SPECTRAL PUSHOVER VERSUS STATIC AND MODAL PUSHOVER ANALYSIS

V.S. Tsiggelis[1] , A.M. Athanatopoulou[2] and I.E. Avramidis[3]

[1] *PhD Candidate, Dept. of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, Greece*
[2] *Assistant Professor, Dept. of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, Greece*
[3] *Professor, Dept. of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, Greece*
Email: avram@civil.auth.gr

ABSTRACT : In this paper a newly developed simplified procedure for estimating seismic demands which is called Spectral Pushover Analysis [SPA] is briefly presented and evaluated. First, the theoretical background is presented, the underlying assumptions and approximations are identified and the successive steps of the method are outlined. Then, in order to evaluate the seismic demands produced by SPA, a parametric study of planar and spatial systems is carried out using: (a) the Nonlinear Static Procedure [NSP] as described in EC8 and ATC55 (=FEMA440), (b) the Modal Pushover Analysis [MPA], (c) the SPA and (d) the Nonlinear Response History Analysis [NLRHA]. A 9-story non-symmetric planar frame as well as a set of single story monosymmetric buildings with different inertial characteristics and eccentricity have been analyzed for 20 recorded ground motions using all the above methods. The comparative evaluation of the results shows that, as far as the planar frame is concerned, the magnitude of errors is similar for all approximate procedures, whereas for the single story buildings SPA produces results that are closer to the "exact" ones produced by NLRHA.

KEYWORDS : Seismic demands, simplified procedures, nonlinear static procedure, modal pushover analysis.

1. INTRODUCTION

Estimating seismic demands at low performance levels, such as life safety and collapse prevention, requires explicit consideration of the inelastic behavior of the structure. While nonlinear response history analysis is the most rigorous procedure to compute seismic demands, it is impractical for routine use. It is now common to estimate seismic demands in a simplified manner by nonlinear static analysis (pushover analysis), which seems to be the preferred method in structural engineering practice. However, this method has some shortcomings and limitations as a result of its underlying assumptions [Chopra and Goel (2001)]. In fact, the static pushover analysis can be reliably applied only to two-dimensional low- and medium-rise buildings that vibrate primarily in the fundamental mode [Krawinkler and Seneviratna (1998)]. During the last ten years much research work has been done in order to improve the results produced by the different simplified nonlinear static procedures suggested by codes [ATC40 (1997), ATC55 (2005), FEMA274 (1997), FEMA356 (2000), EC8 (2002)], as well as to develop new more accurate ones. Concerning the latter aspect, Chopra and Goel developed the so-called Modal Pushover Analysis [MPA] [Chopra and Goel (2001)] in order to take into account the higher-mode contributions. However, the application of this procedure leads in many cases to anomalous capacity curves due to the fact that the higher modes' roof displacements often change their signs. Another simplified nonlinear procedure, the Spectral Pushover Analysis [SPA], has been developed by Anastasiadis [Anastasiadis (2001)]. SPA consists in a mutatis mutandis "translation" of the classical pushover analysis from the static analysis domain to the response spectrum analysis domain. Further improved simplified nonlinear procedures have been proposed by several researchers and their pros and cons have been the subject of many papers. Here, the focus is on the SPA and its comparative performance in refer to NLRHA, to MPA and to the procedures in ATC55 and EC8.

First, the theoretical background of SPA is presented, the assumptions and approximations are identified and the successive steps of the method are outlined (paragraph 2). Then, in order to check the practical applicability, the relative ease of use and the accuracy of the results produced by SPA as well as by the aforementioned code prescribed nonlinear static procedures, a RC planar frame and a set of single story monosymmetric buildings with different inertial characteristics and eccentricity are studied (see paragraph 3). The results produced by SPA are compared with the associated ones produced by NLRHA.

