AN OVERVIEW OF PUSHOVER PROCEDURES FOR THE ANALYSIS OF BUILDINGS SUSCEPTIBLE TO TORSIONAL BEHAVIOR

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ABSTRACT :

This paper compares results from pushover type static analyses of a 5-story building having one axis of symmetry with results obtained by nonlinear dynamic analyses, using semi-artificial earthquake motions generated to match the spectrum with which the building was designed. The analyses aim at evaluating the seismic capacity of the building. Results are also presented for 50% increased earthquake intensity. By considering only one-component motion along the axis of no symmetry, three non-linear static procedures are examined: the so called Modal Pushover Analysis, the N2 method as it was extended for asymmetric buildings and the FEMA recommended procedure for two variations of horizontal load pattern (modal and uniform). It was observed that all three methods, especially the Modal Pushover method, may lead to results in good agreement with those obtained by dynamic analyses for design level earthquakes. However, for increased earthquake intensities, when the behavior of the building is strongly affected by the yielding of structural components, the results differed significantly. In this case nonlinear dynamic analysis appears to be the only appropriate method for the evaluation of the seismic capacity of the building.

KEYWORDS: asymmetric buildings, torsional behavior, pushover analysis, nonlinear dynamic analysis

1. INTRODUCTION

Catastrophic seismic events of the past few decades worldwide have accelerated research in the field of earthquake structural engineering and seismic hazard assessment. As a consequence, new codes for earthquake resistant design along with updated seismic hazard maps have been developed. In addition to this, a significant amount of research has been targeted towards capacity assessment of existing structures, as well as towards new materials, techniques and redesign procedures that may be used for repair and/or strengthening of earthquake damaged or undamaged structures. The results of the aforementioned research effort have been recently codified and published as a series of model codes, pre-standards or recommendations that may be used by engineers for guidance (FEMA 356, FEMA, 2000; ATC-40, ATC, 1996; Part 3 of Eurocode 8, CEN, 2004; Greek Retrofitting Code Drafts, OASP, 2006).

Assessing the seismic capacity and retrofitting an existing structure is a more difficult task than designing a new structure for earthquake loads. The engineer has to cope with a number of uncertainties regarding the structural system and its properties, which even in the case that design drawings are available, are generally not certain since such drawings may often not reflect the as-built conditions. Thus, the engineer has to resort to extensive survey work in order to properly identify the structural system, as well as to carry out on site and laboratory measurements and tests in order to identify soil and material properties, possible deterioration with time, e.g. reinforcing steel corrosion, etc. Since performing an extensive test program can be quite expensive, the aforementioned issues are often addressed in a parametric manner.

In addition to the aforementioned uncertainties, the assessment of seismic capacity of an existing structure involves uncertainties regarding the analytical methods to be used. Conventional procedures used for the design of new buildings are quite adequate, given the factors of safety built into the material properties and into the overall design. On the other hand, approximations such as those involved with the use of a global behavior factor in order to indirectly account for structural ductility and overstrength are not applicable for capacity assessment of existing structures, whose properties are there and leave no room for error compensation through overdesign. In order to address this issue, the aforementioned model codes and standards adopted and implemented a member deformation control approach. According to this approach, the expected member deformations for a design level earthquake, computed from the analysis of a 3-dimensional model of the structure, are compared with previously determined limiting values associated with specific performance levels. In order to have reliable estimates of member deformations, the aforementioned standards specify nonlinear analysis methods, static or dynamic. One of the most frequently used methods at present for capacity assessment is the so called pushover analysis.

Pushover analyses have been used in the past for assessing design weaknesses in fixed offshore structures (Kallaby and Millman, 1975; Gates et al., 1977), whose final design was carried out by conventional static or dynamic methods. In fact, the term "pushover" analysis was coined at that time by Peter Marshall of Shell Oil. The method has gradually found wider application in the field of seismic assessment of existing structures and was implemented as a basic procedure in the FEMA 356 pre-standard (FEMA, 2000). Several variations of varying complexity related to either the assumed vertical load distribution or to the way the target displacement is estimated, have been proposed as more reliable alternatives to the rather simple FEMA procedures (e.g. Gupta and Kunnath, 2000, Elnashai 2001, Aydinoglu, 2003). A detailed assessment of such procedures may be found in a more recent FEMA document (FEMA 440, FEMA, 2005).

