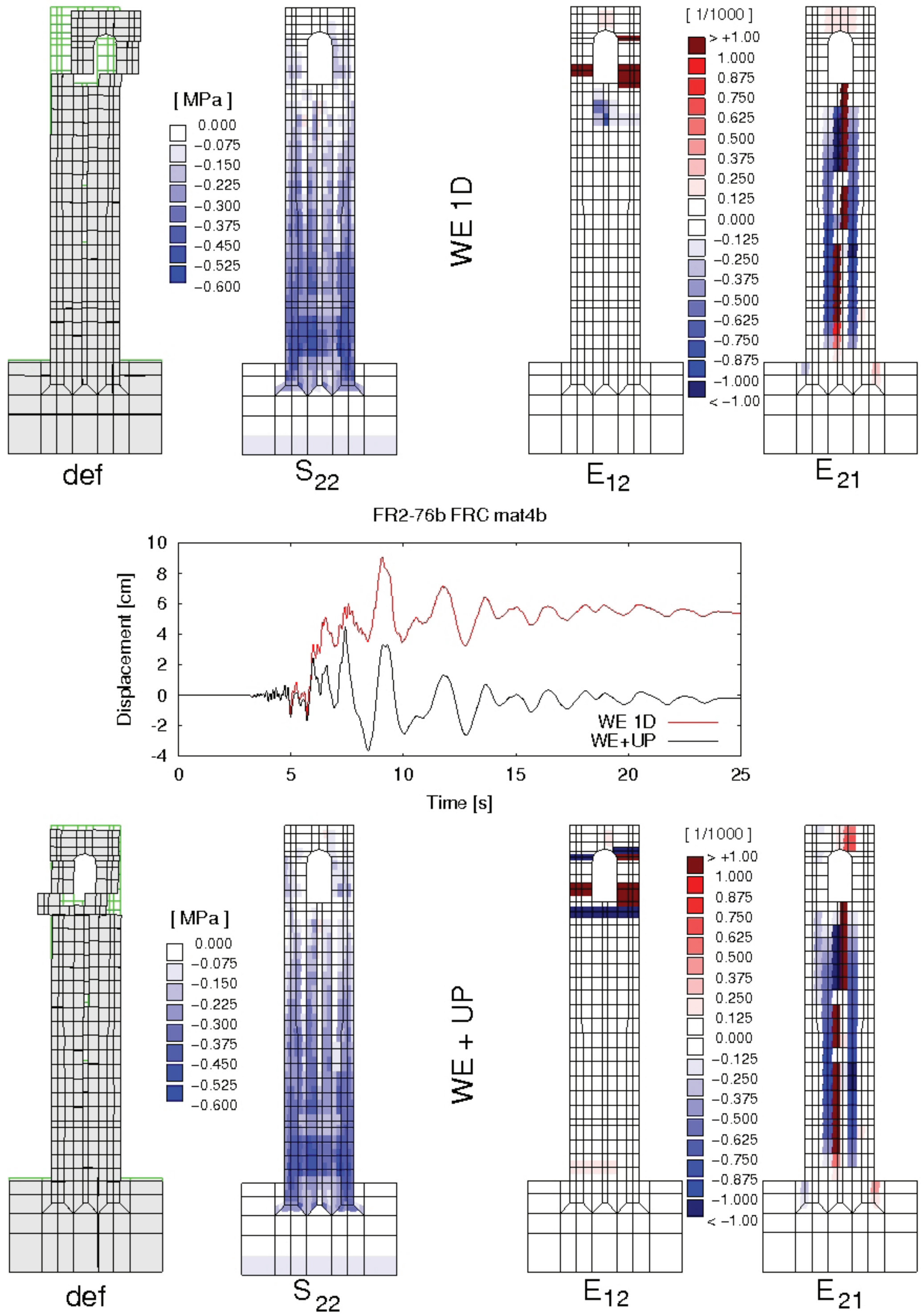

Figure 9: Results of the non linear dynamic analyses for the model: soil B; accelerogram Friuli, Forgaria Cornino, 3^{rd} shock; WE horizontal component (mat4b FR2-76b FRC WE).

