Structural Engineering and Mechanics, Vol. 41, No. 2 (2012) 243-262
DOI: http://dx.doi.org/10.12989/sem.2012.41.2.243

Assessment of FEMA356 nonlinear static procedure and modal pushover analysis for seismic evaluation of buildings

Hamid Reza Khoshnoud* and Kadir Marsono[a]

Faculty of Civil Engineering, University of Technology of Malaysia, Skudai, Johor, Malaysia

(Received August 13, 2010, Revised December 29, 2011, Accepted January 3, 2012)

Abstract. Nonlinear static analysis as an essential part of performance based design is now widely used especially at design offices because of its simplicity and ability to predict seismic demands on inelastic response of buildings. Since the accuracy of nonlinear static procedures (NSP) to predict seismic demands of buildings affects directly on the entire performance based design procedure, therefore lots of research has been performed on the area of evaluation of these procedures. In this paper, one of the popular NSP, FEMA356, is evaluated and compared with modal pushover analysis. The ability of these procedures to simulate seismic demands in a set of reinforced concrete (RC) buildings is explored with two level of base acceleration through a comparison with benchmark results determined from a set of nonlinear time history analyses. According to the results of this study, the modal pushover analysis procedure estimates seismic demands of buildings like inter story drifts and hinges plastic rotations more accurate than FEMA356 procedure.

Keywords: nonlinear static analysis; nonlinear time history analysis; modal pushover analysis; FEMA356; inter story drift; plastic hinge rotation; performance based design

1. Introduction

The increased employment of static nonlinear analysis at practical level reveals a major trend toward using the advantages of nonlinear analysis in finding inelastic response of buildings in a moderate or strong earthquake. The main specification of nonlinear procedure in comparison to linear one is the extended analysis area to inelastic response of system. Although nonlinear time history analysis (NLTH) is the final solution, but its intrinsic complexity and the required additional efforts regarding to thousands run steps for several ground motions causes NLTH to be limited to research area rather than design offices. One of the most popular static nonlinear procedures is pushover analysis which included in several seismic codes like Eurocode8 (CEN 2004, 2005), ATC40 (ATC 1996), FEMA356 (ASCE 2000).

The Pushover analysis is a series of incremental linear analyzes that in each step, a portion of lateral load is applied to the structure (Marsono and Khoshnoud 2010). For monitoring the material nonlinear behavior of elements especially for yielding and post-yielding behavior, plastic hinges or

*Corresponding author, Ph.D. Candidate, E-mail: hrkhoshnoud@yahoo.com
[a]Associate Professor, E-mail: Akadir@utm.my

plastic zones can be defined in two ends of beams or columns or any other locations of elements in which a plastic area may be formed. In each series of linear analysis, the response of system will be determined regarding the assumption that the stiffness of the structure is constant. According to the results of each iteration, the yielding of each element is checked based on predefined criteria. If yielding is occurred the stiffness of structure is modified, lateral load is proportionally increased and another static analysis is performed. This process will continue until lateral roof displacement of building reaches to a predefined target displacement or a mechanism is formed. The result generally is presented in the form of base shear verses top story displacement. The above procedure currently is used in most seismic codes. Two main ideas of this procedure are the seismic behavior of structure based on first mode of vibration and the constant dynamic specifications of structure during the analysis. These two ideas generally are not correct for all buildings (Krawinkler 1998) especially for those that higher modes effects are important. On the other hand with forming plastic zones in structure, it loses its stiffness. Therefore the periods and mode shapes of system will be changed during the analysis. In N2 method (Fajfar 2000), the pushover analysis of MDOF system is combined with the response spectrum of equivalent SDOF system. In this method, the usage of inelastic spectra rather than elastic spectra is one of the main advantages of this method over conventional method like ATC40. In the modal pushover analysis (MPA) (Chopra *et al*. 2002) the seismic demand is obtained by pushover analysis for whole model (MDOF) and nonlinear time history analysis for an equivalent SDOF unless an inelastic response (or design) spectrum is available. This procedure must be iterated for each number of desire first modes and combination of these "modal" demands due to the first (normally two or three) modes provides an evaluation of the total seismic demand on inelastic systems.

In modified modal pushover analysis (MMPA) (Chopra *et al*. 2004) it is assumed that the response of building for higher modes is linear. So in this procedure the elastic influence of higher modes combined with the inelastic response of first mode reduce the computational effort. With the above assumption it can be concluded that MMPA procedure is not more accurate than MPA procedure. In the adaptive pushover analysis (APA), load vectors are progressively updated to consider the change in system modal attributes during inelastic phase (Gupta 2000). More recently, a new adaptive modal combination (AMC) procedure, whereby a set of adaptive mode-shape based inertia force patterns is applied to the structure, has been developed (Kalkan 2006). Although the non adaptive pushover analysis procedures are not necessarily more accurate than adaptive procedures, but their simplicities causes more trend toward using of them especially at practical level. Recently, there are many researches on assessment of current nonlinear static procedure for seismic evaluation of buildings (Kalkan June 2006, Poursha 2008). In this paper, the advantages and limitations of two invariant load procedures of FEMA356 and MPA are considered. The ability of these procedures to simulate seismic demands in a set of reinforced concrete (RC) buildings with two level of base acceleration is explored through comparisons with benchmark results determined from a set of nonlinear time history analyses.

2. Pushover analysis based on FEMA356

FEMA356 has recommended the application of two sets of lateral load distribution for all analysis. One set is a vertical pseudo lateral load (based on equation 3-12 of FEAM356) or a vertical lateral load proportional to first mode or a vertical lateral load proportional to story shear distribution computed via response spectrum analysis. Second set is a uniform lateral load proportional to the total mass at each level or an adaptive lateral load based on yielding of structure. In this study from set one a vertical pseudo lateral load and a vertical lateral load proportional to the first mode and from second set a uniform lateral load have been employed. The results presented in this paper are enveloped of two load sets.