All these variations give reasonable results when applied to symmetric buildings, i.e. when no torsion is present. To account for torsion, pushover methods used in 2-D have been extended to irregular buildings in 3-D (e.g. Chopra and Goel, 2002, 2004, Fajfar, 2000, Fajfar et al., 2005, De Stefano and Ruttenberg, 1998, Moghadam and Tso, 1998, Penelis, 2007). These extensions, however, are still questionable since the quality of the results depends on factors such as the degree of modal coupling due to torsion and the extent of member yielding.

This paper presents some preliminary results of an ongoing investigation aimed at the problem of capacity assessment and design of non symmetric buildings using non linear procedures. It addresses the issue of how inelastic response predictions by pushover analyses compare with predictions from inelastic dynamic analysis, the most reliable method for a given set of earthquake motions. Our investigation is based on the comparison of results regarding the seismic performance of a 5-story concrete building that has one axis of symmetry (y axis) but is eccentric with respect to the x-axis. Three different pushover algorithms are considered: the FEMA prescribed procedure using two variations for the distribution of the horizontal load, the modal and uniform, the Modal Pushover Analysis or MPA procedure (Chopra and Goel, 2002, 2004) and the method known as N2 (Fajfar, 2000, Fajfar et al., 2005). The MPA method uses more than one load distributions, each derived from a different mode, obtained by multiplying the modal acceleration of each joint with the corresponding mass, to determine more than one pushover curves. In our application herein only two modes, modes 2 and 3, are used since the contribution of the others was negligible. The N2 method relies on conventional pushover analysis of a 3-D model of the building using a modal horizontal load pattern similar to the one proposed in FEMA 356, with a target displacement computed from inelastic demand spectra (Fajfar, 2000). Torsional effects are considered by amplifying pushover analysis results by an amplification factor, determined from elastic modal analysis of the 3-D building as the ratio of an element's horizontal displacement to the corresponding displacement at the mass center of the level considered. No de-amplification due to torsion is considered.

For the nonlinear dynamic analyses, five semi-artificial motions have been used, derived from real records but modified to match the design spectrum of the building. The pushover procedures examined herein are limited to those using invariable horizontal load profiles, since it is our belief that the so called "adaptive pushover" methods using a varying load profile are overly sophisticated. We think that in view of the approximations introduced, especially when pushover analyses are extended to 3-D irregular buildings, over sophisticated pushover methods are not justified. In such cases the more rigorous method of multiple inelastic dynamic analyses might be preferable.

2. BUILDING AND EARTHQUAKE MOTIONS USED

The building used for the comparisons presented in this article is a 5-strory concrete building. Its structural system consists of 4 moment resisting frames in each direction with bay lengths equal to 5.00 m along the x-axis and 4.00 m along the y-axis (Fig. 1). It is almost symmetric, except for two shear walls at the back side which create a uniaxial stiffness eccentricity. The layout of a typical floor along with a 3-D view of the building is presented in Figure 1, where it can be seen that the building is symmetric about the y axis and eccentric in the x direction. The center of mass coincides with the geometric center, while the center of stiffness is shifted towards the "stiff side" due to the shear walls, creating an eccentricity e_y=12%. Thus the building will experience torsional motion under earthquake action in the x direction. All story heights are 3.00 m, except for the ground story that is 4.00 m high.

The building was designed for gravity and earthquake loads, for a peak ground acceleration of 0.24g and the design spectrum of the Greek code (same as that of the previous edition of Eurocode 8) for soil category II. This spectrum is shown in Figure 2 along with the average response spectrum of the five semi-artificial motions, created for checking the pushover analysis results. These motions were generated by modifying five historical records, namely El Centro, 1940 (EW), Thessaloniki, 1978 (T), Loma Prieta, 1989 (T), Olympia 1949 (T) and Corinthos 1981 (L), using a special program (Karabalis et al, 1994), in order to have a reasonably good match with the design spectrum. This ensures that the results of nonlinear analyses will not be significantly affected by the characteristics of individual ground motions, thus allowing a fair comparison of the considered pushover procedures.