2. SPECTRAL PUSHOVER ANALYSIS PROCEDURE [SPA]

The basic idea of this procedure consists in the direct "translation" of the classical pushover analysis from the Static Analysis domain to the Response Spectrum Analysis domain. In this domain "loading" is expressed through an elastic design or response acceleration spectrum $S_a(T,\zeta)$, depending on the natural period T and the damping ratio ζ. In the context of a pushover analysis, such a "loading" is applied incrementally, as a small portion of the spectrum. For each spectral increment, the structure under consideration is analyzed using iteratively the traditional linear response spectrum method. Figure 1 shows the spectrum portions for n increments, where λ_k (k=1, 2, …, n) is the proportional spectrum partition. Obviously, the sum of all λ_k is unity:

$$\sum_{1}^{n} \lambda_k = 1 \qquad (k=1, 2, \ldots, n) \tag{2.1}$$

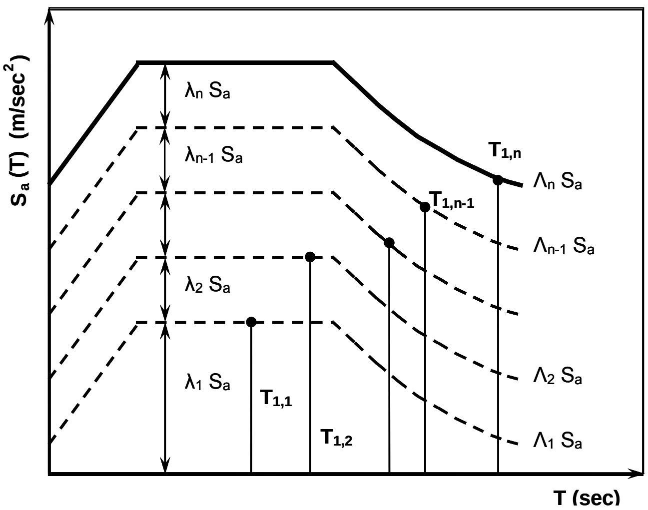


Figure 1 Spectrum portions

By analogy with the classical pushover analysis, the partitioned spectrum is applied incrementally to the structure, starting from the first spectral increment $\lambda_1 \times S_a$ and adding succesively a new spectral increment $\lambda_k \times S_a$ (k=2, …, n) each time one structural element section(s) yields (formation of plastic hinge(s)). The new spectral increment is applied to a revised model of the structure which differs from the previous one Π_{k-1}, as a different stiffness matrix is used for the structural element(s) with the plastic hinge(s). For each spectral increment $\lambda_k \times S_a$, a linear response spectrum analysis of model Π_{k-1} is performed and the peak seismic response is computed and added to the sum of those from the previous increments. Thus, the total response of the nonlinear system is estimated by successive contributions of responses of the linear systems Π_k.

2.1. Flowchart of SPA

The SPA procedure consists of a series of step-by-step computations with systematic updates being performed at the end of each step. The successive steps are presented in Figure 2 in the form of flowchart. The following notations are used:

k : Step number of spectral increment with k= 1, 2, …, n.

Π_k : Model of the structure at step k (Π_0 is the initial model).

$\{F_y\}_{k,i}$: Available strength at critical cross-section i of model Π_{k-1} ($\{F_y\}_{0,i}$ is the yield strength at critical cross-section i in the unloaded structure of model Π_0 and $\{F_y\}_{1,i}$ is the available strength at critical cross-section i of model Π_0 after the action of gravity loads).

$\{F\}_i$, : Response quantity (bending moment, axial or shear force) at critical cross-section i.

$\{R\}$: Response quantity (force or deformation).

$\{F_g\}_i$, $\{R_g\}$: Gravity load effects in model Π_0 ($\{F_g\}_{0,i}=\{F_g\}_i$, $\{R_g\}_0=\{R_g\}$).

