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NON LINEAR MODELLING OF FORNASINI TOWER AFTER THE 2012 EMILIA EARTHQUAKE (ITALY)

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Abstract. *After the seismic sequence that hit the Emilia Region on May 2012, serious damage has been observed in historical fortresses, churches, bell towers and towers, that are particularly common in the area. This has been highlighted by some distinctive features of such seismic motion that revealed to be very critical for structures characterized by long periods, that is: a high spectral displacement demand at long periods and a very high vertical component. Within this context, the paper deals with the interpretation and numerical simulation of the seismic response of Fornasini Tower, that suffered serious damage after such earthquake. To simulate and interpret its seismic response, nonlinear numerical simulations are performed by using in an integrate way various modelling approaches, in particular: the Finite Element (FE) and Equivalent Frame (EF) Models. Both the global seismic response and that of cross vaults at the ground floor have been investigated through the use of nonlinear analyses. In particular, the capability of the EF approach to simulate the global response of the tower is investigated by performing both nonlinear static and dynamic analyses and by using results coming from the more detailed FE approaches to calibrate the parameters in elastic phase. To this aim, the comparison with the real damage occurred is essential to verify the reliability of results achieved.*

1 INTRODUCTION

After the recent earthquake sequence that stroked the Italian region of Emilia Romagna on May 2012, several monumental masonry buildings have been severely damaged, specially fortresses, churches, bell towers and towers, that are particularly common in the area [1,2]. This paper presents the study of the seismic response of the Fornasini Tower (Poggio Renatico): a typical medieval masonry tower that suffered several damages after such earthquake and is characterized by a complete asymmetric configuration of the four perimeter walls (due to a different opening pattern) that induced a significant torsional effect.

In particular, to simulate and interpret its seismic response, nonlinear numerical simulations have been performed by using two different modelling approaches: the Finite Element (FE) and the Equivalent Frame (EF) Models. While results of FE models have been already presented by Authors in other papers [3,4], herein the attention is focused on the integrate use of such approaches in particular to calibrate some of mechanical parameters which the EF approach is based on and to verify its reliability to capture the real global response, by performing both nonlinear static and dynamic analyses. To this aim, the results of FE modal analysis have been used to support the calibration of parameters that affect the elastic phases while those from nonlinear static analyses to compare the resulting pushover curves. Thus, nonlinear dynamic analyses performed through EF constitute the essential tool to validate also the adopted strength parameters by comparing the damage simulated with the real one. Finally, not only the global response of the tower has been considered but also some local effects on the cross vaults that are present at first floor and constitute a relevant feature for the seismic response of the structure.

2 DESCRIPTION OF THE BUILDING AND SEISMIC RESPONSE

Torre Fornasini is a typical brick masonry tower of the 13th century, located in Poggio Renatico near Ferrara, the ancient capital of the Estense Dukedom. It is a three storey tower with a rectangular plan shape (of about 7.50 x 10.49 meters) and it is 17.50 meters high. The external walls, made of fired clay brick masonry with lime mortar, are quite slender (with a thickness in a range of about 0.50-0.80 meters), except the four corners that presents a thickening. As regard the two horizontal intermediate floors, the first level is characterized by two brick cross vaults, while the second one by a light wooden floor. The heavy reinforced concrete roof was built in 1963, during some restoration work following some damage due to a lightning. One of the wall on the long sides (in correspondence to the main entrance at the ground floor) is characterized by the presence of two arched openings, while the other is almost blind (Figure 1).

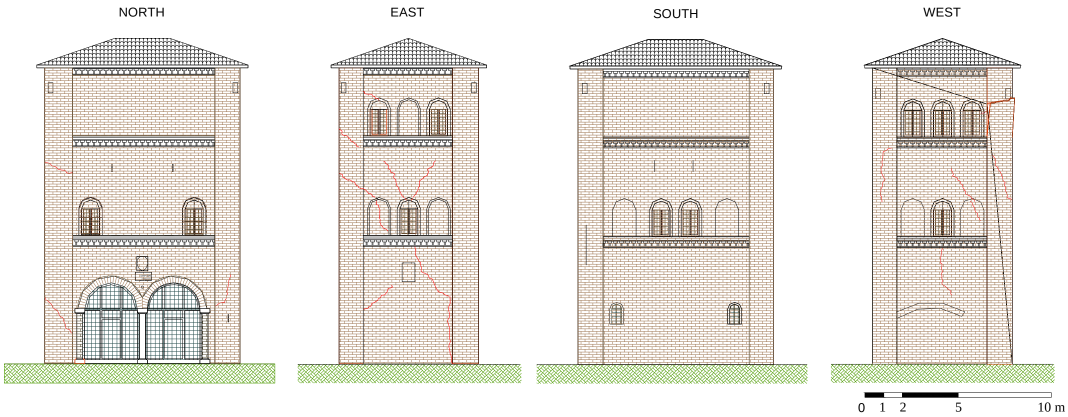

Figure 1. External walls of Fornasini tower and their crack patterns.

