PUSHOVER ANALYSIS OF PLAN IRREGULAR MASONRY BUILDINGS

A. Giordano[1], M. Guadagnuolo[2] and G. Faella[3]

[1] *Ph.D., Dept. of Structural Engineering , University of Naples Federico II, Naples. Italy*
[2] *Ph.D., Design Knowledge Dept., Second University of Naples, Aversa, Italy*
[3] *Full Professor, Design Knowledge Dept., Second University of Naples, Aversa, Italy*
E-mail: algiorda@unina.it , m.guadagnuolo@unina2.it

ABSTRACT:

The paper investigates the seismic response of plan irregular masonry building structures in order to evaluate the magnitude of torsional coupling and the applicability of 3D pushover analysis for assessing the behavior under earthquakes. As a test example a simple plan asymmetric two-storey masonry building is selected. The nonlinear dynamic response obtained under both several generated records and selected earthquake ground motions is initially compared with the one obtained for a symmetric building variant. Subsequently, the nonlinear dynamic analysis results are compared with the pushover analysis results. The pushover analyses are performed up to the maximum top displacement obtained by the nonlinear dynamic analyses (measured at the mass center), computed for each earthquake record separately. The deflection profiles and damage at the stiff and the flexible building sides are compared with the peak response obtained by nonlinear dynamic analysis.

KEYWORDS: Masonry buildings, Torsional response, Nonlinear dynamic analysis, Pushover analysis.

1. INTRODUCTION

Most masonry buildings are characterized by irregular configuration. This is mainly due to asymmetric or non-uniform plan arrangement of masonry walls. Besides, building walls are frequently different in geometry from each other since windows and openings are unequal and not aligned. Additionally, irregularities can arise from the presence of vaults and timber floors at the same level. Irregularities in plan and lack of symmetry may imply significant eccentricity between the building mass and stiffness centers, giving rise to damaging coupled lateral/torsional response. Irregularities may also cause stress concentration and local failures since some masonry portions are prone to vibrate separately from the remaining part of the structure. As well, masonry walls may be contemporaneously subjected to in-plane and out-of-plane earthquake forces, and then suffer large damage, or may tend to separate at the building corners, causing collapse of floors.

Seismic codes recognize the importance of torsional behavior and try to reduce torsional effects through design rules concerning building regularity, lateral and torsional stiffness and seismic input. However, as for framed structures, the magnitude of the actual torsional effects may differ from the estimated one conventionally used for design. Therefore, analysis techniques that allow to take carefully into account the torsional behavior and the resulting damage distribution must be used.

The inelastic torsional behavior of structures, as well as the torsional provisions in building standards, was extensively and deeply studied over the last years (Rutenberg 1992, 2002, Sadek and Tso 1989, Jeong and Elnashai 2004, Chopra and Goel 2006, Fajfar et al. 2006), but, despite the large number of researches, some issues concerning the irregular masonry buildings are not yet adequately investigated. In fact, as for regular buildings, seismic performance of plan asymmetric framed structures mainly relies on a ductile response anyhow, and the plastic mechanisms that can be activated are clearly defined and broadly recognized. Furthermore, the researches are largely based on the response of single-storey asymmetric building models to simplify analyses and to facilitate parametric studies (De Stefano et al. 1993, Myslimaj and Tso 2002, De la Llera and Chopra 1994a, De Stefano and Pintucchi 2004), and the effect of coupling between lateral and torsional motion is frequently studied in terms of element ductility demand. Additionally, most of papers concerning multistory framed buildings are aimed at assessing the capability of seismic code procedure in capturing the actual seismic torsional response and ensuring adequate buildings safety (De la Llera and Chopra 1994b, Perus and Fajfar 1999, Aziminejad et al. 2006, Humar and Kumar 2006). Finally, the improvement of seismic behavior of unsymmetric buildings is frequently based on the optimization of plan distribution of lateral strength among resisting elements, and then on reduction of the strength eccentricity; alternatively, the improvement is achieved through different increases in lateral strength in resisting elements in order to limit the ductility demand.