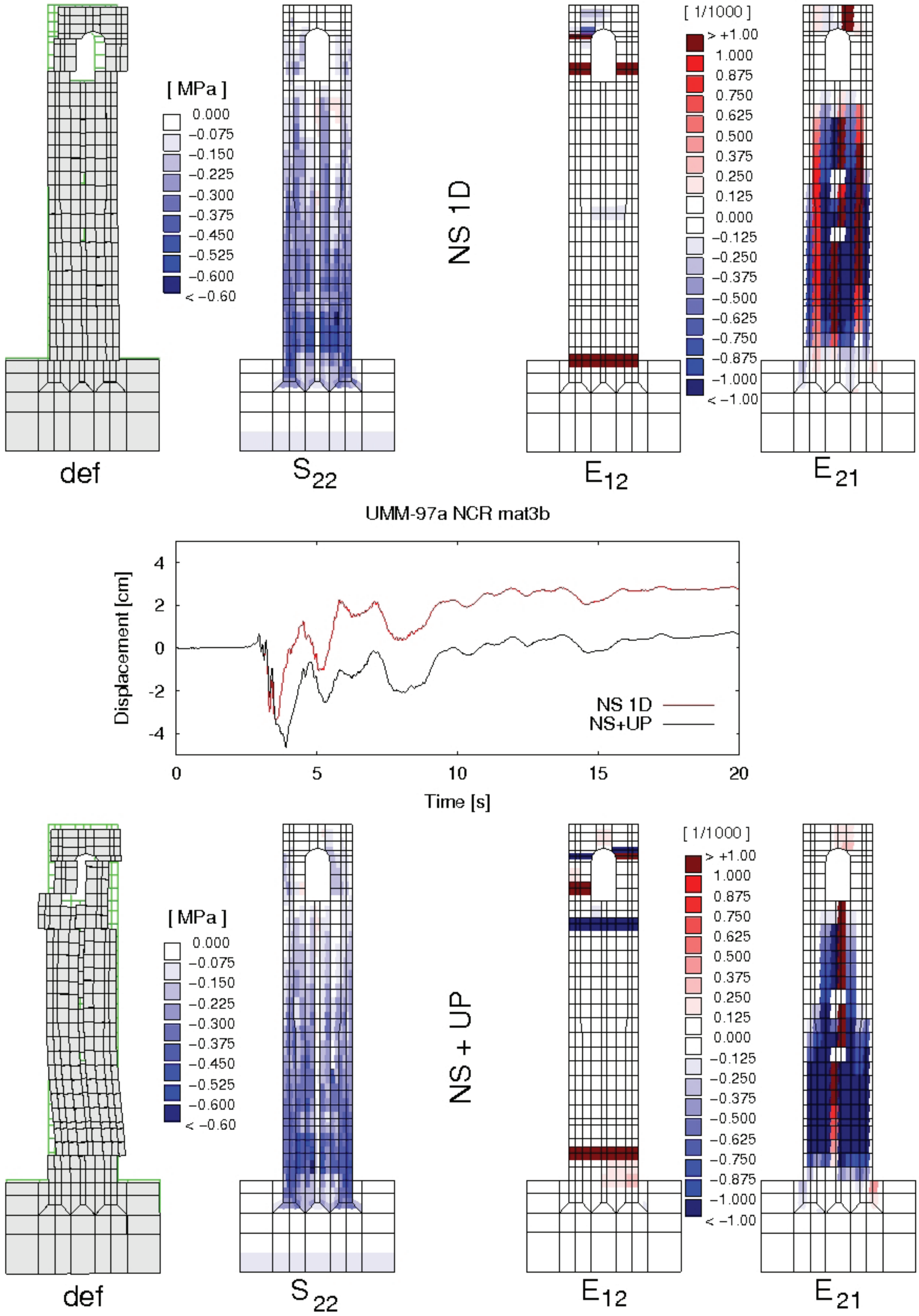

Figure 10: Results of the non linear dynamic analyses for the model: soil A; accelerogram Umbria-Marche, Nocera Umbra; NS horizontal component (mat3b UMM-97a NCR NS).