This irregularity in plan may be probably one of the cause of a significant torsional response. The walls most significantly damaged have been the East and West walls, showing several diagonal and pseudo-vertical cracks, and the South-West corner (Figure 2b). As concern the horizontal diaphragms, the main damage has been concentrated on the cross vaults. Figure 2c shows the cracks patterns on the extrados that has been clearly revealed after the removal of the upper infill, preliminary step of some strengthening interventions recently carried out on the structure.

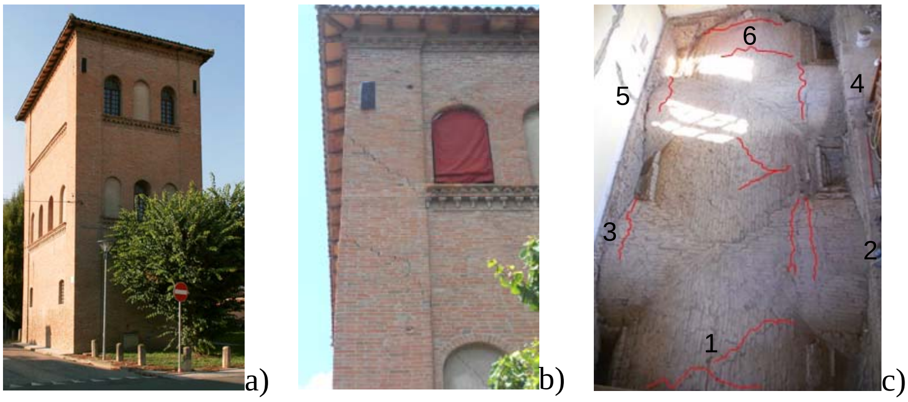

Figure 2. a) An external view of the Tower before earthquake; b) a detail of diagonal cracks on the East wall corner; c) a view of the crack patterns on the vaults extrados.

3 MODELLING APPROACHES

As introduced in §1, the seismic response of the tower has been simulated by performing nonlinear analyses through the adoption of two different modelling strategies: the Finite Element (FE, Figure 3) and the Equivalent Frame (EF, Figure 4) Models.

The FE strategy adopted in this paper is based on a macro-modelling constitutive law [5], that consists in considering masonry units (bricks, stones, etc.), mortar and unit-mortar directly smeared into a continuum (either isotropic or orthotropic), whose mechanical properties are derived from available experimental data fitting. In this case, the nonlinear seismic response of the Fornasini Tower have been investigated by using the commercial code DIANA [6].

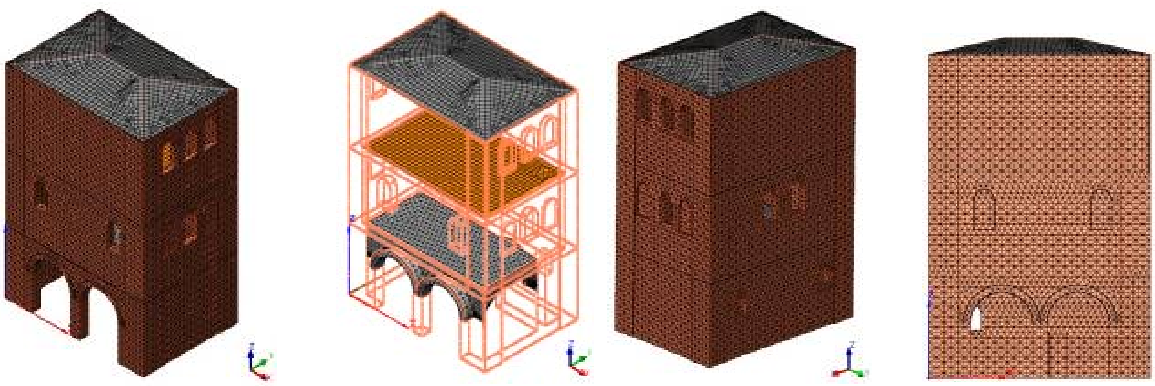

Figure 3. FE discretization of the tower using four-node tetrahedron elements (86713 solid elements and 24189 nodes).

The analysis herein illustrated have been conducted by assuming for masonry a smeared crack total strain model, while in [3] also a multi directional fixed crack approach has been tested by showing a quite low sensitivity to final results on such two different constitutive laws. Although the model is specifically suited for a fragile isotropic material (as is the case of concrete), its basic constitutive laws can be adapted for reproducing masonry properties in the inelastic range. In particular, the “concrete model” basic characteristics well reproduce uniaxial masonry behaviour near collapse, as for instance: tensile failure due to cracking and consequent softening branch; compression crushing failure; strain softening during compression crushing until an ultimate strain value is reached, at which the material totally fails. Figure 3 shows some views of the FE model while Table 1 summarizes the most relevant parameters which the constitutive law and model adopted are founded on. Their definition (at the end based also on values proposed in [7]) has benefited from the support of some experimental tests carried out on the structure before the execution of the aforementioned strengthening interventions. In the FE model the vaults are modeled in detail by taking into account their actual geometry and also the infill contribution. In particular, the role of this latter (as a function of different values assumed of the dilatancy angle) on the response of the cross vaults has been deepened in [4].