Elastic eigenvalue analyses, carried out with the SAP2000 (CSi, 2007) program, provided the dynamic characteristics of the building. Table 1 summarizes the first 6 periods and corresponding effective modal mass ratios. The modes are presented in Figure 3. The fundamental bending mode is a pure mode in the y direction, which together with the 2nd y-bending mode (mode 4) have an effective modal mass equal to 97% of the total mass. The 2nd and 3rd modes as well as the 5th and 6th modes are modes coupling the x and torsional motion, and thus significant torsional response can be expected under earthquake motion in the x direction.

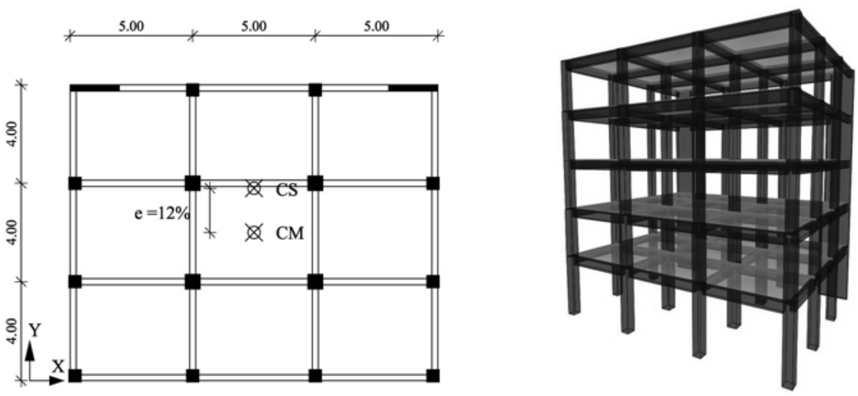

Figure 1 Layout of the building

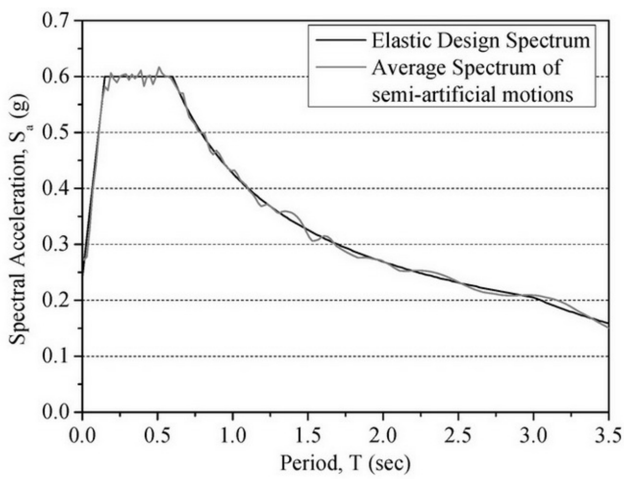

Figure 2 Design spectrum and average response spectrum of the five semi-artificial motions

Table 1 Building Periods and effective modal masses

Mode	Period (sec)	M^*_x (%)	M^*_y (%)	Sum M^*_x (%)	Sum M^*_y (%)
1	0.558	~0.00	87.35	~0.00	87.35
2	0.546	55.34	~0.00	55.34	87.35
3	0.399	26.78	~0.00	82.12	87.35
4	0.190	~0.00	9.53	82.12	96.88
5	0.184	6.40	~0.00	88.52	96.88
6	0.123	5.73	~0.00	94.25	96.88