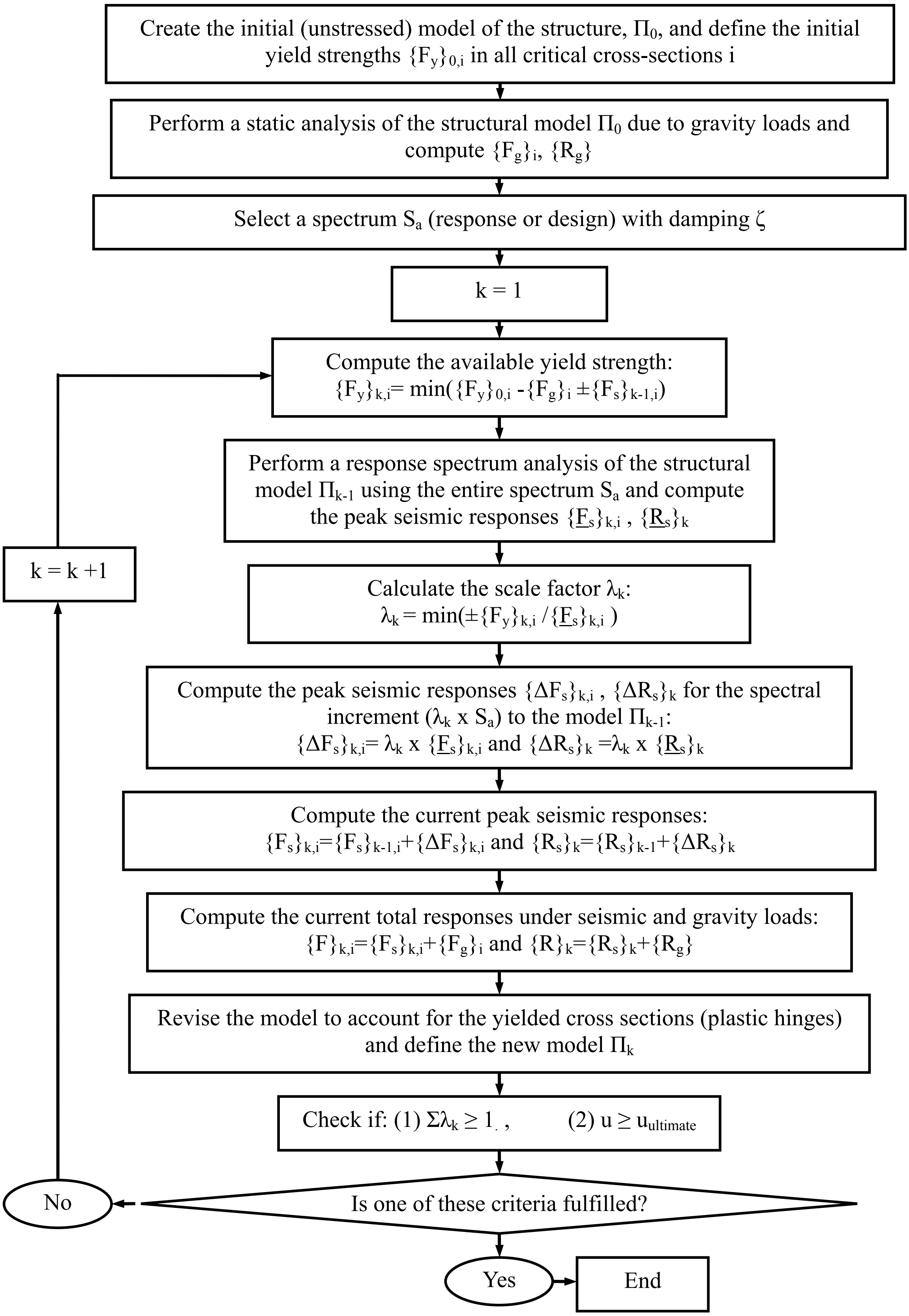


Figure 2 Flowchart of SPA

$\{\underline{F}_s\}_{k,i}$, $\{\underline{R}_s\}_k$: Peak seismic responses of model Π_{k-1} under the action of the entire spectrum S_a ($\{\underline{F}_s\}_{0,i}$=0, $\{\underline{R}_s\}_0$=0).
$\{\Delta F_s\}_{k,i}$, $\{\Delta R_s\}_k$: Increase of peak seismic responses under the action of spectral increment $\lambda_k \times S_a$ on the model Π_{k-1} ($\{\Delta F_s\}_{0,i}$=0, $\{\Delta R_s\}_0$=0).
$\{F_s\}_{k,i}$, $\{R_s\}_k$: Sum of $\{\Delta F_s\}_{k,i}$, $\{\Delta R_s\}_k$ from step 1 till step k ($\{F_s\}_{0,i}$=0, $\{R_s\}_0$=0).
$\{F\}_{k,i}$, $\{R\}_k$: Current total responses under seismic and gravity loads.

2.2. Significant assumptions of SPA

Initially, the following assumptions are made concerning the inelastic behavior of structural elements: a) inelastic beam and column members are modeled as elastic elements with plastic hinges forming only at their ends, b) all plastic hinges are of flexural type, c) the plastic hinges are described by elastic-perfectly plastic moment-rotation relationships, d) no limit value is assumed for the sections' plastic rotations, e) P-δ effects are neglected and f) the effect of cyclic earthquake loading on the inelastic response of cross-sections is neglected.

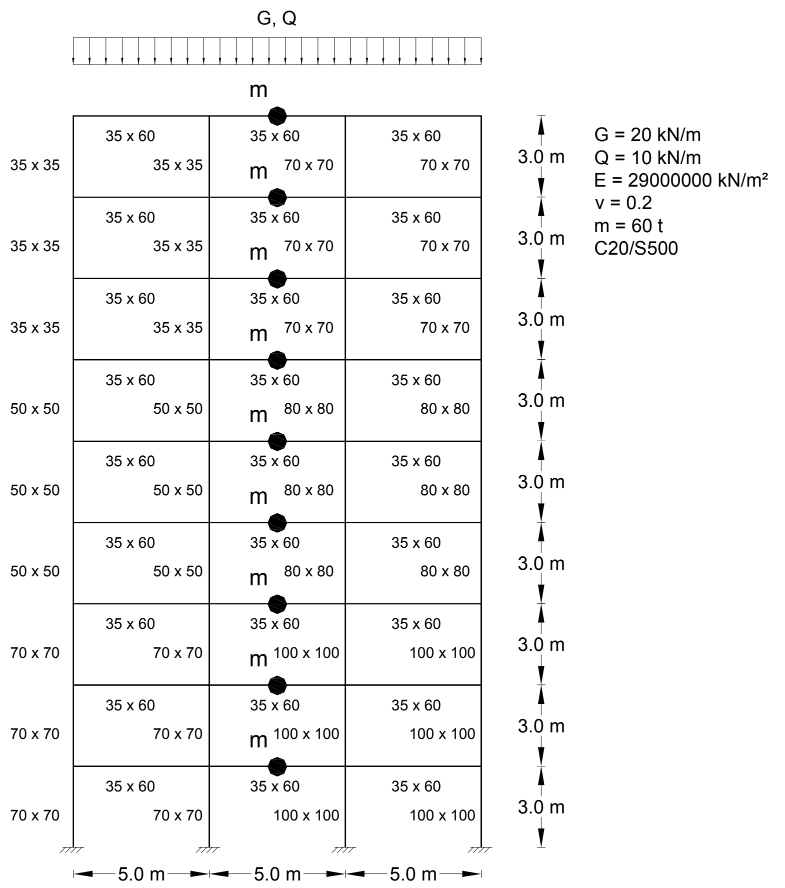
Except the assumptions concerning the inelastic behavior of structural elements, some additional assumptions are adopted in order to apply SPA. Firstly, the direct addition of the peak responses of the successive structural models is accepted as valid. Secondly, we make the particular assumption that the peak responses at different cross sections and joints of model