Table 1: Summary of parameters adopted for masonry in FE and EF models

FE model		**EF model**				
E [MPa]	1500	**E [MPa]**	2000	**DLs attainment**		
ν	0.2	**G [MPa]**	800	**[%]**	Piers (*)	spandrels
ρ [kg/m^3]	1800	**kr / kel**	0.66/0.7	δ_{E3}	0.6/0.3	0.2
ft [MPa]	0.18	$\tilde{\mu}$	0.35	δ_{E4}	1/0.5	0.6
ε_u	0.0001	$\tilde{c}$	0.11	δ_{E5}	1.5/0.7	2
f_M[MPa]	2.4	**f_M[MPa]**	2.4	β_{E3}	0/15	50
β (**)	0.33/0.05	**f_{bt}[MPa]**	1.2	β_{E4}	30/60	50

Legend:

E= Young Modulus; G= shear Modulus; ν =Poisson ratio; ρ =specific weight; f_t=tensile strength; ε_u=ultimate tensile strain; f_M=masonry compressive strength; β=shear retention factor; $\tilde{\mu}$ =equivalent friction coeffient [12]; $\tilde{c}$ =equivalent cohesion [12]; f_{bt}=tensile strength of blocks[12].

Notes:

(*) The first value is assumed in the case of prevailing flexural behaviour, while the second in the case of the shear one

(**) The 0.05 value has been adopted in the case of FE model analyzed to deepen the vaults response

Differently from the FE model, the Equivalent Frame approach is based on a rougher discretization of the walls by a set of masonry panels (piers and spandrels), in which the nonlinear response is concentrated, connected by rigid area (nodes). In this case, the analyses have been performed by using the Tremuri software [8]. Although respect to the FE model this kind of approach presents a lower degree of accuracy, it has the advantage to perform nonlinear analyses, both static and dynamic, with a less computational effort and also on very complex structures: thus, the validation of its use given by the comparison with more detailed strategies is very useful.

In addition to the different modeling strategy of masonry (at scale of structural elements in the EF model instead of that of material in the case of the FE one), two further issues differentiate the approaches adopted in the paper: i) their capability to consider the combined contribution of the in-plane and out-of-plane response (being this latter neglected at all in the EF model); ii) the modeling of floor elements, in particular of the vaults. As regard this latter, in the Tremuri software diaphragms are modelled as horizontal orthotropic membrane finite elements: thus, they are characterized by an equivalent stiffness aimed to reproduce the global effect of the floor of which the actual geometry and punctual modeling of single elements are then neglected.

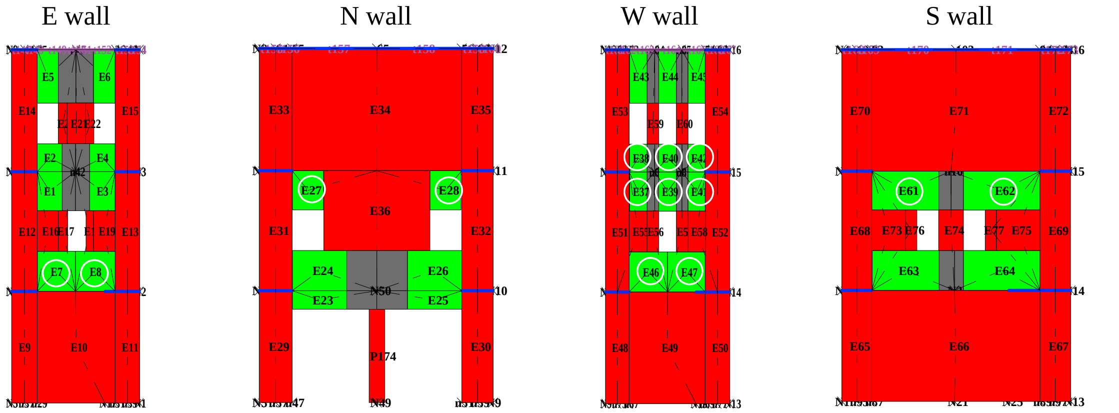

Figure 4. Equivalent frame idealization of the four external walls adopted in Tremuri (piers, spandrels and rigid nodes are marked in red, green and grey respectively).

As aforementioned, one of the crucial points of the EF model consists in a proper equivalent frame idealization of masonry walls. Figure 4 shows that adopted in this case that considers some specific tricks in order to limit some approximations of such approach. In particular: i) all discontinuities in thickness and of structural nature (in particular related to the presence of infilled openings) have been taken into account by defining distinctive adjacent piers (like as the case of elements no. 73 and 76 in the South wall); ii) some very small openings supposed to not affect the crack pattern in piers (as confirmed in this case also by the evidence of the actual damage occurred) have not be modeled (as in the South wall at ground floor); iii) some spandrels with indefinitely elastic behavior (those marked by a white circle in Figure 4) have been modeled in some areas that – following the conventional rules usually adopted – should be assumed as rigid nodes. In particular, this latter trick aims to limit the overestimate in the stiffness that usually can derive by the modeling of rigid portions in the equivalent frame approach: indeed, in the final frames defined such portions (marked in grey in Figure 4) are very limited.