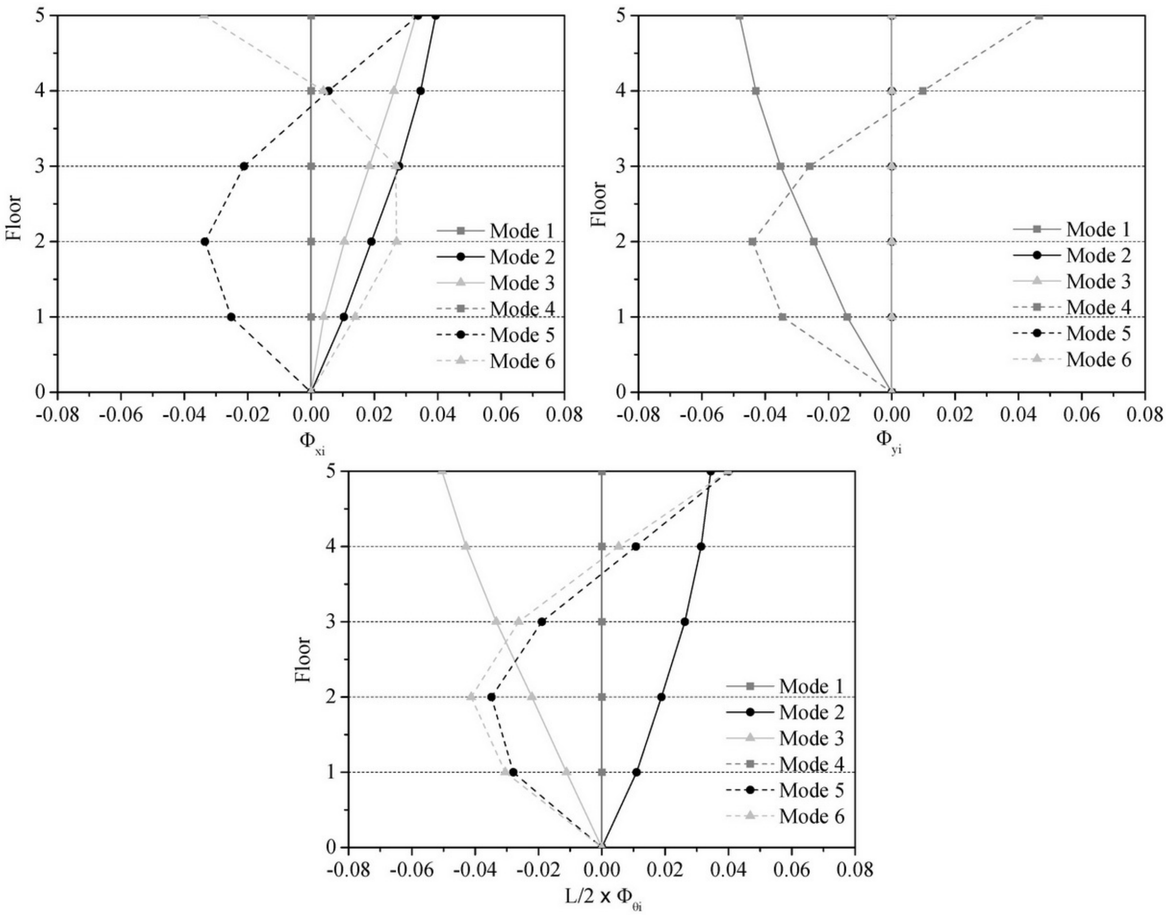

Figure 3 Building modes

3. RESULTS AND DISCUSSION

SAP2000 was also used to carry out the inelastic static pushover and dynamic time history (RHA) analyses. Table 2 (page 7) lists the target, top story, displacements in the x-axis by the various procedures for the mass center CM, the stiff side (back) and the flexible side (front) of the building, for the two levels of motion intensity. Corresponding peak displacement profiles are compared in Figure 4, whereas Figure 5 presents the comparisons of interstory drifts. The values given for the RHA are mean and maximum values for the five records. The displayed results follow the trends of the peak-target displacements. The FEMA procedure led to conservative results in all cases with the exception of the displacement profiles and interstory drifts at the flexible side of the building, where the use of modal distribution led to results which appear to be in good

agreement with those obtained by nonlinear RHA. At the CM and the stiff side of the building the N2 method gives slightly better results than FEMA, while at the flexible side it gives overly conservative results due to large values of the amplification factor. Finally, the MPA seems to be the more accurate of the methods examined herein for design level earthquakes. For higher intensity, however, when the behavior of the structure is strongly nonlinear, the MPA procedure underestimates top-story displacement demands, leading in some cases to significantly lower displacements than those by RHA. In this case, the envelope of the MPA values (displacements and drifts) appears to be in better agreement with the nonlinear RHA results.
Finally, the maximum plastic hinge rotations in beams for the various procedures and the two motion intensities are presented in Figure 6. They do not follow the same trend with interstory drifts as they are affected by the yielding of the shear walls at the stiff side and of the columns at the flexible side (the latter being observed for the highest earthquake intensity examined). It should be noted that for the stiff side the MPA seems to lead to plastic hinge rotations for beams consistently lower than those obtained by RHA, while giving a better