Π_k are concurrent. This is a conservative approximation leading to a cumulative magnification of responses, because in reality these peak responses are not concurrent. Moreover, one more assumption concerns the position of the new plastic hinges in columns of the structure. As the behavior of a beam element is controlled by one parameter -bending moment- a new plastic hinge is formed at the cross section where $\{M\}_{k,i}$ becomes equal to yield strength (moment) $\{M_{,y}\}_{0,i}$. However, the formation of a plastic hinge at a column element is more complicated due to the bending moment-axial force interaction and, in addition, due to the indefinability of algebraic signs for bending moments and axial forces in the modal combination. Plastic hinge at a column cross section is assumed to be formed only if either the two points defined by (+M, +N) and (+M, -N) or the two points (-M, +N) and (-M, -N) are outside the bending moment-axial force interaction surface. A last assumption concerns the question: When must the SPA procedure be terminated? At first, it seems to be rational to terminate the procedure only if the sum of factors λ_k for all steps becomes equal to unity. In spite of that, the analyses' results of several structural systems produced by SPA have indicated that the errors are strongly dependent on ground motion intensity. This fact has led to the prescription of an additional limit concerning the total displacement: $u \leq u_{ultimate}$. However, the choice of $u_{ultimate}$ is based inevitably on engineer's judgement (for example, $u_{ultimate}$ can be the value of the top-floor displacement determined by a response spectrum analysis of the initial structural model).