3. Modal Pushover Analysis (MPA)

In the modal pushover analysis (MPA), which has been developed by Chopra and Goel (2002), the seismic demand is determined by pushover analysis for whole model (MDOF) and nonlinear time history analysis for an equivalent single degree of freedom or the peak value can be estimated from the inelastic response (or design) spectrum (Chopra 2007, section 7.6 and 7.12.1) for each modes. Combining these "modal" demands due to the first two or three modes provides an evaluation of the total seismic demand on inelastic systems. Details of the implementation are described in Chopra and Goel (2002). In the following, a brief explanation for MPA procedure is presented. The governing equation on the response of a multistory building with linear response is

$$mu'' + cu' + ku = -miu''_g(t) \tag{1}$$

Where u is the vector of N lateral floor displacements relative to ground, m, c and k are the mass, classical damping and lateral stiffness matrices of the systems $u''_g(t)$ and is the horizontal earthquake ground motion and each element of influence vector i is equal to unity. In a system with linear response, the lateral forces f_s have a linear relation with displacement vector u and stiffness of k as ku. It means the stiffness of system during the analysis does not change. Therefore the response of the system has a constant slope as k. With the formation of plastic hinges in the structure, it losses its stiffness so the lateral forces f_s has a nonlinear relation with displacement vector u. For the matter of simplicity, for each structural element, the nonlinear relation can be idealized as a bilinear curve. On the other hand, the unloading and reloading curves differ from the initial loading branch. Thus, for each displacement point like u_1 is more than one lateral force f_s. So for finding f_s, it is necessary to know the path history of displacement because the amount of f_s is depending on the path of loading or unloading. First differential of displacement u or u' (speed vector) can give the path history of loading, therefore in inelastic system Eq. (1) is as shown below

$$mu'' + cu' + f_s(u, sign u') = -miu''_g(t) \tag{2}$$

It can be shown that with assumption of $u = D_n\Phi_n\Gamma_n$, Eq. (2) will be as follows

$$D''_n + 2\zeta_n\omega_n D'_n + \frac{F_{sn}}{L_n} = -u''_g(t) \tag{3}$$

With

$$F_{sn} = F_{sn}(D_n, sign D'_n) = \phi_n^T f_s(D_n, sign D'_n) \tag{4}$$

Eq. (3) is the governing equation for the n^{th} mode inelastic SDOF system with natural frequency

ω_n and damping ζ_n and modal coordinate D_n. The Eq. (3) can be solved if the relation of F_{sn}/L_n and D_n are available. If the curve of base shear and displacement V_{bn}-u_m is obtained from a pushover analysis for whole structure then it can be converted to F_{sn}/L_n-D_n as shown in Eq. (5)

$$F_{sn} = \frac{V_{bn}}{\Gamma_n}, \quad D_n = \frac{u_{rn}}{\Gamma_n \phi_{rn}} \tag{5}$$

Or

$$\frac{F_{sn}}{L_n} = \frac{V_{bn}}{L_n \Gamma_n} = \frac{V_{bn}}{M_n^*}, \quad D_{ny} = \frac{u_{rny}}{\Gamma_n \phi_{rn}} \tag{6}$$

F_{sn}/ L_n is acceleration because it is from dividing force of F_{sn} by mass of L_n. On the other hand we have

$$\frac{F_{sn}}{L_n} = \omega_n^2 D_{ny} \tag{7}$$

The term of $\omega^2{}_n D_{ny}$ is acceleration too. Knowing F_{sny}/ L_n and D_{ny} from Eq. (6), the elastic vibration period T_n of the n^{th} mode inelastic SDOF system is computed from

$$T_n = 2\pi\left(\frac{L_n D_{ny}}{F_{sny}}\right)^{1/2} \tag{8}$$

This value of T_n, which may differ from the period of the corresponding linear system, should be used in Eq. (3). Therefore MPA procedure could be summarized as bellow (Chopra and Goel 2002)

1. Compute ω_n and modes Φ_n for linear elastic vibration of the building.
2. For the n^{th}-mode, develop the base shear-roof displacement, V_{bn}-u_m pushover curve for force distribution $s^*{}_n = m\ \Phi_n$.
3. Idealize the pushover curve as a bilinear curve.
4. Convert V_{bn}-u_m to F_{sn}/L_n-D_n curve by using Eq. (6), $\Gamma_n = \Phi^T{}_n m1/\Phi^T{}_n m\Phi_n$.
5. Compute peak deformation D_n of the n^{th}-mode inelastic SDOF system define by the force-deformation relation and damping ratio ζ_n and the elastic vibration period T_n by Eq. (8). Peak deformation D_n can be calculated by nonlinear time history analysis (NLTH) or from the inelastic design spectrum. The authors of current paper have been developed a computer program for solving nonlinear time history of SDOF systems.
6. Compute peak roof displacement u_m associated with the n^{th} mode inelastic SDOF system from $u_m = \Gamma_n \Phi_m D_n$.
7. From the pushover database (step 2), extract values of desired response r_n (floor displacement, story drifts, plastic hinge rotations, etc.) at peak roof displacement u_m computed in step 6.
8. Repeat steps 3-7 for as many modes as required for sufficient accuracy. Typically, the first two or three modes will suffice.
9. Determine the total response (demand) by combining the peak modal responses using the SRSS rule: $r = \sqrt{\sum_n r_n^2}$

4. Nonlinear time history program for SDOF systems

For finding peak response for each SDOF systems, a nonlinear dynamic analysis program, based on direct integration method, is developed by the authors. The pseudo code of program is included point scale algorithm; main body algorithm and the Coverage algorithm utilize modified Newton-Raphson iteration is shown in Figs. 1 to 3 respectively. In point scale algorithm, all acceleration of each records will scaled to design spectrum acceleration as the response of record and design spectrum is same in the period of building. According to algorithm presented in Fig. 1, the program read acceleration for each records, period of building and damping ratio (line 12-14), then set the load vector with records data (line 15) and find pseudo acceleration Sa_{spec} for record t (line 16) and pseudo acceleration Sa_{Design} for design spectrum (line 18) for both at desire period of building. The point scale for each record is the ratio of these two pseudo accelerations (line 20). By applying the point scale ratio to each record it will be scaled to desired design spectrum and the acceleration response of record and design spectrum will be the same in period of building.

