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UNSYMMETRIC-PLAN MASONRY BUILDINGS: PUSHOVER VS NONLINEAR DYNAMIC ANALYSIS

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ABSTRACT

This paper deals with the inelastic torsional behavior of unreinforced low-rise masonry buildings. First, the nonlinear dynamic response of unsymmetric-plan buildings under different seismic inputs is compared with the one of the counterpart symmetric variant in order to assess the torsional coupling arising from unsymmetry. The buildings are studied through the finite element method, adopting suitable constitutive models which implement well known concepts of damaged elasticity in combination with tensile and compressive plasticity. The accuracy of 3D pushover analysis, performed with two invariant distributions of lateral forces, is then assessed comparing the results in terms of horizontal displacement and damage at walls with the ones provided by the nonlinear time-history analysis. It is shown that pushover can adequately evaluate floor displacements but significantly underestimates damage at walls, even if combined with results of linear dynamic spectral analysis.

Introduction

The observation of earthquake damage indisputably showed that besides the quality of masonry material, building configuration is of remarkable importance. Masonry buildings with regular structural layout and walls well connected together at floor levels frequently showed satisfying performance, even when not designed to resist earthquakes. In fact, if the building structure is regular, gravity and seismic loads are evenly distributed among resisting elements, seismic energy can be dissipated almost uniformly over the entire structure.

Symmetric-plan distribution of resisting elements can prevent significant torsional vibration that frequently causes unexpected response when the building structure is subjected to strong input ground motion. Although the traditional masonry structural systems consist of load-bearing walls and cross walls that have simple plan distribution and constant thickness along the height, masonry buildings often show torsional response under earthquake. Many buildings are indeed not approximately symmetrical along each principal axis in plan, regarding both lateral stiffness and mass distribution, also because windows are of different dimensions and not aligned. The plan configuration recurrently results from composite shape (L, T, U, etc.), whereas long rectangular

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buildings can suffer torsional effects resulting from differences in ground motion. Additionally, irregularities may arise from the presence of different types of structural floor (vaults, timber structures, etc.) at the same building level.

The lack of symmetry may imply eccentricity between centers of mass and stiffness, inducing 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. Moreover, masonry walls may be contemporaneously subjected to in-plane and out-of-plane forces, and then suffer large damage or tend to separate at the building corners, causing local collapses or failure of floors.

The torsional response of building structures was extensively studied over the last years (Rutenberg 1992, De La Llera and Chopra 1995, Rutenberg 2002, Tena-Colunga and Pérez-Osornio 2005), but most researches are based on asymmetric single-storey models to simplify analyses and facilitate parametric studies (De Stefano et al. 1993, De la Llera and Chopra 1994). Furthermore, the effect of coupling between lateral and torsional motion is often studied in terms of element ductility demand, since seismic performance of framed structures mainly relies on ductile response. Most studies concern the optimization of the design of unsymmetric-plan buildings (Myslimaj and Tso 2002, Aziminejad et al. 2006). The improvement is frequently based on searching the best distribution in plan of lateral strength among resisting elements, and then on the reduction of the strength eccentricity. Alternatively, the improvement is achieved through differential increases in lateral strength of resisting elements in order to limit the ductility demand. Accordingly, many papers are aimed at assessing the reliability of seismic code procedure in ensuring adequate safety to multi-storey irregular framed structures (De la Llera and Chopra 1994, Humar and Kumar 2006).

The seismic behavior of irregular masonry buildings shows several differences with respect to r/c and steel framed structures, and some effects characterizing the framed structure response do not necessarily show up. Therefore, results from the above approaches may not be immediately extended to masonry structures, especially in case of retrofit of existing buildings. For the above reasons, thorough nonlinear analyses of multi-storey irregular masonry buildings are needed, since no previous study analyzed exhaustively their torsional response.