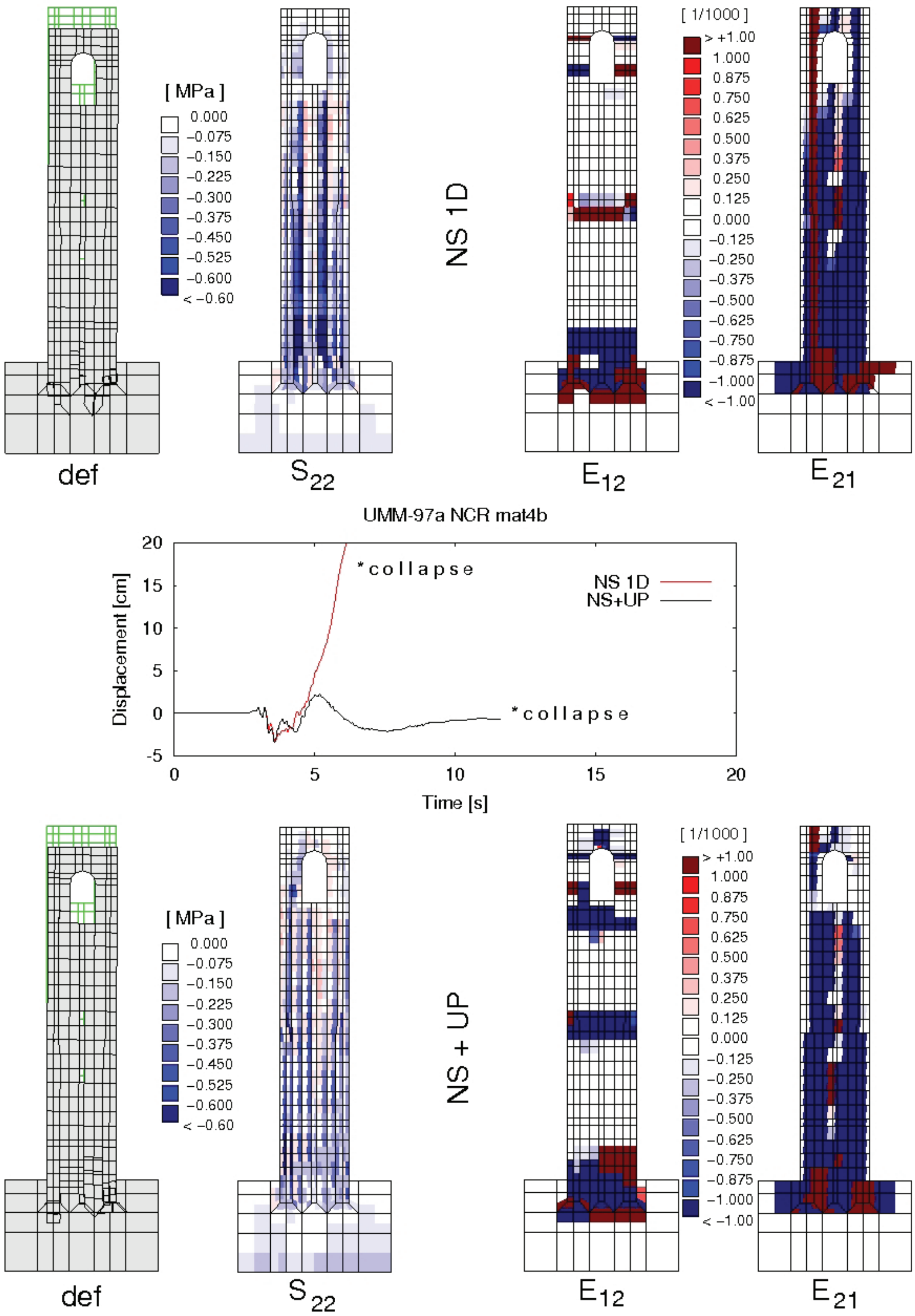

Figure 11: Results of the non linear dynamic analyses for the model: soil B; accelerogram Umbria-Marche, Nocera Umbra; NS horizontal component (mat4b UMM-97a NCR NS).

5.1 Combination C1

Soil type A

The results for the this combination, which is relative to the seismic event Fr2-76-b (NS component + UP component), are reported in Figure 6.