3. APPLICATIONS

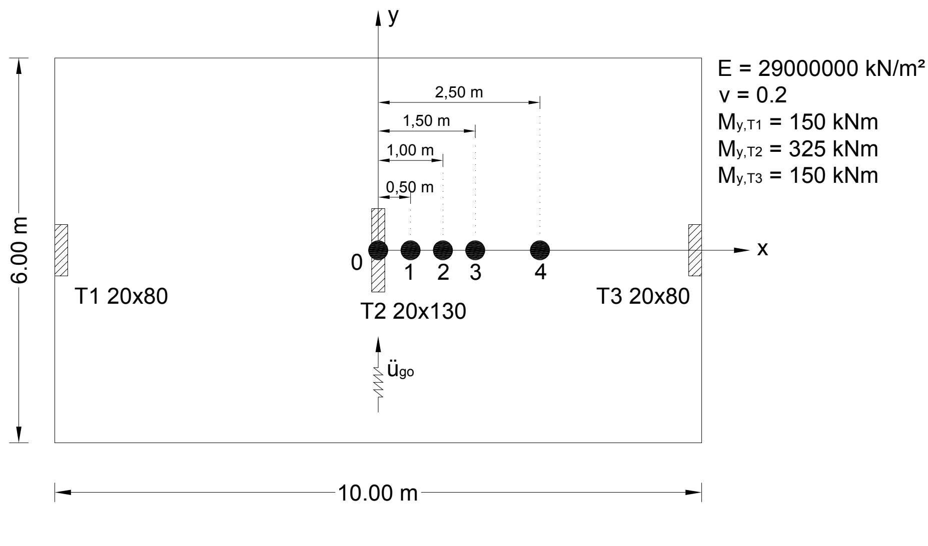
A critical and comprehensive evaluation of the methodology is performed by means of analyzing a variety of structural systems using the proposed SPA as well as the conventional static pushover procedures. The effectiveness of this procedures to capture the inelastic response of structures is evaluated on the basis of results produced by nonlinear time-history analysis, which is used here as a reference solution.

3.1. Structural Models, Ground Motions And Methods of Analyses

Several planar frames are analyzed in order to cover a wide range of fundamental periods. In this paper, an non-symmetric, 3-bay, 9-story, RC frame with constant story height is selected to be presented (Fig 3a). The seismic excitation is represented by the response spectrums of 20 recorded ground motions listed in Table 3.1. For the same ground motions and the corresponding response spectrums a set of single story monosymmetric buildings with different inertial characteristics and eccentricity are analyzed (Fig 3b).