Current structural engineering practice uses simplified non-linear static procedure, originally limited to symmetric structures and then extended to unsymmetric-plan buildings (Faella and Kilar 1998, Moghadam and Tso 2000, Chopra and Goel 2004, Fajfar et al. 2005, Chopra et al. 2006, Fajfar et al. 2006). The building capacity is computed by pushover analysis of the structure subjected to monotonically increasing lateral forces until a target displacement is reached. Usually, force distribution and target displacement are determined on the base of the fundamental mode, assuming that the mode shape remains unchanged after the structure yielding. Seismic demand is frequently computed by inelastic spectra and depends on the period of the idealized equivalent SDOF system. Some researches proposed to combine pushover analysis of 3D structural models with modal response spectrum analysis, to control target displacement and distribution of deformation over the height of the building, as well as the torsional amplification (Fajfar et al. 2006). Obviously, these assumptions are approximate, but it was demonstrated that satisfactory prediction of seismic demands can be obtained for low- and medium-rise structures, provided that the inelastic action is distributed throughout the height of the structure (Chopra and Goel 2002). Quite clearly, similar approaches can be applied to unsymmetric-plan masonry buildings for estimating torsional effects, though studies in depth are needed for evaluating their suitability.

Galasco and Penna (2007) performed a comparison between nonlinear static and dynamic

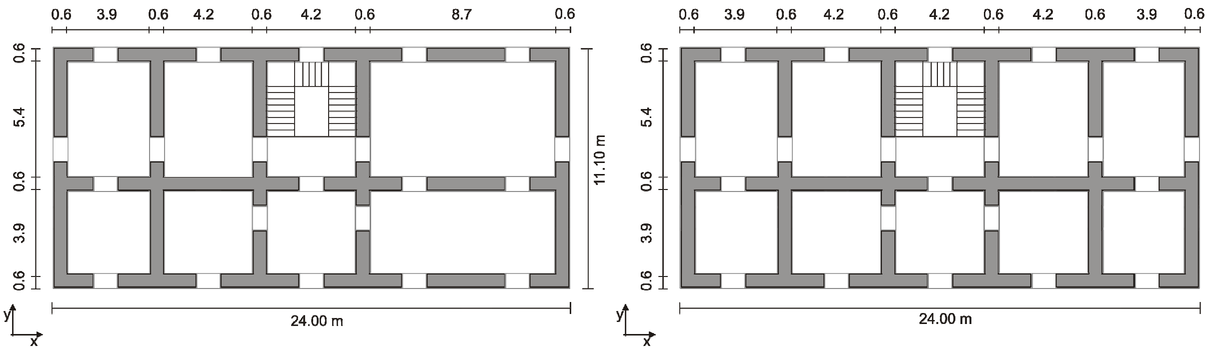

Figure 1. Plan layout of buildings

analysis for regular and irregular masonry buildings using a macro-element modeling approach, whose reliability in nonlinear dynamic analysis was demonstrated in (Penna et al. 2004). Specifically, it is shown that pushover analyses can provide suitable forecast of failure mechanisms and damage distribution among the resisting elements for irregular masonry buildings too, even though the scatter with respect to nonlinear dynamic results can be remarkable.

This paper is aimed at investigating the nonlinear response of unsymmetric-plan low-rise unreinforced masonry structures. First, nonlinear dynamic analyses are performed using a refined finite element approach in order to assess the magnitude of torsional response. Second, pushover analyses controlled by results of modal response spectrum analysis are carried out in order to study their suitability as tool for representing the torsional response of masonry buildings.

Buildings description

Figure 1 shows the structure plan layout. The buildings are rectangular with a 24.00 x 11.10 m plan envelope; the stories are 3.50 m high. All the openings are 1.20 m wide and 2.50 m high. The wall thickness is kept unchanged at all stories and equal to 0.60 m. The building plan is not symmetric about both x and y axes. The asymmetry in x-direction arises from the position of the longitudinal inner wall that is not barycentric. The asymmetry in y-direction is due to the lack of the second-last wall. The building variant, which includes this wall, is assumed as reference symmetric structure in identifying the response effects due to torsional coupling, when subjected to ground motion in y-direction.

Floors and roof are subjected to dead loads equal to 5 kN/m^2 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 load the walls. The design was performed through a simplified modal response spectrum analysis of a 3D model.

Buildings modeling

All analyses here described have been performed using the computer program ABAQUS® (Hibbit et al. 1997). The selected model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of masonry material.

Under uniaxial tension, the stress-strain response follows a linear elastic relationship until the failure stress is reached, corresponding to 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, the response is linear until the initial yield. In the plastic regime, the response is modeled with stress hardening, followed by strain softening beyond the maximum stress. When the element is unloaded from any point on the softening branch of the stress-strain curve, 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 that are assumed to be function of plastic strains 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 if the load changes sign during the uniaxial cyclic test. The stiffness recovery effect is more pronounced if 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.