Fig. 2 shows the pseudo algorithm of main body of program. The program read point scale factor for record t, which is, calculated in point scale subroutine. In addition, program read acceleration for each record, yield force, period, damping ratio and post yielding slop of stiffness of system for SDOF system (line 19-22). After initialize data, program set the external load for the current and next time steps. The internal force of system will be found according to the path history of displacement (line 27-28). Because in bilinear force displacement relation, the amount of post yielding internal force, is no longer single value for each displacement and it is depend on whether displacement is currently increasing (velocity is positive) or decreasing (velocity is negative). The

1.	**INPUT:**
2.	Acc(t, i), Acceleration of record number t in time step i.
3.	T_building, period of building in desire mode shape.
4.	ζ , Damping ratio of SDOF system in time step i.
5.	**PARAMETER:**
6.	ΔP_i , increment of load.
7.	Sa_{spec} , Spectral acceleration from record t.
8.	Sa_{Design} , Design spectral acceleration from Code.
9.	**OUTPUT:**
10.	$Po\text{int}Scale(t,T\_building)$, Point Scale Factor for record t
11.	**BEGIN:**
12.	**Read** T_building
13.	**For each** t=T **Do**
14.	**Read records** //Acceleration for record t.
15.	$\Delta P_i = -[Acc(t,i+1) - Acc(t,i)]g$ // Set load vector from record t
16.	**Call FINDSPECT** (T_BUILDING, t)// Find Pseudo acceleration
17.	Sa_{spec} at T_Building for record t.
18.	**Call FINDSPECTCODE** (T_BUILDING, t)// Find Pseudo acceleration
19.	Sa_{Design} at T_Building for design spectrum.
20.	$Po\text{int}Scale(t,T\_building) = Sa_{spec} / Sa_{Design}$ //Point Scale for record t
21.	**Repeat**
22.	**END**

Fig. 1 Point scale algorithm for all records

INPUT:
$Acc(i)$, Acceleration for time step i. fsy, T_SDOF, ξ, yield force and Period and damping ratio of SDOF system. α, post yielding slop of stiffness of system.
PARAMETER:
$P(i), g$, Load vector and gravity acceleration.
β, γ, parameter for average and linear acceleration methods.
NPTS, DT, Number of acceleration and time step in each record file
$K_i^T, \hat{K}_i^T$, Tangent stiffness SDOF and equivalent SDOF system in time step i. c, m , damping and mass of SDOF. $fs(i)$, Internal force and state of Yielding of SDOF in time step i.
OUTPUT:
$D(i), V(i), A(i)$, $D_{\max}, V_{\max}, A_{\max}$ displacement, velocity and acceleration and their maximum response. $Yield(i)$, state of yielding of S.D.O.F at time step i.
BEGIN:
Read $Po\text{int}Scale(t, T\_building)$, Read point scale factor for record t
Read fsy, T_SDOF, α, ξ
Read Record(t) // read acceleration data from record number t.
ScaleRecord // Scale the record by applying scale factor to record
Set initial conditions
For each i = NPTS **do**
$P(i) = -mAcc(i)g$ // Set load vector from record
$P(i+1) = -mAcc(i+1)g$
FindFS, $fs(i)$ //Find the internal force of system based on path history of displacement (displacement and sign of velocity).
$\hat{K}_i^T = K_i^T + c\gamma / \beta DT + m / (\beta DT^2)$
Converge //Modified Newton-Raphson iteration
$D(i+1) = U_{i+1}$
$fs(i+1) \leftarrow$ Modify $fs(i+1)$ // Modify internal force by state of Yielding
$Yield(i)$ // save the state of Yielding
$\Delta V(i) = (\gamma / \beta DT)\Delta D(i) - (\gamma / \beta)V(i) + DT(1 - \gamma / 2\beta)A(i)$
$\Delta A(i) = (1/\beta DT^2)\Delta D(i) - (1/\beta DT)V(i) - (1/2\beta)A(i)$
$V(i+1) = V(i) + \Delta V(i)$
$A(i+1) = (P(i+1) - cV(i) - fs(i+1))/m$
Repeat
Findmax $D_{\max}, V_{\max}, A_{\max}$ // Find maximum of Displacement, Velocity and Acceleration vector
Drawcurve // Draw Graphical Output
END

Fig. 2 Nonlinear time history analysis algorithms for SDOF system

sign of velocity shows the direction of loading.

As the next step displacement of system is unknown yet therefore we must use tangent stiffness instead of second stiffness in equivalent force displacement relation curve. The equivalent tangent

INPUT:
K_i^T ,Tangent stiffness SDOF system in time step i
$\hat{K}_i^T$, Tangent stiffness of equivalent SDOF system in time step i.
$\Delta\hat{P}_i$, Load increment of equivalent SDOF system in time step i.
PARAMETER:
$\Delta u_i(j)$, displacement increment at iteration j.
$\Delta u_i^T(j)$, Total displacement increment at iteration j.
$fs_i(j)$, Internal force at iteration j.
$\Delta R(j)$, Residual force at iteration j.
OUTPUT:
U_{i+1}, $fs(i+1)$, Displacement and Internal force in time step i+1.
BEGIN:
Set initial conditions
For j =max_iteration **Do**
$\Delta u_i(j) = \Delta R(j) / \hat{K}_i^T$
$\Delta u_i^T(j) = \Delta u_i^T(j) + \Delta u_i(j)$
$u_{i+1}(j) = u_{i+1}(j-1) + \Delta u_i(j)$
$fs_i(j) = fs_i(j-1) + \Delta u_i(j) K_i^T$
$\Delta fs_i(j) = fs_i(j) - fs_i(j-1) + \Delta u_i(j)(\hat{K}_i^T - K_i^T)$
$\Delta R(j+1) = \Delta R(j) - \Delta fs_i(j)$
If $\Delta R(j+1) < Tolerance$ then Lx
Repeat
Not convergence // increase max_iteration number.
Lx:
$U_{i+1} = u_{i+1}(j)$, $fs(i+1) = fs_i(j)$
END

Fig. 3 The Coverage algorithm

stiffness of SDOF is calculated with tangent stiffness, damping and inertia of system (line 29). The internal force and displacement in next time step is calculated with utilizing of modified Newton-Raphson iteration in subroutine Converge (Fig. 3).

After modify internal force by state of yielding, the increment of velocity and acceleration as well as velocity and acceleration of next time step will be obtained (line 35-39/Fig. 2). This process will be repeated for all time steps and the maximum displacement, velocity and acceleration will be found (line 41-42/Fig. 2). Finally the graphical output is drawn by another subroutine for displacement, velocity, acceleration, internal force and yielding function history.