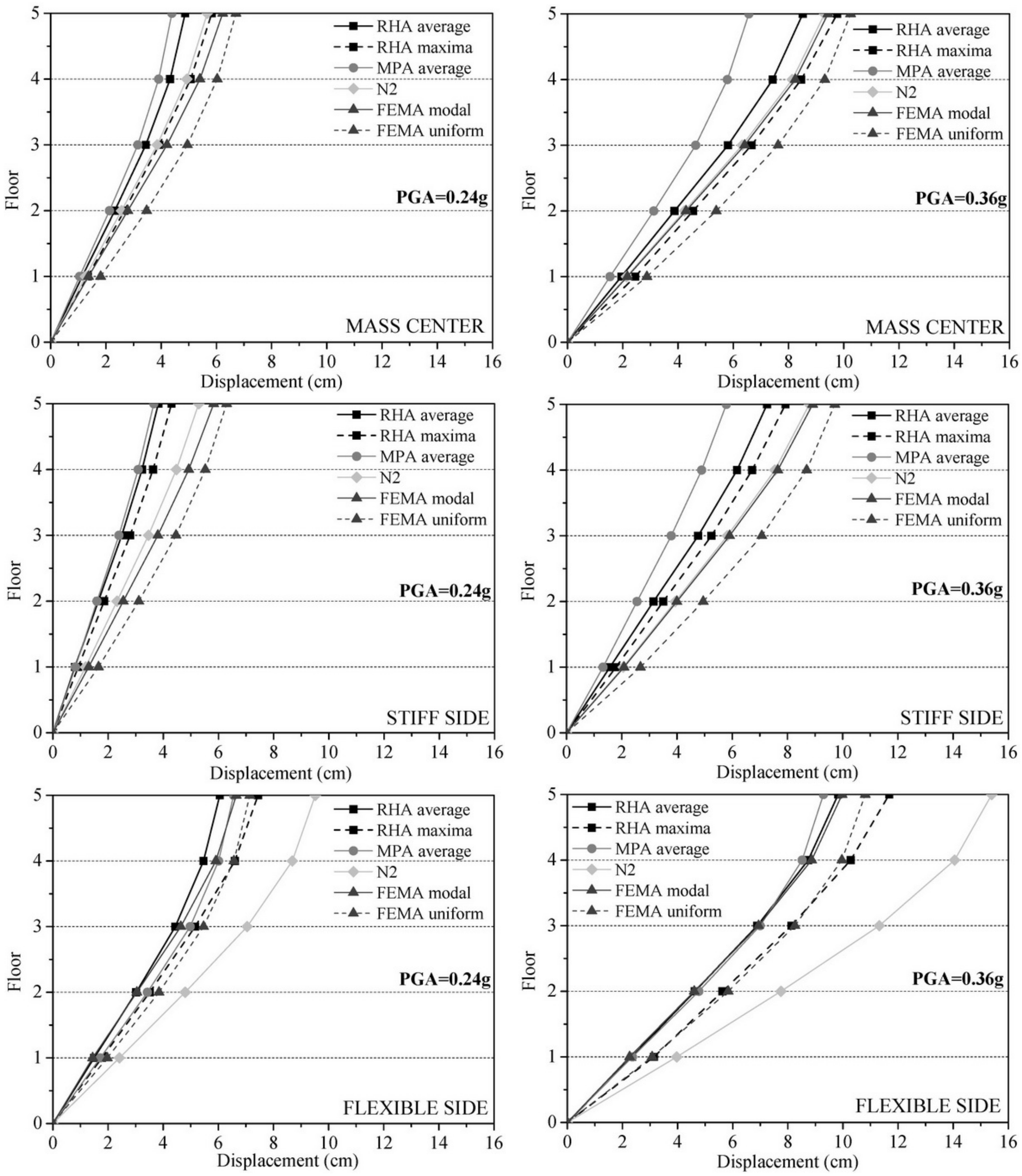

Figure 4 Peak x-displacement profiles at three building locations for two earthquake intensities