Each floor diaphragm is assumed to have the stiffness in its own plane deriving from its thickness. In the dynamic analysis, the damping matrix is supposed to be proportional to the mass matrix and the initial stiffness matrix. The target damping is assumed equal to 5% for the first two modes.

Input ground motion

Registered earthquake records and generated acceleration signals were used in performing nonlinear dynamic analyses. Specifically, El Centro record (Imperial Valley Earthquake, 05/18/40, S00E Component, PGA = 0.348 g) was used as registered ground motion. The generated accelerograms constitute a spectrum-compatible ensemble of ground motions (peak ground acceleration $a_g = 0.25$ g, constant branch of the response spectrum between 0.15 and 0.50 seconds, soil factor S = 1.25) complying with the requirements of Italian seismic code. The input ground motion is taken to act separately along the two main orthogonal horizontal axes of the structure and without combinations of effects, in order to obtain results that are not influenced by the simultaneous action of both the earthquake components.

Nonlinear response history analysis

The amount of torsional coupling is assessed comparing the nonlinear dynamic response of unsymmetric-plan buildings with the counterpart symmetric-plan variant. Each analysis is performed until any further increment in displacement is impossible due to material collapse or numerical loss of convergence. Results for y-direction only are presented, since it was proved to be the critical direction.

Table 1 contains the maximum roof displacement at the stiff and flexible sides due to the registered earthquake record, as well as the mean value of the maximum roof displacement due to the ensemble of generated acceleration records. Table 1 also reports the variation with respect

Table 1. Maximum roof displacement at perimeter walls

	Acceleration record	Symmetric Bldg. [mm]	Unsymmetric-plan Bldg. Stiff side [mm]	Unsymmetric-plan Bldg. Var. [%]	Unsymmetric-plan Bldg. Flexible side [mm]	Unsymmetric-plan Bldg. Var. [%]
2-story Bldg.	El Centro	1.35	1.25	-7.4	1.61	+19.3
	Generated	1.46	1.34	-8.2	1.78	+21.9
3-story Bldg.	El Centro	2.38	2.37	-0.4	3.03	+27.3
	Generated	2.64	2.42	-8.3	3.25	+23.1

to the symmetric building variant. Values of Table 1 show that the torsional response leads to moderate decreases in floor displacement at the stiff side and to large increases at the flexible side. Therefore, Table 1 confirms that favorable torsional effects on the stiff side (i.e. reduction in displacements compared to the counterpart symmetric building), frequently arising from elastic analysis, disappear in the inelastic range, as it is frequent in unsymmetric-plan framed structures.

In Figure 2 the tensile damage at the walls in y-direction of the symmetric three-story building and damage at two walls of the unsymmetric-plan building under El Centro earthquake record are compared. Figure 2 confirms that the unsymmetric building undergoes larger damage than the symmetric one, and damage is distributed quite uniformly among the walls. Namely, the wall at the flexible side does not suffer significantly larger damage than the inner ones, despite the displacements are greater. Figure 2 also shows that damage is mainly localized in the spandrel beams and at the bottom of the walls. Similar results were obtained for two-story buildings (Giordano et al. 2008).

To explain this outcome, the response history of roof displacement at the flexible side due to El Centro ground motion is presented in Figure 3. Figure 3 shows that the wall undergoes the largest displacements during the first 10 seconds of the earthquake time-history. This implies that first damage involves the wall at the flexible side, reducing its bearing capacity.

Consequently, at first the seismic action largely rests on the wall at the flexible side, whereas in subsequent times during the response history it is carried more by the inner walls.