(a)



Fundamental natural period

	$T_1 = 0.2$ sec		$T_1 = 0.5$ sec	
Σημείο 0:	m = 26.55 t	J_m = 300.88 t m²	m = 165.93 t	J_m = 1880.52 t m²
Σημείο 1:	m = 24.94 t	J_m = 282.62 t m²	m = 155.85 t	J_m = 1766.36 t m²
Σημείο 2:	m = 21.94 t	J_m = 248.69 t m²	m = 137.14 t	J_m = 1554.29 t m²
Σημείο 3:	m = 18.99 t	J_m = 215.18 t m²	m = 118.67 t	J_m = 1344.88 t m²
Σημείο 4:	m = 14.12 t	J_m = 160.07 t m²	m = 88.28 t	J_m = 1000.46 t m²

(b)

Figure 3 Geometric and mechanic characteristics of the investigated systems: (a) non-symmetric planar 9-story RC frame, (b) single story monosymmetric buildings (ten different cases)

Each system is analyzed using all methods listed in Table 3.2. The results derived by LRSA are used to define the $u_{ultimate}$. Non-Linear Response History Analysis (NLRHA) is performed in order to obtain the "exact" values of the response. The target roof displacement for NSP according to FEMA is determined by: (i) Equivalent Linearization and (ii) Displacement Modification. For NSP according to EC8 the target displacement is determined by NLRHA of an "equivalent" single-degree-of-freedom-system (SDOF). For the application of MPA to the planar frame three "modes" are taken into account and the peak "modal" displacement of each inelastic SDOF system is determined by NLRHA. The single-story spatial buildings are analyzed for unidirectional excitation along the y-axis (Fig. 3b).

Table 3.1 List of ground motions

	Earthquake Name and Location	$\ddot{u}_{go}$ (cm/sec^2)
1	1989 Loma Prieta (Agnews State Hospital)	169
2	1989 Loma Prieta (Capitola)	435
3	1989 Loma Prieta (Gilroy Array #3)	360
4	1989 Loma Prieta (Gilroy Array #4)	208
5	1989 Loma Prieta (Gilroy Array #7)	221
6	1989 Loma Prieta (Hollister City Hall)	242
7	1989 Loma Prieta (Hollister Diff. Array)	274
8	1989 Loma Prieta (Sunnyvale - Colton Ave.)	203
9	1994 Northridge (Canoga Park)	412
10	1994 Northridge (LA - N Faring Rd)	268
11	1994 Northridge (LA - Fletcher Dr)	236
12	1994 Northridge (Glendale - Las Palmas)	202
13	1994 Northridge (LA - Hollywood Stor FF)	227
14	1994 Northridge (La Crescenta - New York)	156
15	1994 Northridge (Northridge - Saticoy St)	361
16	1971 San Fernando (LA - Hollywood Stor Lot)	171
17	1987 Superstition Hills (Brawley)	153
18	1987 Superstition Hills (El Centro Imp. Co. Center)	351
19	1987 Superstition Hills (Plaster City)	182
20	1987 Superstition Hills (Westmorland Fire Station)	169