The nonlinear response of masonry panels is modelled by adopting nonlinear beams with a multilinear constitutive law recently implemented in Tremuri [9]. The elastic phase is described according to the beam theory by defining the initial Young (E) and Shear (G) moduli of masonry; then the progressive degradation is computed in an approximate way by a secant stiffness (by assigning a proper ratio k_r between the initial and secant stiffness at the point in which the maximum strength is reached, and a ratio k_{el} between the shear at the end of the elastic phase and the shear strength). The maximum shear strength is defined on basis of common criteria proposed in literature as a function of different failure modes examined (if flexural or shear). In particular, in the case of the flexural response the criteria proposed in [10] and in [11] have been assumed for piers and spandrels, respectively; while in the case of the shear, the Coulomb-type criterion proposed in [12] has been adopted as considered particularly effective for the masonry type examined. This latter is based on an equivalent cohesion ($\tilde{c}$) and equivalent friction ($\tilde{\mu}$) parameters, computed starting from those of mortar joints and including also the effect of masonry texture. Indeed, differently from the flexural response, in the case of shear failure mode, a direct correspondence between the mechanical parameters adopted in FE and EF models cannot be established. The tensile strength adopted in the FE model roughly corresponds to the equivalent tensile strength adopted in the criterion proposed in [13,7], most suitable for masonry with a isotropic behaviour: thus, as unique reference criterion it has been checked that, with the assumed parameters, criteria coherent with [12] and [13] provide similar results in the range of low axial forces (those that characterize the given structure).The progressing of nonlinear response is defined through subsequent strength decay (β_{Ei}) and drift limits (δ_{Ei}), which are associated to the achievement of reference damage levels (i=1,..5). The set of values adopted are summarized in Table 1. A latter issue concerns the role of thickened external piers of the structure: two models have been considered by neglecting (quoted as "pmod") or not (quoted as "pnl") the possible activation of the flexural response for such elements.

As regard the diaphragms, Table 2 shows the parameters adopted for the equivalent orthotropic membranes. Indeed, in the case of vaults two hypotheses have been considered: one representative of a presumable limited stiffness of such elements (quoted as "FV" and calibrated on basis of expressions proposed in [14]); one representative of a more rigid behaviour (quoted as "RV").

Values assumed for E and G of masonry and the RV case of vaults derive from a preliminary calibration of the EF model in the elastic phase supported by the comparison with the results of the modal analysis performed with the FE model. The adoption of higher values of E and G than those assumed in the FE model is not surprising due to the out-of-plane contribution neglected at all by the EF approach.

Table 2: Summary of parameters adopted for diaphragms in EF model

Diaphragm	**E_1 [MPa]**	**E_2 [MPa]**	**G_{eq} [MPa]**	**t [m]**
Cross vaults (FV)	230	230	77	0.12
Cross vaults (RV)	230	230	770	0.12
Timber floor	7800	7800	1000	0.04
Roof	15000	15000	5000	0.06

Legend:
E_1 =Young modulus along the principal direction (floor spanning orientation); E_2 =Young modulus along the perpendicular direction; G_{eq} = shear modulus; t= thickness

4 ANALYSIS OF THE GLOBAL RESPONSE

4.1 Seismic demand

Since the main aim of the paper is to validate the capability of numerical modelling in simulating the actual behaviour exhibited by the tower, the seismic demand used in nonlinear static and dynamic analyses has been defined on basis of some available records of the event that hit L'Emilia in 2012. In particular, two different real accelerograms which were recorded during the May 20th and 29th 2012 earthquake have been used: one in Casaglia (FE), at about few kilometres from Poggio Renatico, while the second one in San Felice sul Panaro (MO). The original records have been then scaled to some plausible values of the Peak Ground Acceleration (PGA equal to 0.91 and 1.5 m/s^2) as estimated for the main shock at Poggio Renatico from the Shakemap Working Group [15]. The dynamic analyses have been performed by applying at the same time X, Y and Z signal components.

4.2 Modal analysis

Table 3 shows the comparison between the first four modes obtained from the modal analysis performed on the FE and EF (RV case) models. A quite good agreement in terms of period values and participating mass can be noticed.

Table 3: Comparison between FE (from [3]) and EF modal analysis results (first 4 modes).

Mode	**1**		**2**		**3**		**4**	
Model	**FE**	**EF**	**FE**	**EF**	**FE**	**EF**	**FE**	**EF**
T [s]	0.281	0.333	0.251	0.326	0.108	0.253	0.088	0.160
M_x [%]	0.003	0.004	66.25	39.65	0.004	0.002	14.92	35.52
M_y [%]	54.67	62.68	0.007	0.001	22.56	18.38	0.067	0.006

4.3 Nonlinear static analyses

In the following the results obtained from nonlinear static analyses are illustrated.