5. Structural models and assumptions

In this study, a set of 5, 10 and 15 story concrete frames have been analyzed and designed with 3 spans of 4 meters with the height of 3.2 m for each story, based on equivalent static analysis according to Iranian code of practice (Building and housing research center CODE2800 2007) and ACI318-2005 (American Concrete Institute 2005) for concrete design (Fig. 4). The compression strength of concrete and yield stress of bars is assumed $f'_c = 210$ kg/cm^2, $f_y = 4000$ kg/cm^2 respectively, for all frames. The dead load is considered 3.6 t/m for stories and 1.8 t/m for roof and live load, 0.8 t/m for all stories. All type of frames are intermediate reinforced concrete moment resisting frame with behavior factor $R = 7$ and importance factor $I = 1$. It is assumed that all buildings are located in a high level of seismic zone with a design base acceleration 0.3 g and soil profile type III (180-360 m/s, $T_0 = 0.15$, $Ts = 0.7$ sec, $S = 1.75$) equal to site class D of NEHRP (FEMA368 2000). In calculation of fundamental period of frames the effective stiffness of the cracked sections is considered by taking $0.5I_g$ for beams and I_g for columns. The I_g is the gross moment of inertia for the member's cross-section neglecting its reinforcement. For Analysis and designing of building and for controlling of drift limitation of frames the effective stiffness of the cracked sections is considered by taking $0.35I_g$ for beams and $0.7I_g$ for columns.

The buildings have been analyzed and design based of CODE2800 and ACI-318-2005. In this stage, the design base acceleration assumed 0.3 g and the fundamental period of vibration of all building is calculated based on dynamic analysis instead of using empirical formula ($T = 0.07H^{0.75}$). In this study, a concentrated uncoupled moment hinges ($M3$) and a concentrated coupled P-$M2$-$M3$ hinges are used for modeling of plastic zone of beams and columns respectively. To perform nonlinear static and dynamic analysis for MDOF buildings, the SAP200 NL version was employed and for nonlinear time history analysis for equivalent SDOF system a program was developed by authors. Fundamental period, weight, base share and other equivalent static analysis parameters are shown in Table 2.

$$V = CW = \frac{ABI}{R}W \tag{9}$$

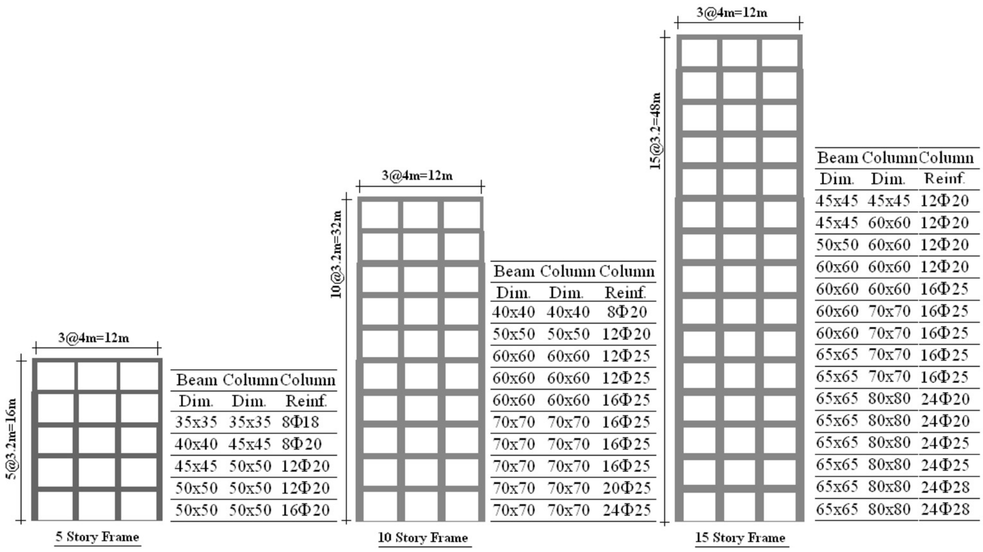

Fig. 4

$$B = 1 + S(T/T_0) \qquad 0 \le T \le T_0 \tag{11}$$

$$B = 1 + S \qquad T_0 \le T \le T_S \tag{12}$$

$$B = (1 + S)(T_S/T)^{2/3} \qquad T_S \le T \tag{13}$$

$$F_i = (V - F_t)\frac{W_i h_i}{\sum_{j=1}^{n} W_i h_i}, \qquad F_t = 0.07TV \tag{14}$$

After linear analysis, drifts of stories have been controlled with the limitations of Code2800 as follows

$$\Delta_M < 0.025\text{h } \textit{for } T < 0.7\text{Sec} \tag{15}$$

$$\Delta_M \le 0.020\text{h } \textit{for } T \ge 0.7\text{Sec} \tag{16}$$

$$\Delta_M = 0.7R\Delta_w \tag{17}$$

$$T = 0.07\text{h}^{3/4} \tag{18}$$

Δ_M =*The Actual design story drift*
Δ_w =*The design story drift*
R = *Building behavior factor*

The Δ_M and Δ_w are the actual design story drift and the design story drift respectively. In fact Δ_M and Δ_w are drifts of inelastic and elastic of building. Therefore drift limitations for all frames are 0.00408. After the drift control, each building has been designed for reinforcement. The results of the building's design for 5, 10 and 15 stories have been depicted in Table 1.

6. Ground motion ensemble

Seven ground motions were intended to be far 5 to 25 km*,* for a set of fault rupture with strike-slip mechanism at magnitude range 6.1 to 7.8. The soil at the site correspond to NEHRP site class *D* for *Vs* (Shear-wave velocity) 180-360 *m/s* which is equal to soil type *III* according to code2800.

Table 1 Parameters of equivalent static analysis

	Fundamental Period	Self Weight	Weight due load	Total Weight	B	C = ABI/R	V	Ft	$C = (V\text{-}F_t)/W$
5-story	0.9098	61.4	204	265.4	2.313	0.0991	26.31	1.67	0.0928
10-story	1.098	211.25	429.6	640.85	2.043	0.0875	56.11	4.31	0.0804
15-story	1.54	386.31	655.2	1041.5	1.643	0.0704	73.34	7.9	0.0628

Table 2 List of used ground motions

No.	Earthquake[a]	Date	Magnitude	Record	Dist.[b] (km)	PGA (g)	PGV (cm/s)	PGD (cm)
1	Duzce Turkey	1999/11/12	$Ms = 7.3$	Duzce/ Bol090	17.6	0.822	62.1	13.55
2	Imperial Valley	1979/10/15	$Ms = 6.9$	Elcentro Station 5165	5.3	0.707	20.7	11.55
3	Kocaeli Turkey	1999/08/17	$Ms = 7.8$	Duzce/ 270ERD	12.7	0.358	46.4	17.61
4	Landers	1992/06/28 11:58	$Ms = 7.4$	1158,yermo 270,22074	24.9	0.245	51.5	43.81
5	Manjil	1990/06/20	$Ms = 7.4$	Abbar,2100	12.6	0.538	-	-
6	Park Field	1966/06/28 4:26	$M = 6.1$	Cholame, 05085, Station1014	5.3	0.442	24.7	5.15
7	Superstitu Hills	1987/11/24 13:16	$Ms = 6.6$	Elcentro Station 01335	13.9	0.358	46.4	17.5