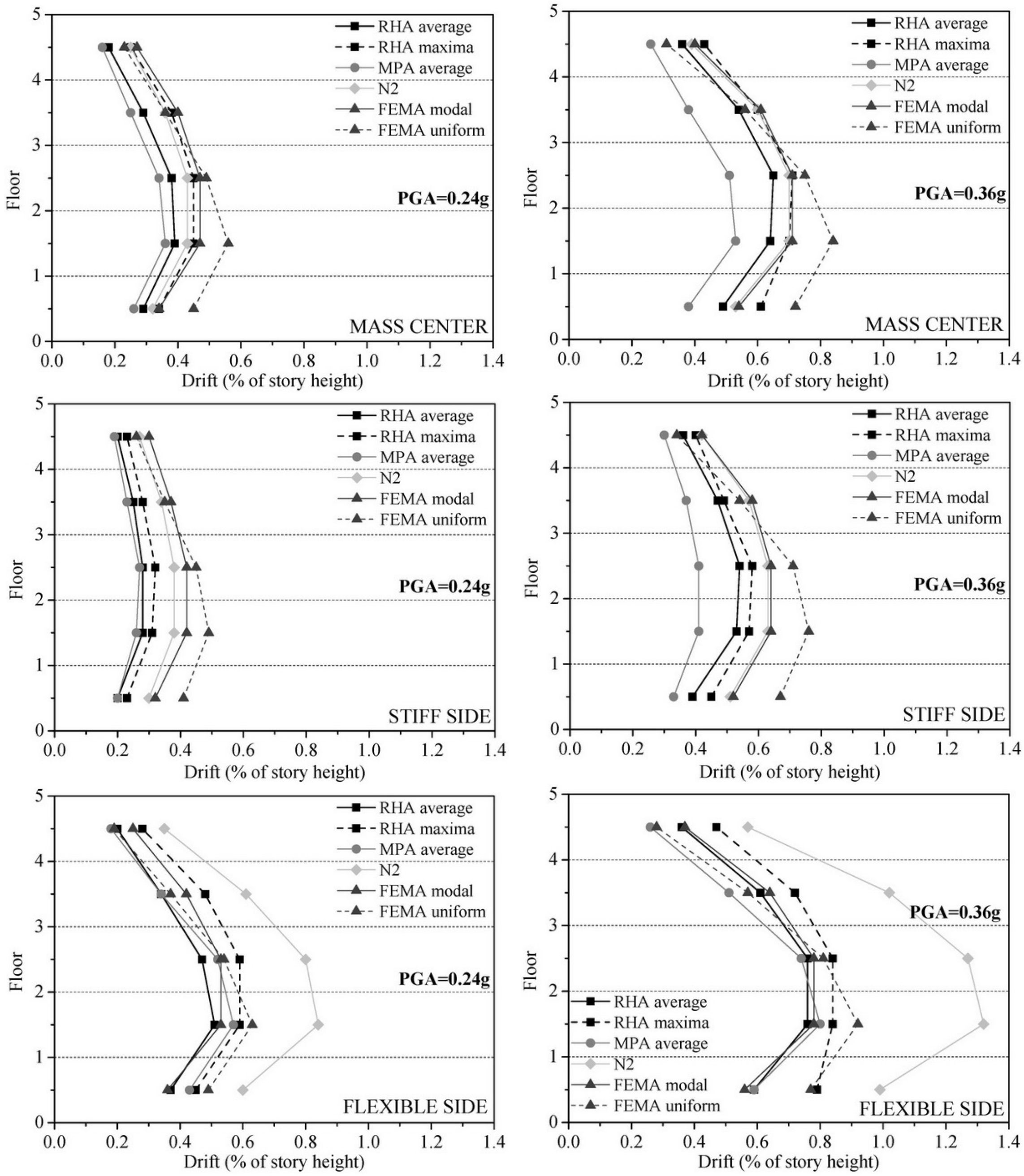

Figure 5 Interstory drifts, x-direction, at three building locations for two earthquake intensities

agreement at the flexible side. The opposite happens with the N2 results: better agreement at the stiff side and substantial overestimation at the flexible side. The differences become greater at the higher motion intensity. The FEMA procedure with a modal load distribution overestimates plastic rotations at the stiff side but shows reasonable agreement at the flexible side. The FEMA uniform distribution is not doing as well as the modal distribution and even if it is used to provide envelope values, it adds no improvement to the results by the modal distribution.