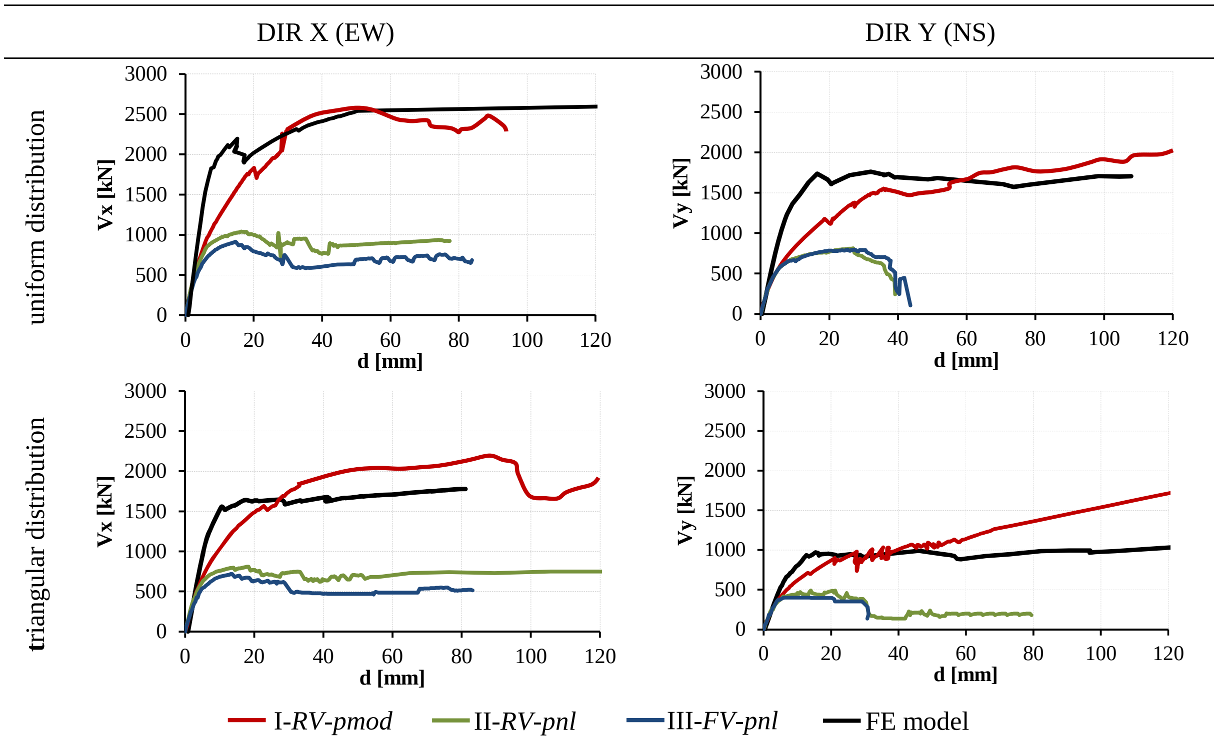

Figure 5. Comparison between FE and EF nonlinear static analyses in terms of pushover curves as a function of two different load patterns adopted and main directions (X and Y).

In particular, in the case of EF approach, according to the different alternative hypotheses assumed for the stiffness of vaults and the possible failure mechanisms considered for the thickened external piers (as introduced in §3), the results of three different models are presented named as "I-*RV-pmod*", "II-*RV-pnl*" and "III-*FV-pnl*". Nonlinear static analyses have been performed on the two main directions (Y and X) and by applying two different horizontal loads patterns: one proportional to the mass (*uniform* distribution), while the other proportional to the mass by the height (*triangular* distribution). Figure 5 summarizes the comparison of results in terms of pushover curves obtained from the FE and EF models; the displacement has been computed as average of that of the nodes located on the top level.

According to the calibration done through the modal analysis, the elastic phase of two approaches is coherent; then, in terms of overall base shear, results of the EF model are in general lower than those of the FE one, apart the I-*RV-pmod* case that provides the best fit among those analyzed. The strength decay in the EF model reflects the adoption of multi-linear constitutive laws assumed for masonry panels. Finally, it can be noticed that an higher difference between FE and EF approaches is obtained in the case of *uniform* distribution than in the triangular one: this is due to the fact that in the EF model forces are concentrated in nodes while in the FE model are more spread.

4.4 Nonlinear dynamic analyses

In order to verify the plausibility of mechanical parameters adopted and of different hypotheses considered in the case of EF model (since in some cases results greatly differ from those of the FE model), results of nonlinear static analyses have been compared with those of dynamic ones. Figure 7 illustrates those associated to cases "I-*RV-pmod*" and "III-*FV-pnl*" (being at global scale the results obtained in cases of "II-*RV-pnl*" and "III-*FV-pnl*" quite similar), by way of example as achieved from the application of Casaglia input with PGA equal to 0.91 m/s^2. The results are expressed in terms of base shear-displacement curves and compared with the pushover curves presented in §4.3. By considering also the evidences from the application of the PGA equal to 1.5 m/s^2 (here omitted for sake of brevity), in general nonlinear dynamic results match better with the backbone associated to the *triangular* load patter.

It can be observed that, even if the case I-*RV-pmod* seems to be more in accordance with the FE nonlinear static results, the results of nonlinear dynamic analyses show that the building still works in the elastic phase in both X and Y directions: this is not coherent with the actual damage occurred. Indeed, the model III-*FV-pnl* results are more reliable and highlight a more significant progressing of the nonlinear response, in particular in the Y direction that is the weakest one. That is confirmed also by the damage pattern simulated as illustrated in Figure 6. It should be noted that the green color indicates the damage level 2 which, however, may be associated to different conditions: it may indicate a panel just beyond the yield condition or close to the damage level 3. By analyzing in detail the drift values achieved in the piers, it can be proved that damage is more widespread and sever at the higher level and in the walls oriented towards Y direction, coherently with the actual one (see Figure 1).