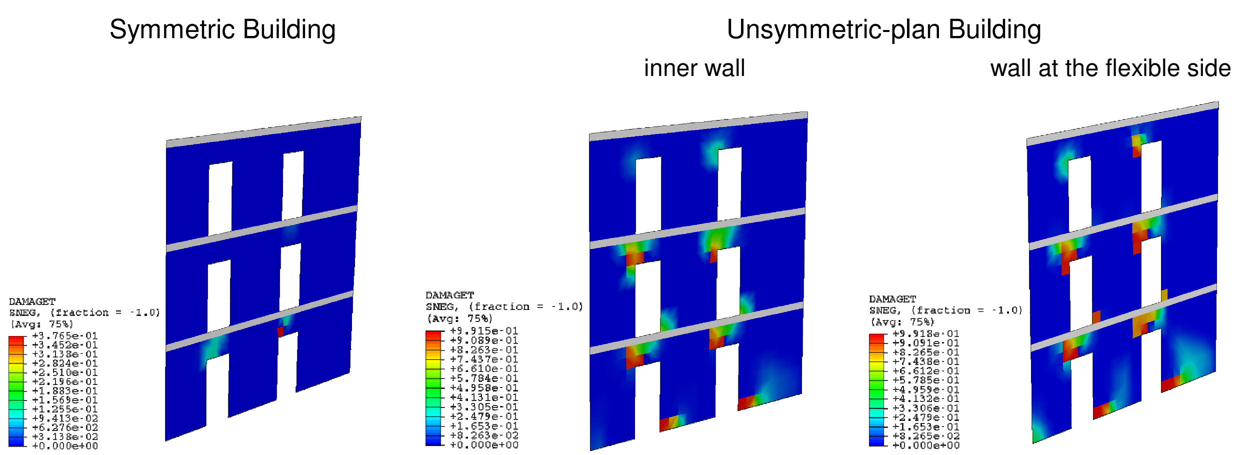

Figure 2. Tensile damage at walls due to El Centro ground motion

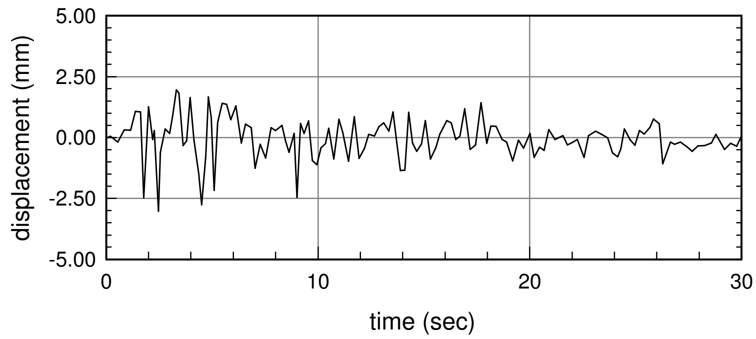

Figure 3. Response history of roof displacement due to El Centro ground motion

This is confirmed by Figure 4 that contains the distribution of the base shear among the walls at some selected steps of the response history. Figure 4 shows that at the beginning the wall at the flexible side sustains 28% of the seismic load, while the inner walls carry on less than 20% and the wall at the stiff side about 15% of the seismic load. Afterwards, the distribution of seismic loads tends to be uniform and at the end of the response history all the walls sustain about 20% of the seismic load. Therefore, the wall at the flexible side suffers damage at the beginning whereas the inner walls are damaged later, thus resulting all the walls similarly damaged at the end of the earthquake history, as it is shown in Figure 2.

This distinctiveness in dynamic torsional response of low-rise masonry buildings makes more complex the use of pushover analysis, since it would reproduce comparable damage in all the walls although the inner ones undergo smaller displacements.

Pushover analysis

The pushover analysis is performed applying loads to the structure in a two-step sequence. Firstly, the vertical loads are applied and then the horizontal forces are monotonically increased. Two invariant lateral load distributions are selected, that is a force distribution proportional to the actual distribution of masses within walls and floors (DM) and a force