Data Source: PEER (http://peer.berkeley.edu.smcat) for No. 1 to 4, 6,7 and Cosmos (http://db.cosmos-eq.org)) for No. 5
[b] Closest distance to fault

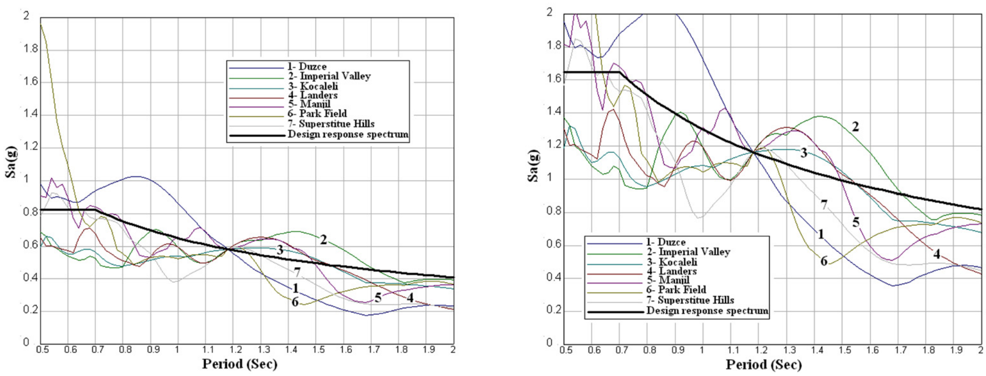

Fig. 5 Standard response spectrum and 5%-damped response spectra of scaled motions, used for 10 story frame with $T = 1.182$Sec and design base acceleration 0.3 g in left and 0.6 g in right

The specifications of the used records are given in Table 3. Each ground motion was scaled so that the five-percent-damped spectral ordinate at the period of the spectrum of ground motion matched that of the CODE2800 design response spectrum (soil profile type III, 180-360 m/s, $T_0 = 0.15$, $Ts = 0.7$sec, $S = 1.75$) at the same period. All records first were scaled up to 0.3 g based on CODE2800 and then to ensure that the structure responses well into the plastic range were scaled up to 0.6 g (Fig. 5).

7. Analyzing of frames by MPA procedure

According to the previous section, first a linear dynamic analysis performed for all frames to find dynamic characteristics like periods and modal mass participation (Table 3). In Table 3 $\alpha_n = L_n\Gamma_n / M$ or $\alpha_n = M^*/M$ and $M_n = \Sigma M_i\varphi^2_i$ or $M_n = L_n/\Gamma_n$. In the next step, a nonlinear static analysis conducted for all frames with different base acceleration in order to develop the base shear-roof displacement, V_{bn}-u_m pushover curve and convert it to F_{sn}/L_n-D_n curve. Then for each case a nonlinear time history analysis performed to realize peak deformation of D_n of the n^{th}-mode inelastic SDOF system (Fig. 6). First column of Fig. 6 shows the response of inelastic SDOF to different excitation under nonlinear time history analysis. In all cases the response of system in first mode is inelastic. It is worth noticing that the axis of oscillation will be moved and the system will oscillate around the new position after yielding.

Fig. 7 shows Modal participation of each mode in response of SDOF system. It shows that the first mode has most effects in the response of system. Moreover it can be declared that almost second and third mode remains elastic with expectation of third mode to record no. 5 for base acceleration 0.3 g (acceleration of practice code). It could be the idea of modified modal pushover analysis (MMPA), which combines the elastic influence of higher modes with the inelastic response of first mode pushover analysis using modal combination rules (Chopra *et al*. 2004). Fig. 8 shows the comparison of first mode response of SDOF by MPA procedure with nonlinear time history roof displacement response of 10 story building with base acceleration 0.3 g and 0.6 g. it shows acceptable estimation of first mode response of SDOF for actual response of system by NLTH. It is interesting that the time of performing a SDOF nonlinear time history analysis in comparison of whole system is very small.

8. Evaluation of FEMA-356 and MPA

According to the results of analysis, the FEMA-356 and MPA nonlinear procedures are assessed by comparing maximum story displacements, inter story drift and beam plastic rotations to nonlinear

Table 3. Characteristic of MPA procedures

	Mode	T_n Sec	M Ton	α_n	L_n Ton	Γ_n	M_n	M^*_n	V_Y Ton	U_n Cm	F_{sn}/L_n	D_n Cm	T_n Sec
5-Story	1	0.839	265.39	0.777	141.98	1.453	97.70	206.3	60.77	7.82	0.29	5.38	0.858
	2	0.324	265.39	0.113	-45.63	-0.66	68.94	30.11	36.89	2.12	1.22	3.21	0.325
	3	0.19	265.39	0.05	41.35	0.32	128.96	13.23	34.56	0.76	2.61	2.38	0.192
10-Story	1	1.182	640.85	0.771	340.73	1.45	235.58	492.81	64.66	8.32	0.31	5.73	0.858
	2	0.444	640.85	0.097	-87.56	-0.71	122.88	62.40	44.33	2.51	1.47	3.79	0.322
	3	0.267	640.85	0.047	60.98	0.50	122.92	30.26	34.56	0.76	2.61	2.38	0.192
15-Story	1	1.646	1041.51	0.742	533.28	1.45	367.00	774.92	138.27	14.35	0.28	9.92	1.193
	2	0.608	1041.51	0.119	-175.10	-0.71	247.79	123.73	68.72	3.73	1.10	-5.24	0.438
	3	0.356	1041.51	0.044	93.85	0.49	191.35	46.03	60.20	1.80	1.99	3.63	0.271