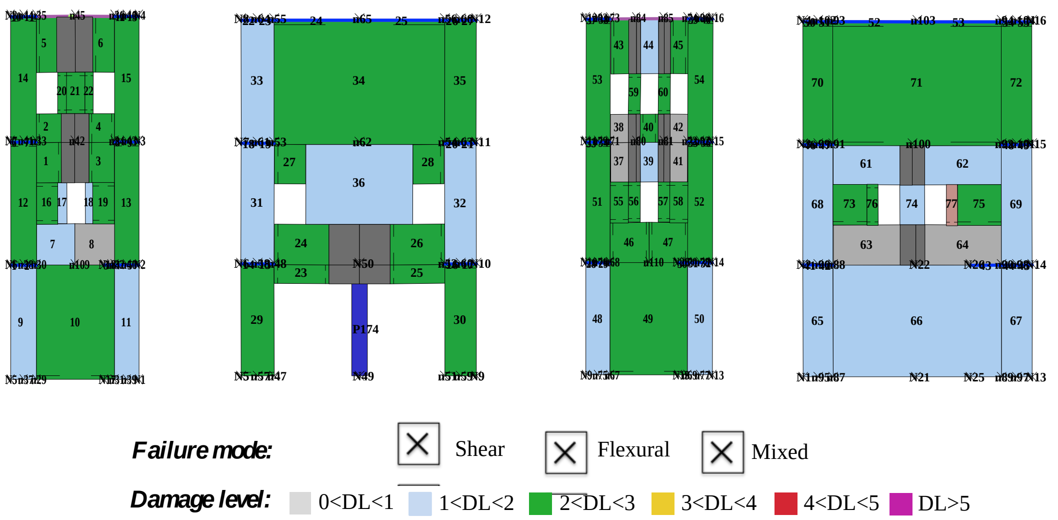

Figure 6. EF Model III-*FV-pnl*: walls damage pattern simulated by nonlinear dynamic analyses (Casaglia, PGA=0.91 m/s^2)

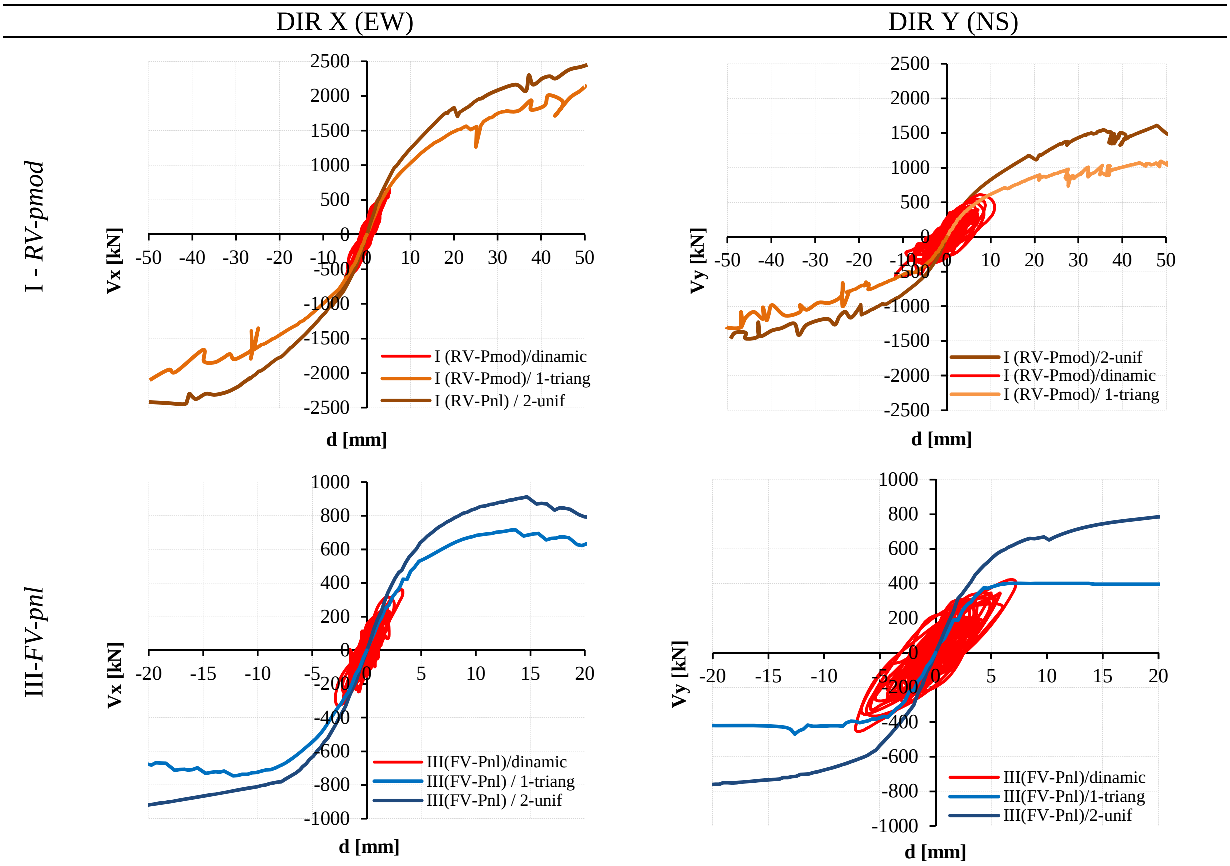

Figure 7. Results of nonlinear dynamic analyses of EF model in terms of base shear-displacement curves (Casaglia – PGA = 0.91 m/s^2)