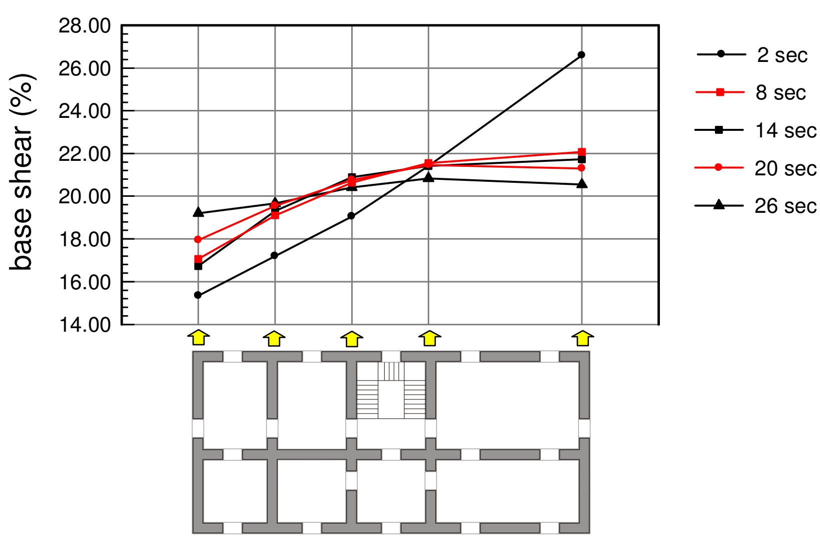

Figure 4. Time distribution of base shear among the building walls [El Centro record]

distribution proportional to the components of the first mode of vibration, weighted by the storey mass (VM). The second distribution derives from (Tso and Moghadam 1997, Fajfar et al. 2006, Chopra and Goel 2002) and has a clear physical background. If the structural behavior was elastic, this distribution would correspond to the effective distribution of the earthquake forces; in inelastic range, instead, the displacement shape changes with time and the assumption represents an approximation. The lateral loads are increased until the roof displacement at the center of mass equals the maximum displacement obtained by the nonlinear dynamic analysis, computed for each earthquake record separately.

Table 2 contains the first three natural periods of the buildings as well as the shape of the modes of vibration. Obviously, the first two modes of the symmetric buildings are translational (in y-direction and x-direction respectively), whereas the unsymmetric buildings show noteworthy translation and rotation of floors in the first mode. As expected, the third mode is always characterized by rotation of floors. Such results confirm that the unsymmetric-plan masonry structures may be characterized by torsional response under earthquakes, similarly to unsymmetric framed structures. Therefore, some remarks 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 5 shows the base shear - top displacement relationship for the symmetric and the plan asymmetric three-story building, under the two horizontal load distributions previously specified (DM and VM). The displacement at the center of mass, at the stiff and the flexible side for the unsymmetric building are plotted on x-axis. The 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 (Figure 5a) is slightly larger than the one of the plan asymmetric structure (Figures 5b and 5c). Furthermore, the comparison between Figure 5b and Figure 5c shows that the force distribution based on the components of the first mode of vibration (VM) leads to larger floor displacements at the flexible side than the force distribution DM, whereas smaller displacements are computed at the stiff side. This implies that pushover procedures considering force distribution resulting from linear dynamic spectral analysis could be less conservative at the stiff side.

Pushover analysis vs. Nonlinear dynamic analysis

The accuracy of pushover analysis in comparison to nonlinear dynamic analysis of unsymmetric-plan masonry buildings is firstly performed in terms of floor displacements. The comparison is carried out for the maximum roof displacement at the center of mass provided by the nonlinear response history analysis, separately for each earthquake record. Therefore, the target displacement up to which the building must be pushed is assumed to be known.

Table 2. Natural periods and modes of vibration of buildings

Building	Mode 1		Mode 2		Mode 3	
	T [sec]	shape	T [sec]	shape	T [sec]	shape
2-story symmetric bldg. variant	0.093	y-translational	0.088	x-translational	0.079	rotational
2-story unsymmetric-plan bldg.	0.101	rotational	0.088	x-translational	0.080	rotational
3-story symmetric bldg. variant	0.136	y-translational	0.125	x-translational	0.113	rotational
3-story unsymmetric-plan bldg.	0.145	rotational	0.122	x-translational	0.115	rotational