Table 3.2 Methods of analyses

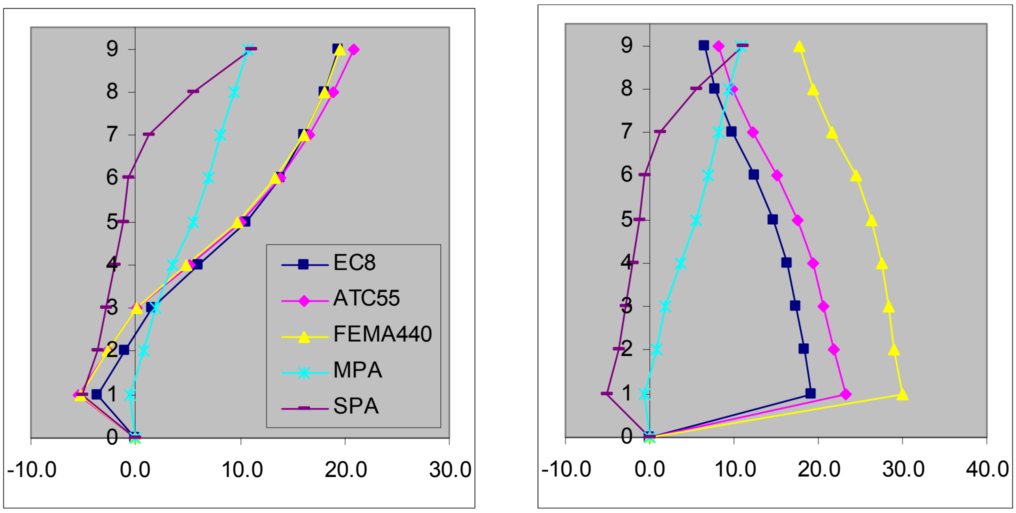
	Procedure		9-storey planar frame	Single-storey monosymmetric building
1	Linear Response History Analysis (LRHA)		☑	☑
2	Linear Response Spectrum Analysis (LRSA)		☑	☑
3	NonLinear Response History Analysis (NLRHA)		☑	☑
4	Nonlinear Static Procedure-Equiv. Lin. (NSP- ATC55)	NSP's	☑	☑
5	Nonlinear Static Procedure-Disp. Mod. (NSP- FEMA440)	NSP's	☑	☑
6	Nonlinear Static Procedure EC8 (NSP- EC8)	NSP's	☑	☑
7	Modal Pushover Analysis (MPA)		☑	☑
8	Spectral Pushover Analysis (SPA)		☑	☑

3.2. Analyses results

The median error of a response value determined by each one of the applied methods with respect to the corresponding "exact" value determined by NLRHA is calculated using the following formula:

$$\text{error}\ (\%) = \frac{1}{20}\sum_{1}^{20}\left(\frac{u_{iP} - u_{iD}}{u_{iD}}\right)\cdot 100 \qquad (3.1)$$

where u_{iP} is the displacement at level i determined by the approximate method P (P: methods no. 4 to 8 of Table 3.2) and u_{iD} is the corresponding displacement derived by NLRHA. The closer the median error to zero, the closer the response derived from the respective approximate method to the "exact" value. The approximate method underestimates the median response if the ratio (Eq. 3.1) is less than zero, and provides an overestimate if the ratio exceeds zero. The present study is limited to the evaluation of displacements. The median errors for the floor displacements of planar frame are illustrated in Figure 4, while the median errors for the displacements of CM and the left side of the spatial models are shown in Tables 3.3 and 3.4, respectively.



NSP: “modal” distribution NSP: “uniform” distribution

Figure 4 Median error of roof displacements of 9-storey frame

Table 3.3 Median error of displacements of CM

Model	CM	NSP- ATC55	NSP- FEMA440	NSP- EC8	MPA	SPA
	0	-0.74	11.52	23.22	4.50	-1.82
	1	12.02	24.88	28.10	-10.20	0.30
T_1=0.2sec	2	20.43	27.34	33.71	-10.91	-11.27
	3	17.02	22.75	25.37	-9.51	-10.42
	4	6.20	11.74	13.02	0.97	-1.23
	0	80.57	24.83	19.74	19.01	24.95
	1	95.49	35.85	29.23	1.98	-17.42
T_1=0.5sec	2	114.71	31.38	43.52	-24.28	-2.72
	3	153.40	33.66	62.76	-16.87	10.90
	4	138.99	33.05	55.34	-10.34	6.75