The seismic behavior of irregular masonry buildings shows some differences with respect to framed structures, and therefore the above approaches may not allow to obtain sufficient information on seismic response, especially if existing buildings have to be retrofit. The irregularity source in masonry buildings are generally different, may have different implication on seismic behavior, and some typical effects in framed structures do not necessarily show up. Therefore, it is necessary to explore the nonlinear behavior of multi-storey irregular masonry buildings under strong earthquakes in order to better understand their torsional response, since no study analyzed exhaustively the nonlinear dynamic response of plan asymmetric 3D masonry buildings in terms of displacement and damage parameters. Moreover, it needs to investigate how to reduce the torsional response in structures characterized by generally not ductile behavior and in which the strength distribution may be difficult to significantly modify, if not even inessential.

Recently, the pushover analysis demonstrated to be an efficient tool to assess the seismic capacity of multistory buildings. Such analysis was originally devised for multistory structures undergoing translation, and subsequently adapted to asymmetric buildings (Faella and Kilar 1998, Moghadam and Tso 2000, Azuhata et al. 2000, Ayala and Tavera 2002, Chopra and Goel 2004, Fajfar et al. 2005). Furthermore, attempts to combine results of non-linear static analysis of MDOF model (to control the target displacements and the distribution of deformation) with the response spectrum analysis of an equivalent SDOF system (to define the torsional amplification) have been also made (Chopra and Goel 2004, Fajfar et al. 2006), but directions and reliability were mainly studied for framed structures.

This paper investigates the nonlinear behavior of a two-storey irregular masonry building, having rectangular floor plan. The building presents stiffness and mass eccentricity due to an asymmetric plan layout of walls. Nonlinear static and dynamic analyses are carried out using a refined finite element approach. The dynamic analyses are performed by using both a spectrum-compatible ensemble of generated acceleration records and El Centro earthquake record. The pushover analyses are carried out using a lateral load distribution proportional to the masses distribution within walls and floors. The magnitude of the torsional response is assessed comparing the asymmetric building response with the one of the corresponding plan symmetric building variant. The effectiveness of the pushover procedure is recognized comparing the results with the ones obtained through the nonlinear dynamic analyses. The comparison is essentially carried out referring to the lateral displacement envelope of wall elements.

2. DESCRIPTION OF THE BUILDING

A two-storey masonry building is selected to investigate the relevance of torsional response in plan asymmetric masonry building structures, as well as the suitability of results from 3D pushover analyses in envisaging their seismic behavior.

Figure 1 reports the plan layout and the 3D view of the building: the structure is rectangular with a 24.00 x 11.10 m building envelope; the two stories are 3.50 m high. The wall thickness is kept unchanged at both the stories and equal to 0.60 m. As shown in Figure 1, the structure is unsymmetric in plan in both the horizontal directions. The asymmetry about the x direction arises from the position of the longitudinal inner wall that is not barycentric. The asymmetry about the y direction is due to the lack of the second-last transverse wall. The building variant that includes this transverse wall is assumed as reference symmetric structure in identifying the response effects due to torsional coupling, when subjected to ground motion in the y direction.