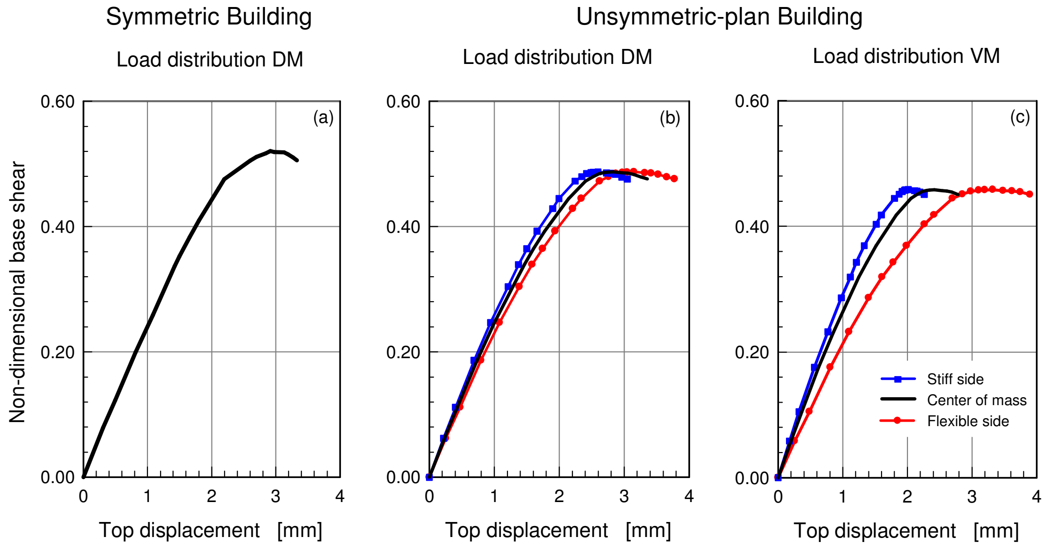

Figure 5. Pushover curves for symmetric and unsymmetric-plan buildings

The peak values of floor displacement at the stiff side (SS), the center of mass (CM) and the flexible side (FS) for the unsymmetric-plan three-story building in y-direction are compared with the values due to El Centro record in Figure 6. Figure 6 shows that pushover with force distribution DM underestimates the displacements, especially at the flexible side. Specifically, if the target displacement is matched at the roof center of mass, the pushover underestimates the roof displacement at FS of 14.19% and yields a displacement 4.17% smaller than the dynamic peak at SS. Contrary, the results obtained for the two-story building showed that the error was bounded (Giordano et al. 2008). The force distribution VM allows enveloping the peak values of floor displacements at the flexible side, but underestimates more the displacements at the stiff side. These results show that results of pushover analysis need to be corrected for providing accurate floor displacements also at the stiff side, and the improvement could be achieved according to extension advised for irregular framed structures (Fajfar et al. 2006).