4. CONCLUDING REMARKS

On the basis of the presented results it is concluded that none of the examined pushover methods can give consistently good agreement with the mean RHA results for both the stiff and the flexible sides of an

Table 2 Building top-story average target displacements (cm)

PGA	CASE	FLEXIBLE SIDE	MASS CENTER	STIFF SIDE
0.24g	FEMA-UNIF.	7.14	6.71	6.29
	FEMA-MODAL	6.65	6.23	5.83
	N2	9.51	5.67	5.28
	MPA	6.56	4.39	3.66
	RHA-AVERAGE	6.05	4.87	3.82
	RHA-MAXIMA	7.44	5.83	4.30
0.36g	FEMA-UNIF.	10.81	10.25	9.72
	FEMA-MODAL	9.97	9.43	8.91
	N2	15.40	9.31	8.79
	MPA	9.29	6.57	5.79
	RHA-AVERAGE	9.83	8.51	7.62
	RHA-MAXIMA	11.68	9.76	7.93

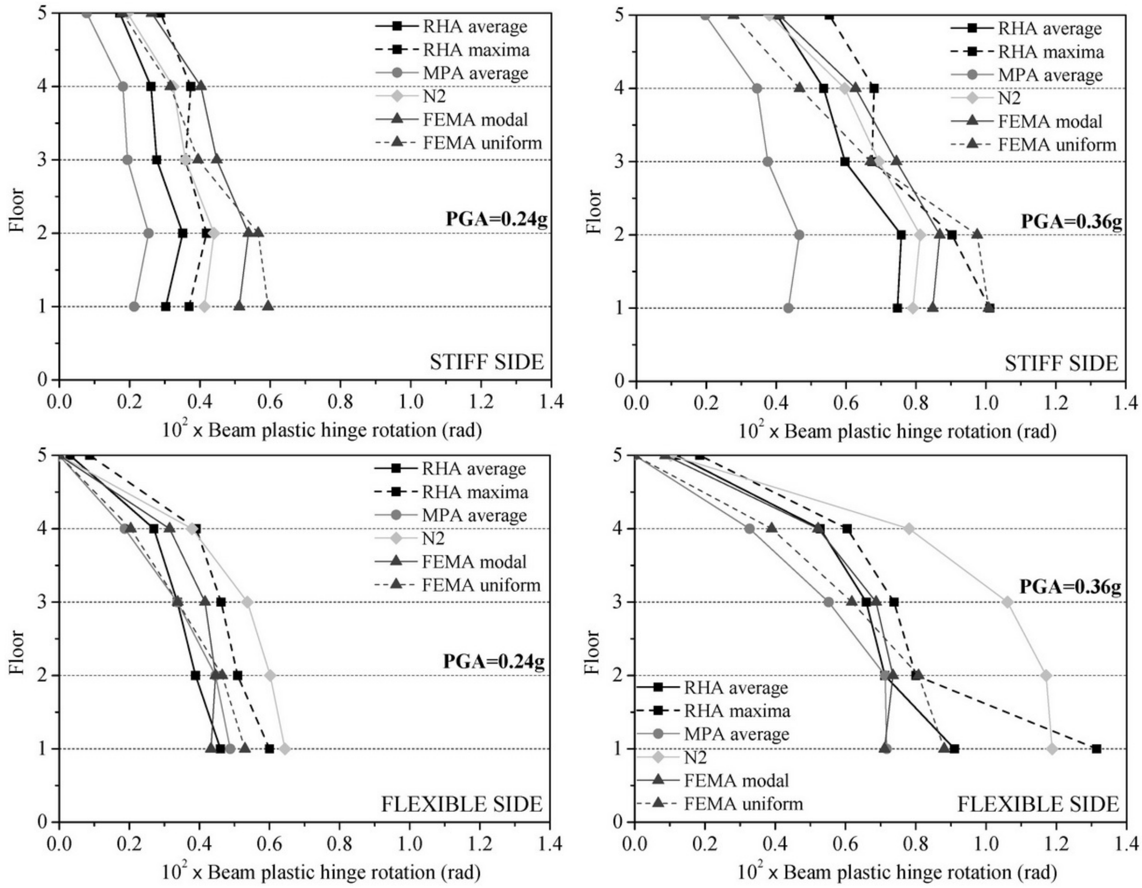

Figure 6 Maximum plastic hinge rotation of beams in x-direction for two earthquake intensities

unsymmetric building undergoing translational and torsional response. The agreement is always better for elements at the center of mass and deteriorates at the two edges where the torsional motion amplifies or de-amplifies the translational response. Moreover the differences tend to increase as the motion intensity increases and the response becomes more non-linear. Thus the problem is still open and until a reliable and simple to apply, pushover type of method that could provide a good assessment of capacity of irregular 3-D structures, with consistent reliability for ALL elements of the building, is devised, the nonlinear dynamic RHA method for a good selection of earthquake records will remain as the method of choice, at least for special and important buildings.