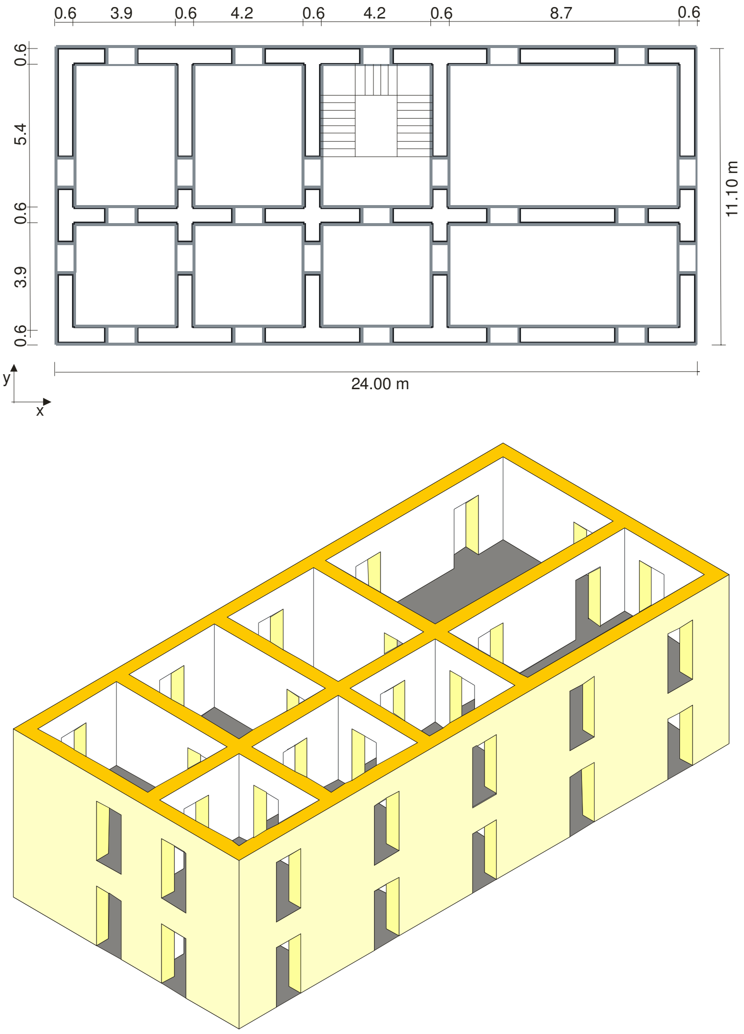

Figure 1. Plan layout and 3D view of the building analyzed

First floor and roof are considered to be subjected to dead loads equal to 5 kN/m^2 (due to self weight presuming timber floor structures and finishes) and to live loads equal to 2 kN/m^2. The masonry is assumed to have compression strength f_{mc} equal to 2 MPa, tensile strength f_{mt} = 0.1 MPa, Young's modulus E_m = 1500 MPa, tangent modulus G_m = 200 MPa. A volumetric mass equal to 17 kN/m^3 is assumed. Tributary areas at each floor are assumed to contribute to the loading of each wall. The design was performed using a 3D model and a simplified modal response spectrum analysis was applied.

3. BUILDING MODELING AND SEISMIC INPUT

The nonlinear static and dynamic analyses are carried out by the computer program Abaqus (Hibbit et al. 1997). The model adopted uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior the material. Figure 2 shows the mesh adopted in modeling the building. The floor slabs are assumed to have the actual stiffness deriving from their thickness.

Under uniaxial tension, the stress-strain response follows a linear elastic relationship until failure stress is reached, corresponding to the onset of micro-cracking in the material. Beyond the failure stress, the formation of micro-cracks is represented macroscopically with a softening stress-strain response. Under uniaxial compression

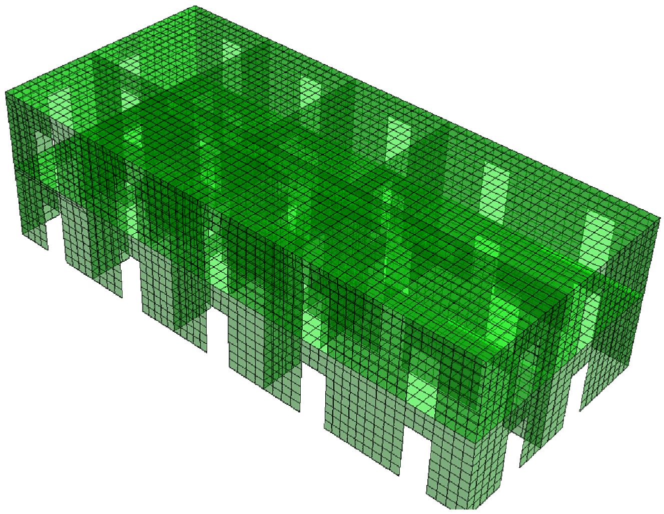

Figure 2. Finite element model of building analyzed

the response is linear until the value of initial yield. In the plastic regime, the response is modeled with stress hardening, followed by strain softening beyond the ultimate stress. When the element is unloaded from any point on the strain softening branch of the stress-strain curves, the unloading response is weakened: the elastic stiffness of the material appears to be damaged (or degraded). The degradation of the elastic stiffness is characterized by two damage variables, which are assumed to be functions of plastic strains, temperature, and field variables.