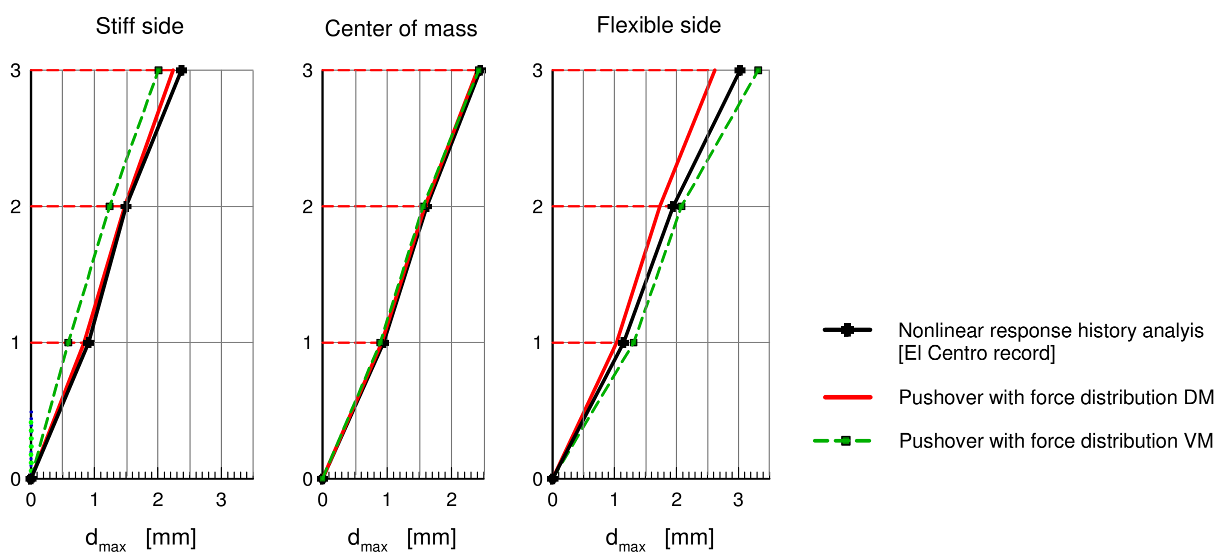

Figure 6. Peak values of floor displacement for the unsymmetric-plan building

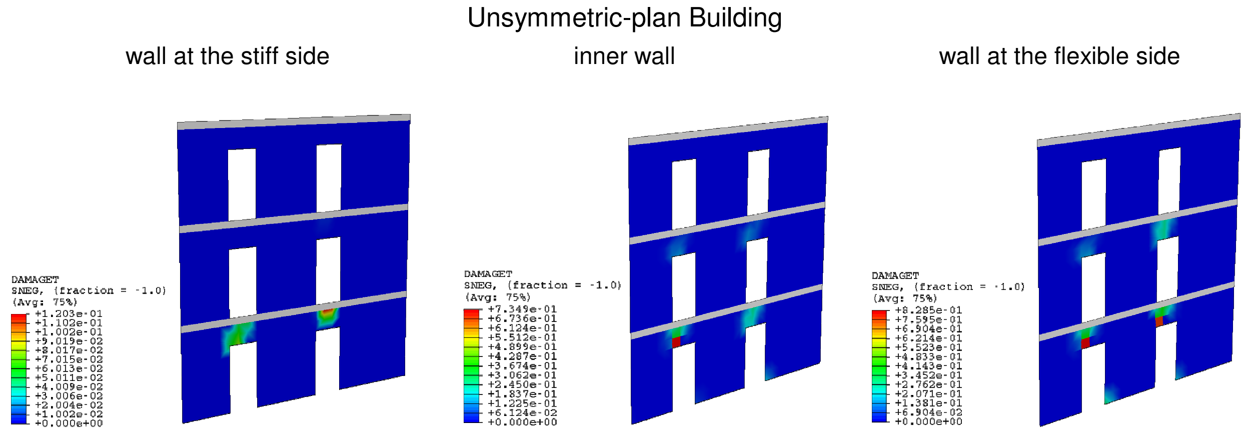

Figure 7. Tensile damage at walls provided by pushover analysis

Moreover, pushover analysis yields inaccurate assessment of damage at walls. Figure 7 shows tensile damage at walls provided by pushover with lateral force distribution VM when the target displacement is achieved at the roof center of mass. Comparing Figure 7 with Figure 2 it is evident that damage is notably underestimated in all the walls, despite the displacements at the flexible side are enveloped. This highlights an intrinsic incapability of pushover analysis in assessing the feasible damage of unsymmetric-plan masonry buildings if performed with invariant distribution of lateral forces.

Conclusions

This paper was aimed toward analysing the torsional response of unsymmetric-plan low-rise masonry buildings and evaluating the accuracy of pushover analysis in comparison to nonlinear dynamic analysis. The investigation has led to the following conclusions.

Unsymmetric-plan buildings show increases in floor displacements respect to the counterpart symmetric variant that are customary in torsionally stiff frame buildings (at the flexible side up to 20% for two-story buildings and up to 30% for three-story buildings). The increase in damage is also significant and involves quite uniformly all the masonry walls, despite the floor displacements are notably different because of the floor rotation. The response histories of floor displacement and base shear show that the wall at the flexible side undergoes larger displacements and major damage during the first seconds of the earthquake loading. To progress of the response history, the seismic action loads and damages the inner walls, thus resulting all the walls similarly damaged at the end of the earthquake history. This distinctiveness in nonlinear dynamic torsional response of low-rise masonry buildings makes more complex the use of pushover analyses, since they would provide comparable damage in walls that undergo different displacements.

Pushover analysis with invariant distribution of lateral forces proportional to the components of the first mode of vibration, weighted by the storey mass, accurately evaluates the floor displacement at the flexible side, but underestimates the torsional effects at the stiff side. Moreover, pushover analysis yields inaccurate assessment of damage at walls, probably highlighting an intrinsic incapability of pushover analysis in estimating effectively the feasible damage of unsymmetric-plan masonry building if performed with invariant distribution of lateral forces. Therefore, further research steps are needed to improve directions of pushover procedure for predicting target displacement, possible displacement amplification at each wall and seismic demand, which are required for performing the performance evaluation according to most seismic codes.