The analysis performed under the horizontal component indicates the presence of a global shear mechanism: the tower tends to be split into two separate vertical portions (see Fig.6, E_{21} map). This is accompanied by a local damage mode involving the belfry, with damage clustered at the base of the belfry (Fig. 6, E_{12} map), and sliding of the whole belfry (Fig. 6, final deformed shape).

When the vertical component of the accelerograms is included in the analysis, it can be observed that, qualitatively, the failure modes engaged are the same but global displacements and the damage level are increased. Moreover, an additional global collapse mechanism is engaged: sliding of the tower, with damage clustered at the base section (see Fig. 6 final deformed shape, E_{21} map and E_{12} map).

Soil type B

In this situation, a damping of the displacement response with respect to the previous case is observed, with a substantially unchanged value of the final displacement. The alterations induced by the presence of the vertical component on the displacement history are negligible, whereas the damage distribution of tensions and the damage pattern are different. The local collapse of the belfry is quite the same, and so the global shear (see Fig. 7 $S22$ map and E_{21} map). An additional partial collapse mechanism is engaged, with the sliding of the upper part of the tower, above the level of the upper window (see Fig. 7, final deformed shape, E_{12} map).

5.2 Combination C2

Soil type A

The results for the this combination, which is relative to the seismic event Fr2-76-b (WE component + UP component), are reported in Figure 8. In this case, there are two failure mechanisms, at the end of the analysis: global shear mechanism (very similar to that of combination C1 (see Fig. 8, E_{21} map); local collapse of the belfry, with damage clustered at the base of the belfry (see Fig. 8, E_{12} map). The behaviour in the presence of the vertical component is qualitatively the same, with an increase of the displacement. The distribution of damage is not significantly altered and, indeed, the global shear damage is even lower, whereas the damage is more concentrated in the local belfry mechanism, whereas the global shear damage (see Fig. 8, E_{21} map and S_{22} map).

Soil type B

What happen, this time, is different than in combination C1. If only the horizontal component is considered, the presence of the softer soil significantly increases the displacements and the damage level. It is interesting to observe, then, the behaviour in the presence of the vertical component. The displacement history is apparently "regularized". Actually, this is related to the modification of the local mechanism of the belfry, that is much more damaged (see Fig. 9, final deformed shape and E_{12} map), with two sections subjected to sliding. The global damage level, concerning a shear type failure, is increased.

5.3 Combination C3

Soil type A

The results for the this combination, which is relative to the seismic Event UMM-97a (NS component + UP component), are reported in Figure 10. The analysis has revealed some peculiar and interesting features. Despite a displacement range that is not much large, a sever damage is suffered by the tower, and almost all the possible failure modes previously observed are simultaneously present: global shear with sliding at the tower base, local collapse of the belfry at different sections (base of the upper pillars, base of the belfry, lintel; see Fig. 10: final displacement, E_{12} and E_{21} maps).

Soil type B

This time, there is a full structural collapse (see Fig. 11, displacement history and tensional maps). The severe level of damage is mainly related to the global shear mode and collapse at the tower base, whereas the belfry damage is more limited. The situation is quite similar with or without the vertical component, even if the vertical component actually accentuates a failure at the mid section of the tower and at the belfry base, more then at the shear mechanism (see Fig. 11, E_{12} and E_{21} maps).

5.4 Final Remarks

The range of possibilities which have been observed is quite diversified. Even if a clear, univocal trend cannot be identified, it should be observed that the incorporation of the vertical components of the accelerograms in the seismic analysis is very important, since it induces relevant effects on the structural response and on the final damaged configuration. Indeed, it significantly influences the seismic safety verification. The most important remark to be done, however, is that it cannot a-priori established if the effect is significantly worse for the structure and above all, which structural part and local mechanisms will be involved. The final situation will depend on the frequency content of the accelerograms and must be specifically examined case by case. It can therefore be concluded, on the basis of this set of analyses, that for the specific typology considered (historical, slender masonry towers), it is highly recommend to resort to put into the field an advanced and complete method of analysis like the nonlinear dynamics in order to have the possibility of capturing the complexity and variability of the structural response. The present paper offers a first glimpse over the issue, which shall be further investigated in two directions: providing a wider set of results with regard to a statistically significant number of input accelerograms; considering a wider range of examples, also concerning real cases of towers collapsed after seismic events.

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