4.5 Validation of nonlinear static procedures

In order to verify the reliability of nonlinear static procedures proposed in codes in evaluating the performance point (PP) given an assigned seismic demand, results of nonlinear dynamic analyses (in terms of maximum displacement evaluated, represented by the vertical line in Figure 8) have been compared with the prediction deriving from the adoption of overdamped spectra [16]. In particular, the capacity curves have been obtained by converting the pushover curves through the use of the participant factor Γ and the mass m* as proposed in [17, 18].

Overdamped spectra have been computed by using the reduction law proposed in [17] and assuming the following expression for the capacity [19]:

$$\xi = \xi_{el} + \xi_{max}\left(1 - \frac{1}{\mu^{\beta}}\right) \tag{1}$$

where: ξ_{el} is the elastic damping (assumed equal to 5%); ξ_{max} is the asymptote of the hysteretic damping (assumed equal to 20%); μ is the ductility (computed starting from the point marked by a green point in Figure 8); β is a coefficient assumed equal to 1.

Figure 8 shows the evaluation of the performance point through the aforementioned nonlinear static procedure and the comparison with the maximum displacement evaluated from the nonlinear dynamic analyses (represented by the vertical line) in the case of two records considered. In general, in particular in the case of Casaglia seismic demand, results are in a quite good agreement: while it is quite obvious in the case of X direction (that remains in the elastic phase), it represents a worthwhile result in that of Y direction.

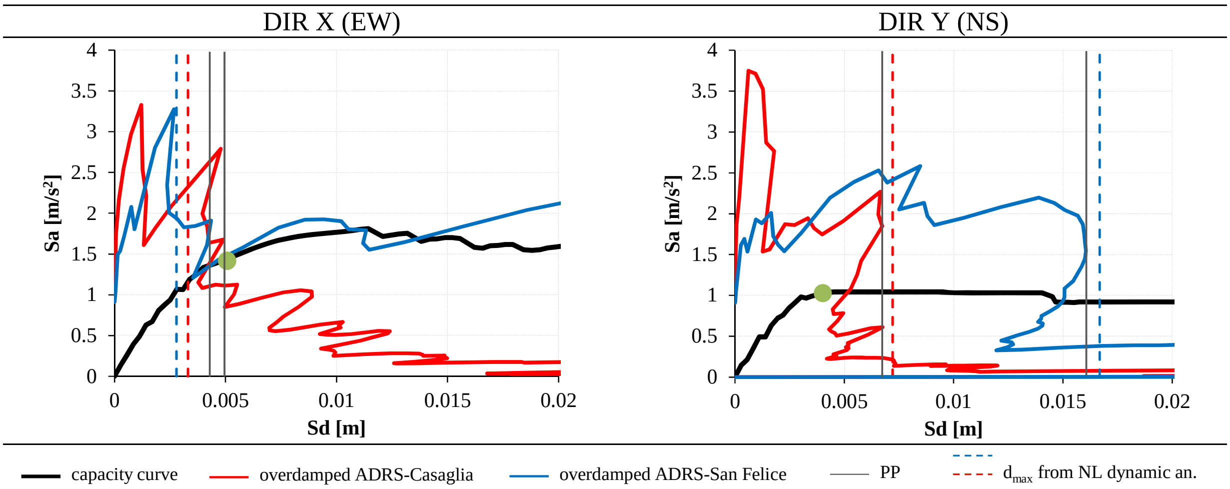

Figure 8. Evaluation of the performance point through nonlinear static procedures based on overdamped spectra (Model III-*FV-pnl* – triangular load pattern) and comparison with results of nonlinear dynamic analyses (PGA = 0.91m/s^2)

5 VAULTS SEISMIC RESPONSE

The seismic behavior of cross vaults at ground floor has been further deepened with a FE model limited to the first level of tower [4] and by processing the results from nonlinear analyses performed (from both such FE model and the EF one) in order to compute the values of angular strain γ attained.

Figure 9 shows the results from nonlinear static analyses performed with the FE model (in Fig.9a also a 3D view of the model is shown) in terms of crack pattern simulated, that matches quite well also with the actual one occurred.