References

Aziminejad, A., A.S. Moghadam and W.K. Tso, 2006. Behavior of asymmetric multistory buildings with different strength distribution. *1st European Conf. on Earth. Engng. and Seismology,* Geneva

Chopra, A.K. and R.K. Goel, 2002. A modal pushover analysis procedure for estimating seismic demands for buildings, *Earthquake Engng. & Struct. Dyn.*, 31, 561-582.

Chopra, A. K. and R.K. Goel, 2004. A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings, *Earthquake Engng. & Struct. Dyn.*, 33, 903-927.

Chopra, A.K., K. Rakesh and R.K. Goel, 2006. Evaluation of the modal pushover analysis procedure for unsymmetric-plan buildings. *1st European Conf. on Earthquake Engng. and Seismology*, Geneva.

De la Llera, J.C. and A.K. Chopra, 1994. Accidental and natural torsion in earthquake response and design of buildings. *Rep. EERC94-07, Earth. Engng. Research Center*, University of California.

De la Llera, J.C. and A. Chopra, 1994. A simplified model for analysis and design of asymmetric-plan buildings. *Earthquake Engng. & Struct. Dyn.*, 24, 573-594.

De La Llera, J., and A. Chopra, 1995. Understanding the inelastic seismic behaviour of asymmetric plan buildings. *Earthquake Engng. & Struct. Dyn*., 24(4), 549-572.

De Stefano, M., G. Faella and R. Ramasco, 1993. Inelastic response and design criteria of plan-wise asymmetric systems. *Earthquake Engng. & Struct. Dyn.*, 22(3), 245-259.

Faella, G. and V. Kilar, 1998. Asymmetric multistory r/c frame structures: push-over versus nonlinear dynamic analysis, *11th European Conf. on Earthquake Engng*., Paris.

Fajfar, P., D. Marušić and I. Peruš, 2005. Torsional effects in the pushover-based seismic analysis of buildings. *Journal of Earthquake Engng*., 9(6), 831-854.

Fajfar, P., D. Marušić and I. Peruš, 2006. The N2 method for asymmetric buildings. *First European Conf. on Earthquake Engng. and Seismology*, Geneva.

Galasco, A. and A. Penna, 2007. Valutazione della risposta sismica di edifici irregolari in muratura. *ANIDIS 2007, 12th Italian National Conf. on Earthquake Engng.*, Pisa (in Italian).

Giordano, A., M. Guadagnuolo and G. Faella, 2008. Pushover analysis of plan irregular masonry buildings, *14th World Conf. on Earthquake Engng.*, Beijing.

Hibbit, Karlsson and Sorensen Inc., 1997. *Abaqus User's Manual*, v.6.4.1, Pawtucket, Rhode Isl., USA.

Humar, J.M. and P. Kumar, 2006. Review of code provisions to account for earthquake induced torsion. *First European Conf. on Earthquake Engng. and Seismology*, Geneve.

Moghadam, A.S. and W.K. Tso, 2000. Pushover analysis for asymmetric and set-back multi-story buildings. *12th World Conf. on Earthquake Engng*., Auckland.

Myslimaj, B. and W.K. Tso, 2002. A strength distribution criterion for minimizing torsional response of asymmetric wall-type systems, *Earthquake Engng. & Struct. Dyn.*, 31, 99-120.

Penna A., A. Galasco A., S. Lagomarsino and S. Resemini, 2004. Non-linear seismic analysis of masonry structures, *13th World Conf. on Earthquake Engng.*, Vancouver.

Rutenberg, A., 1992. Nonlinear response of asymmetric building structures and seismic codes: A state of the art. *European Earthquake Engng.*, VI (2), 3-19.

Rutenberg, A., 2002, EAEE Task Group (TG8): Behaviour of irregular and complex structures - asymmetric structures - progress since 1998. *12th European Conf. on Earth. Engng.,* London.

Tena-Colunga, A. and M.A. Pérez-Osornio (2005). Assessment of shear deformations on the seismic response of asymmetric shear wall buildings. *J. of Structural Engng.*, Vol. 131(11), 1774-1779.

Tso, W.K. and A.S. Moghadam, 1997. Seismic response of asymmetrical buildings using pushover analysis, *Int. Workshop on Seismic Design Methodologies for the Next Generation of Codes,* Fajfar and Krawinkler eds., Bled, Slovenia, A.A. Balkema, Rotterdam, 311-321.