Under uniaxial cyclic loading conditions the degradation mechanisms are quite complex, involving the opening and closing of previously formed micro-cracks, as well as their interaction. It is assumed that there is a recovery of the elastic stiffness as the load changes sign during a uniaxial cyclic test. The stiffness recovery effect is more pronounced as the load changes from tension to compression, causing tensile cracks to close, which results in the recovery of the compressive stiffness.

The damaged plasticity model assumes that the reduction of the elastic modulus is given in terms of a scalar degradation variable. The expression for the scalar stiffness degradation variable is generalized to the multiaxial stress case by replacing the unit step function with a multiaxial stress weight factor.

The nonlinear dynamic analyses are performed using both generated acceleration records and registered earthquake records. The generated accelerograms constitute a suite of ground motions that comply with the requirements of the Italian seismic code. Specifically, the acceleration histories set is a spectrum-compatible ensemble (peak ground acceleration a_g = 0.25 g, type B subsoil having the constant branch of the spectrum between 0.15 and 0.50 seconds, soil factor S=1.25). Results presented in the next section concern the El Centro record also (Imperial Valley Earthquake, 05/18/40, S00E Component, PGA = 0.348 g), in order to investigate the effects of the variation in the seismic motion and then to analyze the nonlinear response of the building differently involved in the inelastic range of behavior.

In the dynamic analyses the damping matrix is taken to be proportional to the mass matrix and the initial stiffness matrix. The target damping is assumed equal to 5% in the first two modes. In the nonlinear static analyses, the loads are imposed onto the structure in a two-step sequence. Firstly, the vertical loads are applied and subsequently the lateral loads are monotonically increased. An invariant lateral load distributions is used to perform the pushover analysis, that is a distribution proportional to the actual distribution of masses within walls and floors. Each analysis is performed until any further increment in displacement is impossible due to numerical loss of convergence or material collapse. As previously stated, the results of analyses are shown only for y-direction, which proved to be the critical direction (Figure 1).

Table 1. Maximum top displacement of perimeter walls

Acceleration record	Symmetric Bldg. [mm]	Plan asymmetric Bldg.			
		Stiff side [mm]	Var. [%]	Flexible side [mm]	Var. [%]
El Centro	1.35	1.25	-7.3	1.61	+ 19.6
Generated A1	1.41	1.30	- 7.8	1.71	+ 21.1
Generated A2	1.48	1.36	- 8.2	1.81	+ 22.1
Generated A3	1.45	1.33	- 8.1	1.78	+ 21.9
Generated A4	1.51	1.38	- 8.8	1.86	+ 23.4
Generated A5	1.44	1.32	- 8.0	1.75	+ 21.2

4. RESULTS

4.1. Building vibration periods and modes

Preliminarily, the natural periods and the vibration modes are computed to recognize the dynamic properties of the building analyzed. Figure 3 shows the first and the third mode of the symmetric building variant and of the plan asymmetric building. The first mode of the symmetric building is predominantly in the y-direction, whereas in the unsymmetric building is characterized by floor translation and rotation. The second mode (not reported in Figure 3) is predominantly in the x-direction in both buildings. In the third mode the two floors rotate in the same direction, which is opposite to the first mode in the case of the unsymmetric building. Such results confirm that the dynamic response of plan asymmetric masonry structures may be assimilated to the one of unsymmetric framed structures. Therefore, some conclusions of the relevant researches can be extended to the masonry structure, even though the distinctiveness of masonry buildings and the differences in nonlinear behavior of masonry walls have to be properly considered.