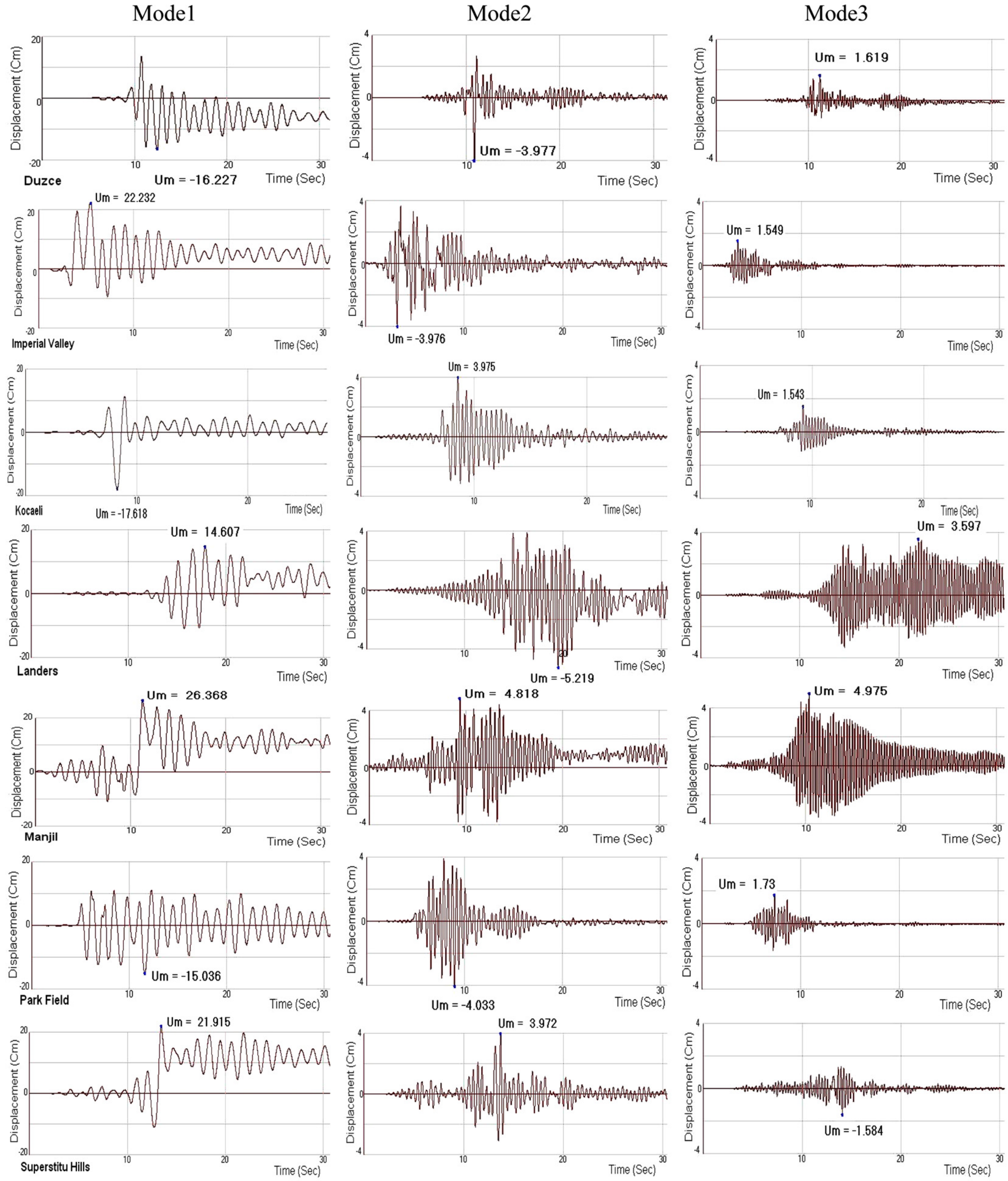

Fig. 6 Response of inelastic SDOF in different modes to ground motion under NLTH analysis

time history dynamic analysis (NLTH). It is assumed that the results of NLTH are the exact solution and are our benchmarks.

8.1 Maximum story displacements

Fig. 9 shows maximum displacement to height ratio evaluated by FEMA356, MPA and NLTH for

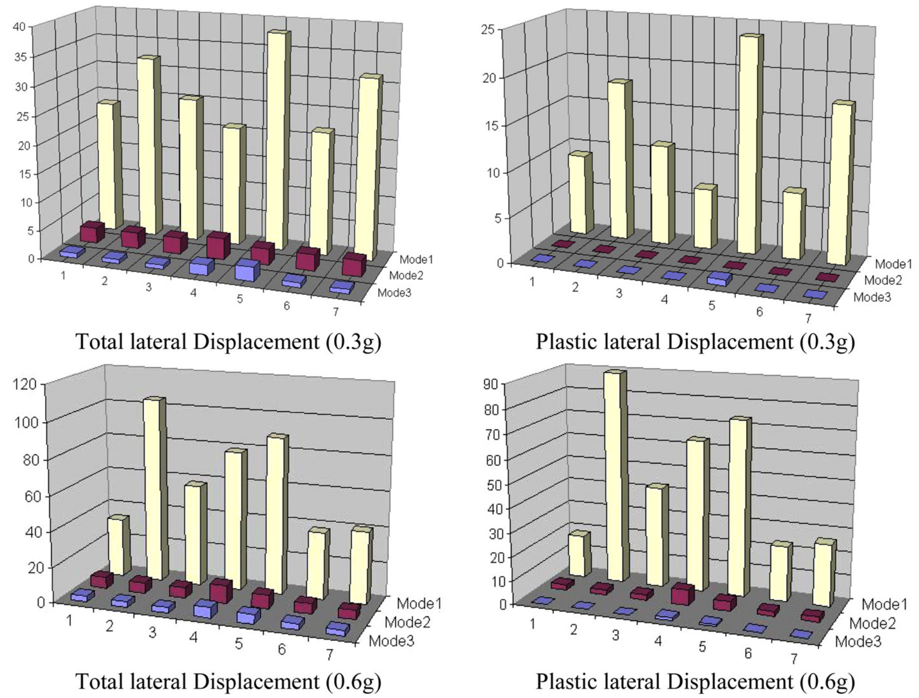

Fig. 7 Modal participation in total and plastic lateral displacement of SDOF system for base acceleration 0.3 g, 0.6 g