REFERENCES

ATC (Applied Technology Council) (1996). Seismic Evaluation and Retrofit of Concrete Buildings. ATC (Report N$^{o.}$ ATC-40), Redwood City, California.

Aydinoglu M.N. (2003). An incremental response spectrum analysis procedure based on inelastic spectral diplacements for multi-mode seismic performance evaluation. *Bulletin of Earthquake Engineering* **1,** 3-36.

CEN (European Committee for Standardisation) (2004). European (draft) Standard EN 1998-1: Eurocode 8 - Design of Structures for Earthquake Resistance **-** Part 3: Assessment and Retrofitting of Buildings – Stage 49. CEN, Brussels.

Chopra A.K. and Goel R.K. (2002). A modal pushover analysis procedure for estimating seismic demands for buildings. *Earthquake Engineering and Structural Dynamics* **31:3**, 561-582.

Chopra A.K. and Goel R.K. (2004). A modal pushover analysis procedure to estimate seismic demands for unsymmetric-plan buildings. *Earthquake Engineering and Structural Dynamics* **33**, 903-927.

CSi. (Computers and Structures Inc.) (2007). SAP 2000 Advanced Structural Analysis Program. CSi., California

De Stefano M. and Ruttenberg A. (1998). Predicting the dynamic response of asymmetric multistorey wall-frame structures by pushover analysis: two case studies. *Proceedings of the 11th European Conference of Earthquake Engineering*, Balkema, Rotterdam.

Elnashai A.S. (2001). Advanced inelastic static (pushover) analysis for earthquake applications. *Structural Engineering and Mechanics* **12:1**, 51-69.

Fajfar P. (2000). A nonlinear analysis method for performance-based seismic design. *Earthquake Spectra* **16:3**, 573-592.

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

FEMA (Federal Emergency Management Agency) (2000). FEMA 356: Pre-Standard and Commentary for the Seismic Rehabilitation of Buildings. FEMA, Washington D.C.

FEMA (Federal Emergency Management Agency) (2005). FEMA 440: Improvement of Nonlinear Static Seismic Analysis Procedures. FEMA, Washington D.C.

Gates W.E., Marshall P.W. and Mahin S.A. (1977). Analytical methods for determining the ultimate earthquake resistance for fixed offshore structures. *Proceedings of the 9th Annual Offshore Technology Conference*, Houston, Texas.

Gupta B. and Kunnath S.K. (2000). Adaptive spectra-based pushover procedure for seismic evaluation of structures. *Earthquake Spectra* **16**, 367-392.

Kallaby J. and Millman D. (1975). Inelastic analysis of fixed offshore platforms for earthquake loadings. OTC #2357, *Proceedings of the 7th Annual Offshore Technology Conference*, Houston, Texas.

Karabalis D.L., Cokkinides G.J., Rizos D.C., Mulliken J.S., Chen R. (1994). An interactive computer code for generation of artificial earthquake records. *Computing in Civil Engineering (ASEE)*, K. Khozeimeh (ed.), 1122-1155.

Moghadam A.S. and Tso W-K. (1998). Pushover analysis of asymmetrical multistorey buildings. *Proceedings of the 6th U.S, National Conference on Earthquake Engineering*, EERI, Oakland, CA.

OASP (Greek Organization for Seismic Planning and Protection) (2006). Greek Retrofitting Code – second draft version. OASP, Greek Ministry for Environmental Planning and Public Works, Athens, Greece. (In Greek).

Penelis G.G. (2007). A contribution to the inelastic analysis of Reinforced Concrete buildings having non-symmetric layout. PhD dissertation, University of Thessaloniki, Thessaloniki, Greece (in Greek)