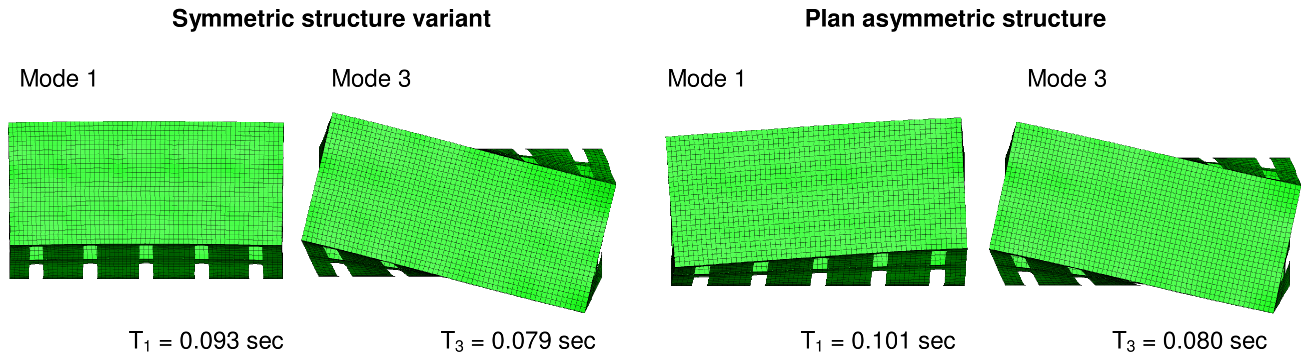

Figure 3. First and third vibration mode of symmetric and plan asymmetric buildings

4.2. Nonlinear dynamic analysis

The comparison of nonlinear dynamic response for symmetric and plan asymmetric building structure is pointed out in Table 1 and in Figure 4. Namely, Table 1 contains the maximum top displacement of the walls at the stiff and the flexible sides computed for all acceleration records, as well as the variation with respect to the symmetric building variant. The values confirm that torsion leads to a moderate decrease in displacement at the stiff side and to a large per cent increase at the flexible side, as it is customary in plan asymmetric framed structures. This means larger damage in perimeter walls as shown in Figure 4 that compares the tensile damage in the transverse walls of the symmetric and the unsymmetric building under El Centro earthquake record.

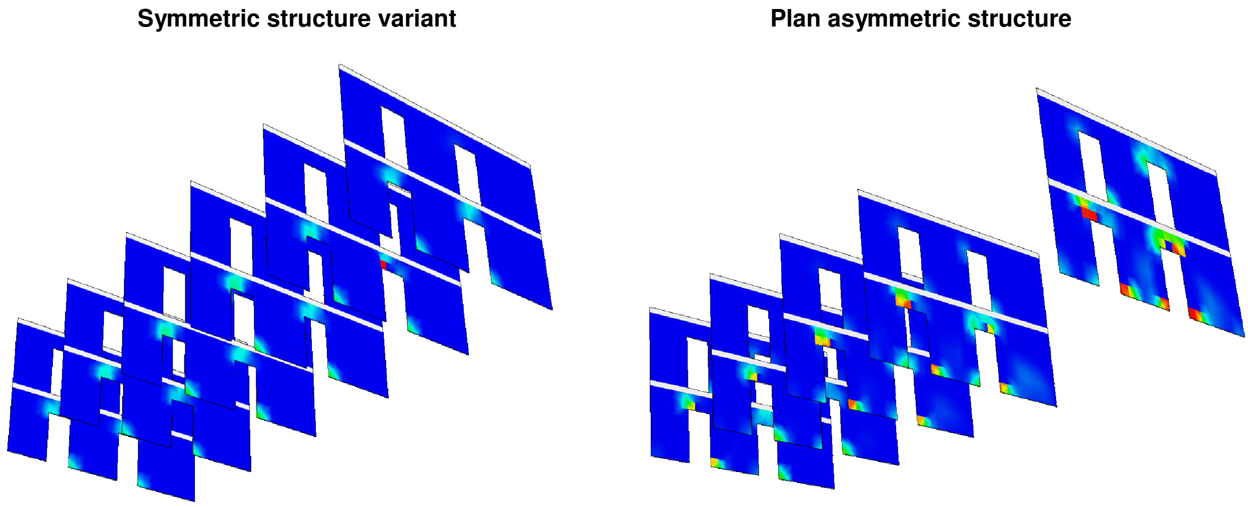

Figure 4. Tensile damage in transverse walls

Specifically, Figure 4 shows that damage is mainly localized in the spandrel beams and at the bottom of walls. Such a concentration of damage was somehow foreseeable and confirms that the strategies for the reduction of torsional response in masonry buildings are constrained, as reported in the introductive section.