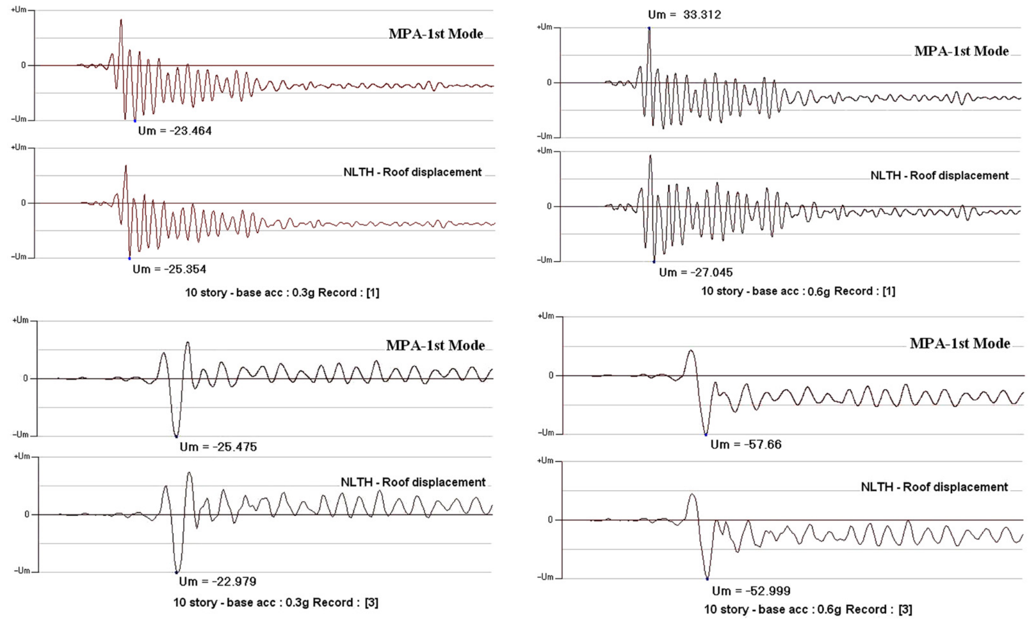

Fig. 8 Compare first mode response of SDOF by MPA procedure with nonlinear time history roof displacement response of 10 story building record number 1 with base acceleration 0.3 g and 0.6 g

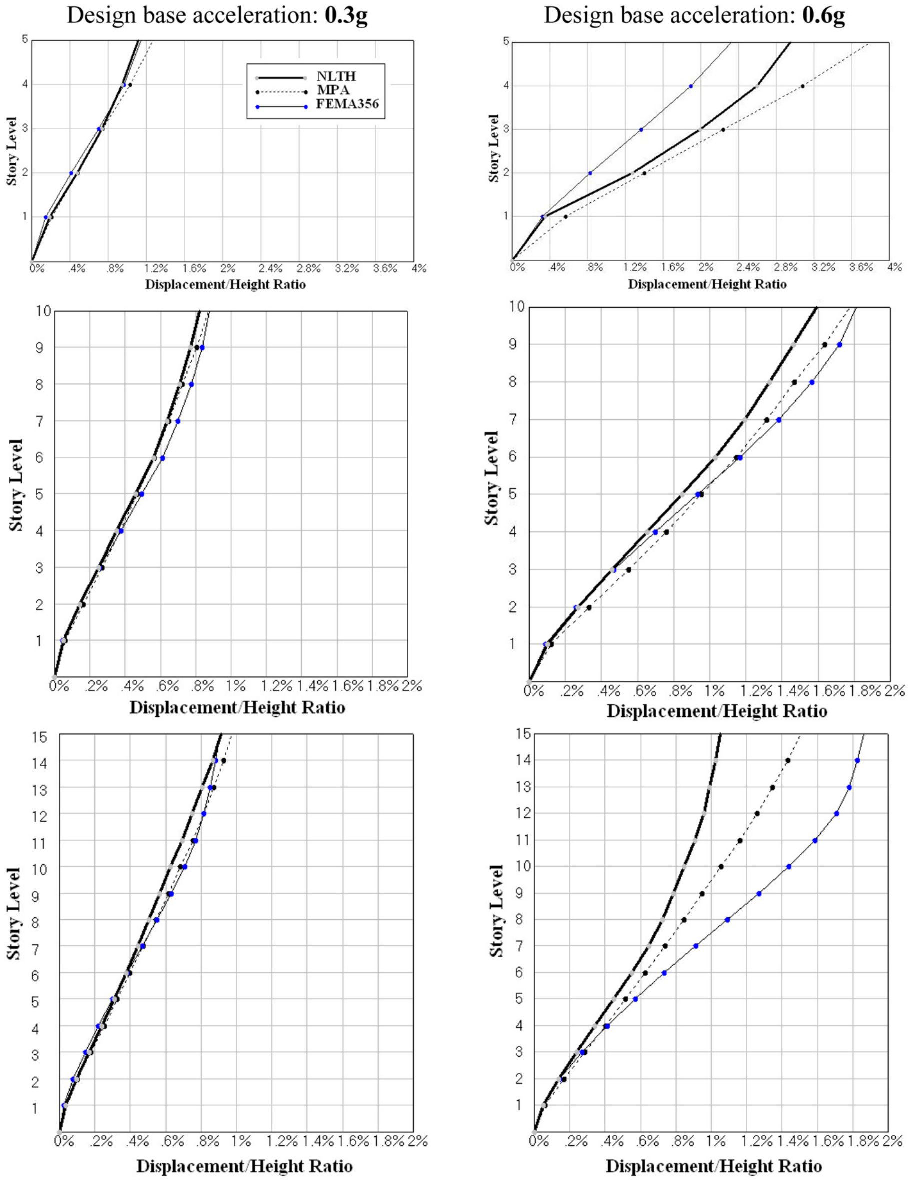

Fig. 9 Maximum lateral displacement to height ratio evaluated by FEMA356, MPA and NLTH for each building with two different base accelerations of 0.3 g and 0.6 g

each building with two different base accelerations of 0.3 g and 0.6 g. The figure shows that FEMA356 pushover procedure overestimates lateral displacement for 10 and 15 stories frame for different base accelerations and underestimates for 5 story frame. On the other hand, MPA procedure overestimates lateral displacement for all buildings and for all base accelerations. In all cases, MPA procedure generally yields better estimation of the lateral displacements in comparison to FEMA356. Almost in all cases both procedures are conservative (with the exception of FEMA356 procedure in 5 story building). The amount of difference between two procedures will be