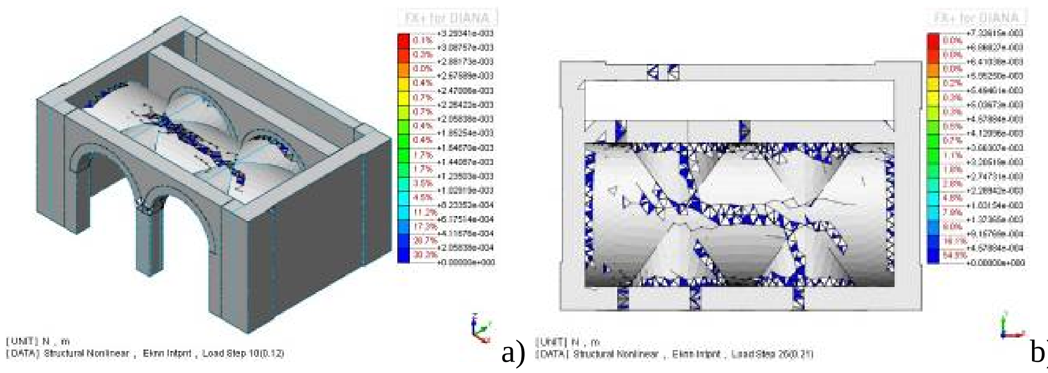

Figure 9. Results of nonlinear static analyses performed through the FE model: crack pattern of the vaults for horizontal loads parallel to Y (NS) (a) and X (EW) (b)

Figure 10 shows the results achieved with the EF model (case III-*FV-pln*) in terms of: evolution of angular strain obtained from the nonlinear static analyses and maximum values obtained from the nonlinear dynamic ones. Vertical lines corresponds to the maximum displacement of the tower resulting from dynamic analyses; it aims to provide an estimate of the reference step to be considered for the comparison with pushover curves. The signs of angular strains (concordant in X dir. and opposite in Y dir.) from these latter are coherent with the main mechanisms activated by the static approach that applies a load pattern according to one main direction and versus (a sketch of such mechanisms is illustrated in Figure 10). Of course, in the case of nonlinear dynamic analyses combined failure modes most properly occur: indeed, results from the dynamic ones highlight a prevailing response in Y direction. As expected, static analyses tend to underestimate the damage with respect the dynamic ones. Values of angular strain are quite low but according to the comparison with those from the SE model and those illustrated in [14] they seem compatible with the spread of a significant damage and degradation of stiffness.

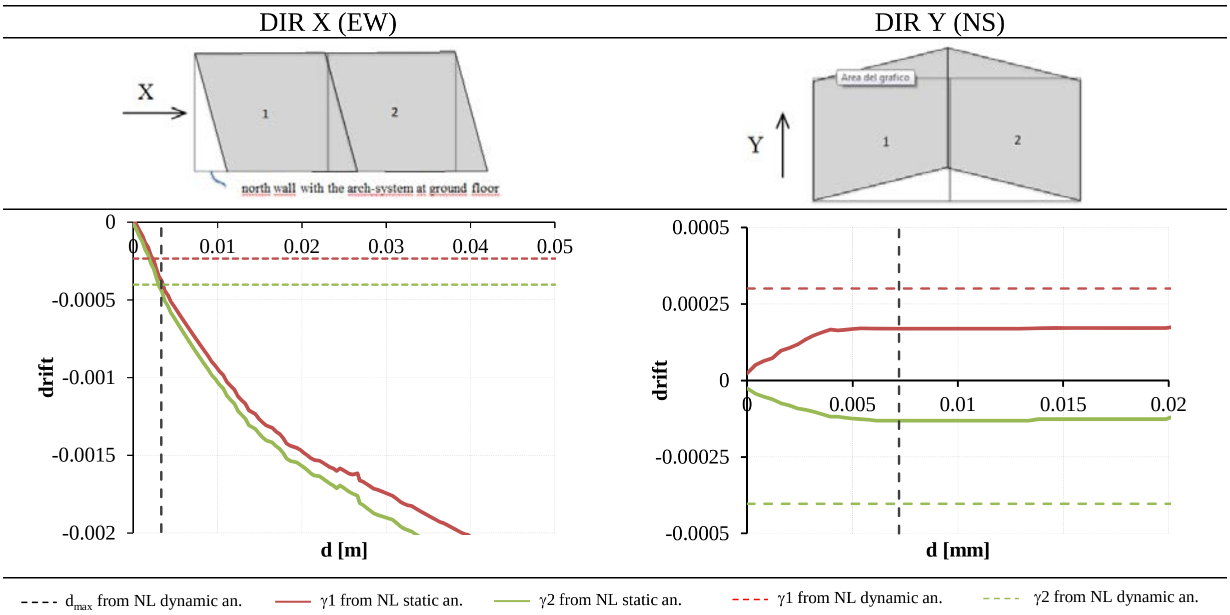

Figure 10. Results obtained from the nonlinear analyses performed with the EF model (case III-*FV-pnl*) in terms of angular strain

6 CONCLUSIONS

The paper illustrates the makings of different modelling strategies in simulating the actual seismic response of Fornasini tower, hit by the 2012 Emilia earthquake; in particular, herein the attention is focused on the integrate use of a more detailed modelling approach (FE) to verify the capability of the equivalent frame one. Both the global response of the tower and some local effects in the cross vaults have been analysed through nonlinear analyses, static and dynamic. Although further developments may be carried out, results are promising in the validation of the use of such approach quite widespread in the engineering practice.

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