Table 3.4 Median error of displacements of left (stiff) side

Model	CM	NSP- ATC55	NSP- FEMA440	NSP- EC8	MPA	SPA
	0	-0.74	11.52	23.22	4.50	-1.82
	1	-45.57	-43.86	-42.45	43.54	24.96
T_1=0.2sec	2	-71.67	-71.07	-71.02	21.50	15.18
	3	-95.79	-95.79	-95.78	25.48	16.05
	4	-58.30	-58.26	-57.24	17.21	12.42
	0	80.57	24.83	19.74	19.01	24.95
	1	-88.83	-88.83	-88.83	174.52	0.46
T_1=0.5sec	2	-93.32	-93.32	-93.32	166.71	28.29
	3	-98.93	-98.93	-98.93	234.43	63.19
	4	-88.43	-88.43	-88.43	202.79	75.29

4. CONCLUSIONS

The presented SPA of structures is a step-wise linear method in which the traditional LRSA is applied iteratively. From the comparative study of the investigated wide-spread approximate nonlinear methods the following conclusions are drawn:

a) Planar frame: The magnitude of errors is similar for all approximate procedures, when the "modal" load distribution is used in the NSP's (NSP- ATC55, NSP- FEMA440, NSP- EC8), while MPA and SPA are more accurate than the NSP's when the "uniform" load distribution is used. Furthermore, both MPA and SPA provide good estimates for floor displacements. However, the computational effort required for MPA is larger than for SPA.

b) Spatial models with fundamental period T_1=0.2sec (Fig. 3b): MPA and SPA underestimate in general the displacements of the Center of Mass (CM) and of the building's right side (~-20% till +5%), while NSP's overestimates them. The opposite occurs for the left side displacement and the diaphragm rotation: MPA and SPA overestimate these values, while NSP's practically fails to estimate them (underestimation of ~-50% to -90%).

c) Spatial models with fundamental period T_1=0.5sec (Fig. 3b): MPA and SPA may overestimate or underestimate the displacements of CM and of the building's right side (~±20%), while NSP's overestimates these quantities (~+50%). Moreover, NSP's practically fails to predict the demands of the left side (errors are -90%). Both MPA and SPA overestimate them, but results obtained by MPA are less accurate than SPA (errors range between +160% and +240% in MPA and between +25% and 75% in SPA).

d) As a general conclusion it can be said that the presented spectral pushover analysis (SPA) possesses some important advantages with regard to the conventional approximate nonlinear procedures: It permits in a straight forward way to account for (a) the step-wise change of structural characteristics (natural periods, modes), (b) the higher mode effects as well as (c) the torsional effects in non-symmetric structures. Moreover SPA does not require neither the compilation of a pushover curve, nor the application of several height-wise distributions of lateral loads. Obviously, further investigation is needed in order to derive conclusive results. However, the presented preliminary evaluation gave very encouraging and promising results.

REFERENCES

ATC40. (1997). Seismic Evaluation and Retrofit of Concrete Buildings, Applied Technology Council, Redwood City, California.

ATC55 (FEMA440). 2004. Improvement of Nonlinear Static Seismic Analysis Procedures, Applied Technology Council, Redwood City, California.

Anastassiadis, K.K. (2001). Spectral pushover analysis for assessment of seismic behavior of buildings. *2nd Greek Conference on Earthquake Engineering and Seismology*, Salonika (in Greek).

Chopra, A.K, and Goel, R.K. (2001). A Modal Pushover Analysis Procedure to Estimate Seismic Demands for Buildings: Theory and Preliminary Evaluation, PEER Report 2001/03, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

EC8, CEN Techn. Comm. 250 / SC8. Eurocode 8: Earthquake Resistant Design of Structures-Part 1: General rules. (ENV1998-1, Parts 1,2,3), CEN. Brussels, 1994, 1995.

FEMA274 (NEHRP). (1997). Commentary on the Guidelines for the Seismic Rehabilitation of Buildings. Prepared by ATC, Washington, D.C.

FEMA356. (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings. Prepared by ASCE, Washington, D.C.

Krawinkler H. and Seneviratna G.D.P.K. 1998. Pros and Cons of a Pushover Analysis of Seismic Performance Evaluation. *Engineering Structures*, **Vol. 20**, **No. 4-6**, pp. 452-464.