4.3. Pushover analysis

Figure 5 presents the base shear - top displacement relationship for the symmetric and the plan asymmetric buildings, under the horizontal loads previously specified. The displacement at the mass center is plotted on x axis in both cases, as well as the one at the stiff and the flexible side in the case of the unsymmetric building. The overall base shear, nondimensionalized to the building weight, is plotted on y axis. It can be seen that the maximum lateral capacity in y direction of the symmetric building structure is slightly larger than the one of the plan asymmetric structure. The next section presents the comparison of pushover results with the ones from nonlinear dynamic analysis in order to assess whether changes need in lateral load distribution to match better the dynamic peak values of response.

4.4. Pushover analysis versus nonlinear dynamic analysis

If the pushover analysis is to be used to simulate the nonlinear dynamic response of building under an

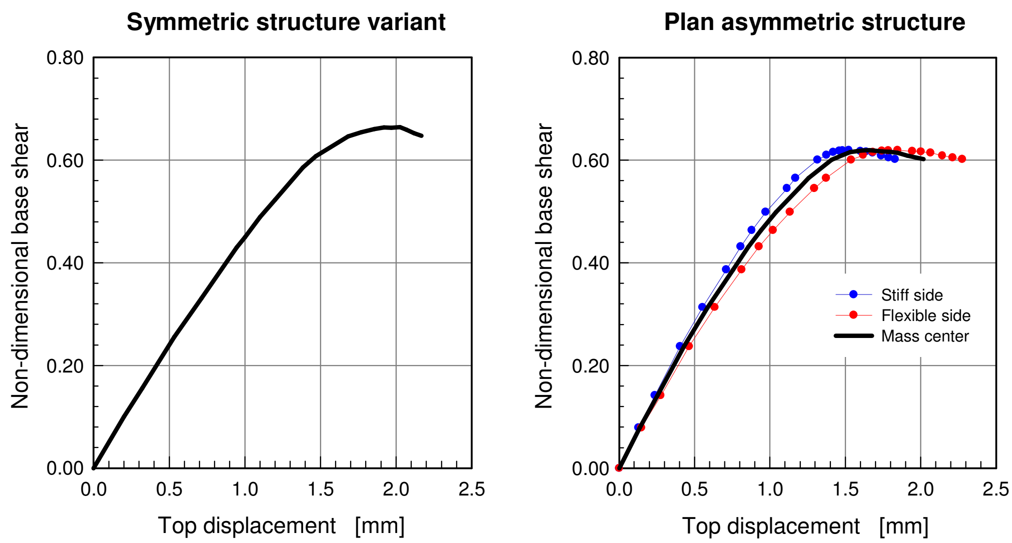

Figure 5. Pushover curves base shear – top displacement

earthquake, the target displacement up to which the analysis must be carried out should be known. In order to compare the results of pushover analysis and nonlinear dynamic analysis in this study the target displacement is determined by the nonlinear dynamic analysis - separately for each used earthquake record - at the mass center of the roof.

Figure 6 compares the displacements of the asymmetric building structure in y-direction computed by the pushover analysis with the ones computed by the nonlinear dynamic analysis under El Centro record and the generated record A4. Particularly, the envelope deflection profiles of the walls at the stiff side (SS), the mass center (CM) and the flexible side (FS) are plotted.

The pushover analysis underestimates the displacements at the flexible side with respect to the peak values of the nonlinear dynamic analysis, whereas the displacement at the stiff side are overestimated. Specifically, if the target displacement is matched at the center CM of the roof, the pushover analysis underestimates the top displacement at the flexible side of -4.35 % (for El Centro record) and of -6.45 % (for the generated A4 record) respect to the dynamic maximum displacement. At the stiff side the pushover analysis provides top displacements that are larger than the dynamic peaks (+5.60 % and +5.80 % for El Centro record and the generated A4 record respectively). These results show that in the building analyzed the error in displacement provided by pushover analysis is bounded. The dynamic values could be enveloped applying the lateral load eccentrically too, shifting the horizontal resultant force towards the flexible side. However, the procedure could be lightly different from the one usually adopted for framed structures, where masses are usually lumped in CM at floors and horizontal forces are eccentrically applied. In (Faella et al. 2008) a simple procedure that improves the pushover results for plan asymmetric masonry buildings is proposed, respecting the actual distribution of masses within walls and floors.

5. CONCLUSIONS

In this paper the torsional response of plan asymmetric masonry building structures and the applicability of the 3D pushover analysis for predicting the seismic response is investigated. The nonlinear dynamic response of a plan asymmetric masonry building is initially compared with the one of a symmetric building variant for several input ground motions, in order to evaluate the effects of the torsional response. Increases in wall displacement of about 20% has been measured at the flexible side that suffers larger damage. Subsequently, the results from pushover analyses are compared to those obtained by nonlinear dynamic analyses. The comparison is performed at the same maximum displacement of the mass center at the roof, computed separately for each earthquake record. The results, though limited to the selected test example, show that the envelope deflection profiles at the

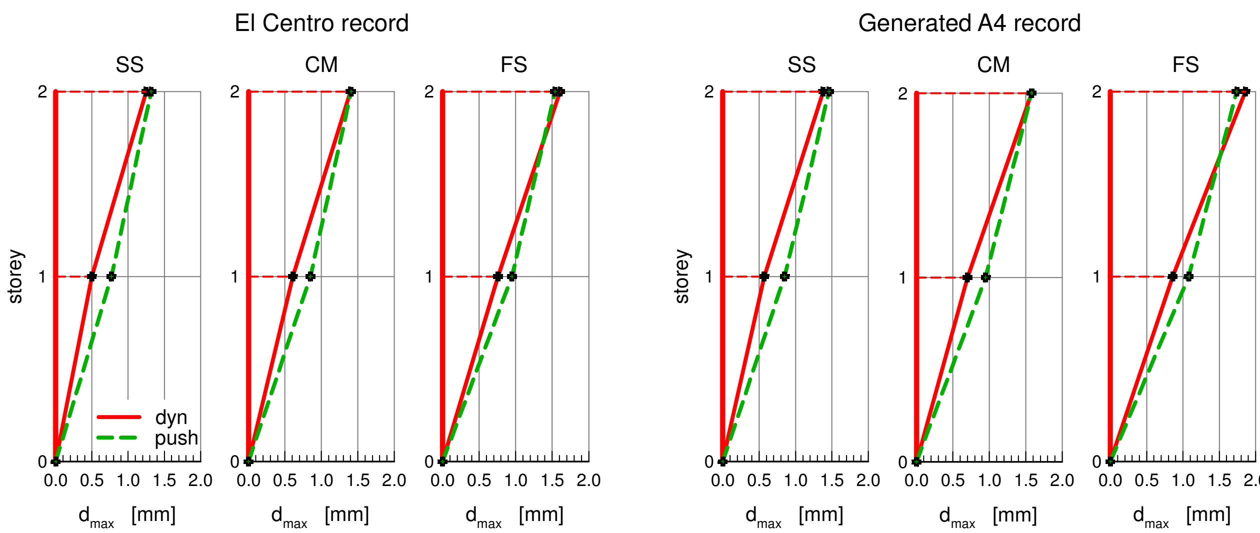

Figure 6. Envelope deflection profiles

stiff side is suitable matched whereas at the flexible side the pushover, performed amplifying the actual distribution of masses within walls and floors, does not cover the peak values of nonlinear dynamic response. This means that also in low-rise masonry buildings the pushover analysis should be performed shifting the horizontal resultant force towards the flexible side to match the peak values of nonlinear dynamic response. However, further researches are needed to develop procedures to predict the top displacement to which the 3D pushover analysis of plan asymmetric masonry building is to be targeted.

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