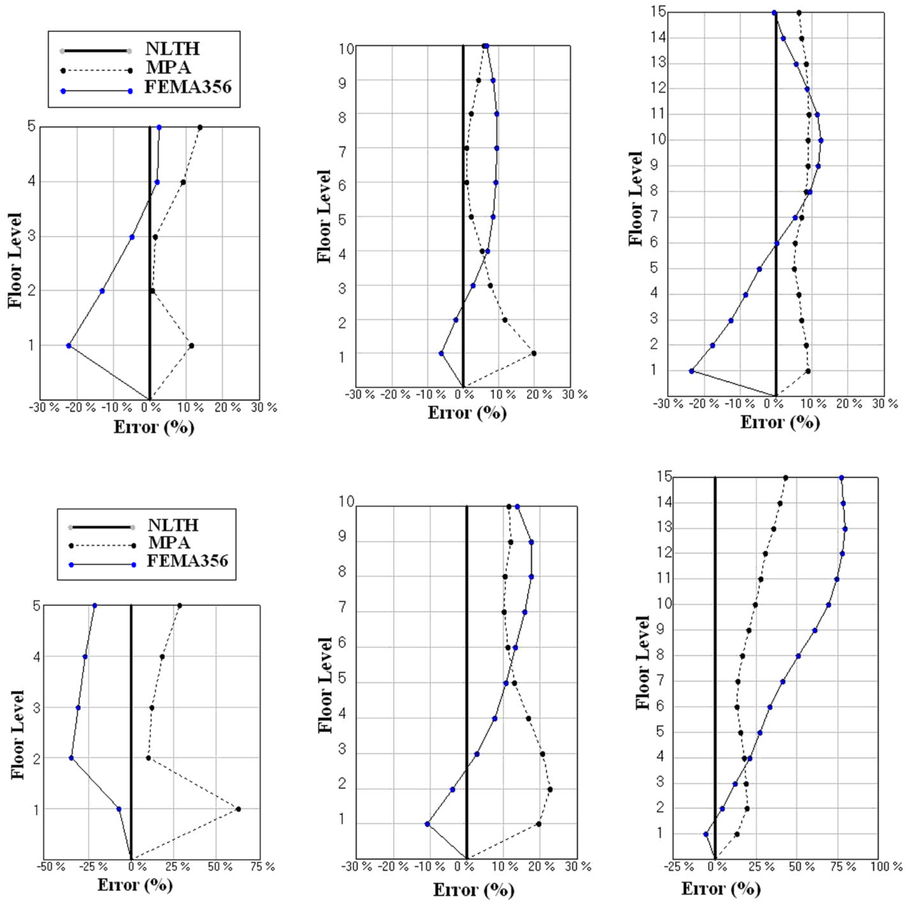

Fig. 10 Errors in maximum displacement to height ratio evaluated by FEMA356, MPA and NLTH for each building with two different base accelerations of 0.3 g (first row) and 0.6 g (second row)

increased with increasing of building height and base acceleration which is because of the participation of higher modes in the response of buildings.

Fig. 10 shows the errors in maximum displacement to height ratio evaluated by different procedures. The figure shows that the errors of FEMA356 pushover and MPA procedures will be increased with increasing of base acceleration from 0.3 g to 0.6 g in all cases. The errors of MPA procedure are less than FEMA356 in estimation of lateral displacement of systems.

8.2 Inter story Drift Ratio

Fig. 11 shows inter story drift ratio evaluated by FEMA356, MPA and NLTH for each building with two different base accelerations 0.3 g and 0.6 g. The figure shows that FEMA356 pushover procedure underestimates inter story drift ratio in lower and upper stories and overestimates for middle stories in all cases with base acceleration 0.3 g. With increasing the base acceleration to 0.6 g, the FEAM356 procedure generally overestimates inter story drift ratio for all cases (with the exception of FEMA356 procedure in 5 story building). The MPA procedure generally overestimates

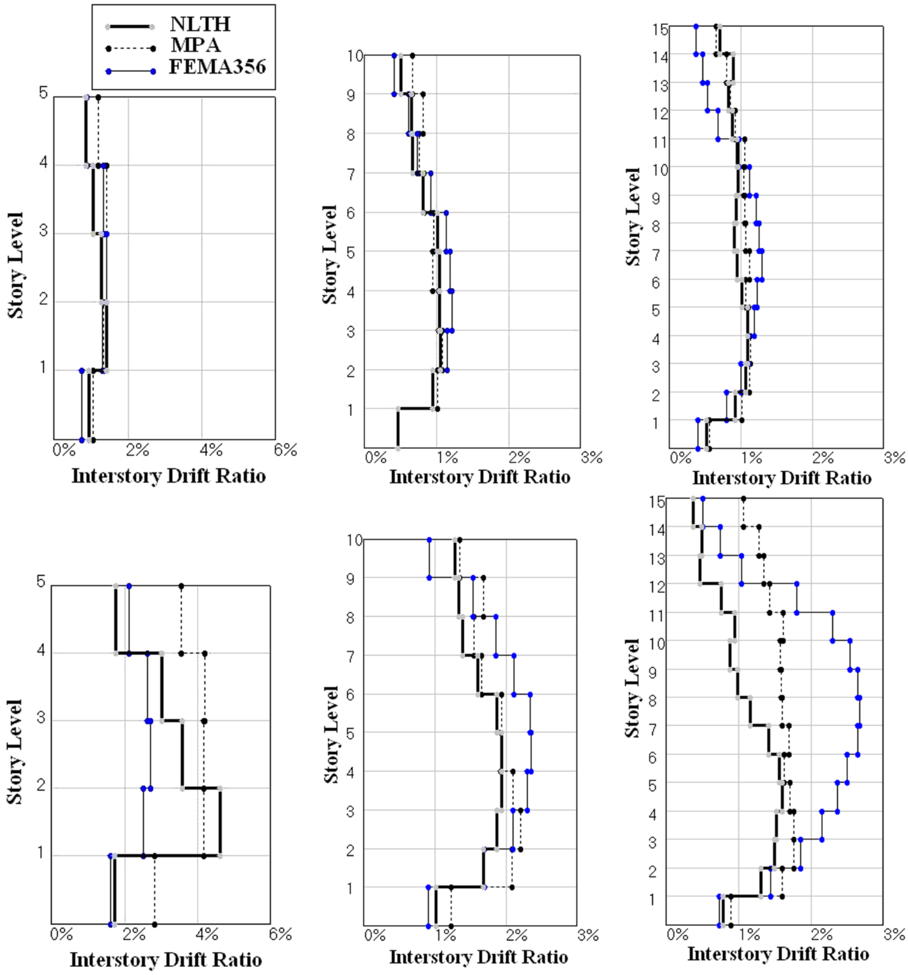

Fig. 11 Inter story Drift Ratio for 5,10 and 15 story building with two different base accelerations of 0.3 g (first row) and 0.6 g (second row)

drift ratio for all buildings especially for base accelerations 0.6 g. The MPA yields better estimations of drift demands in comparison to FEMA356.

Fig. 12 shows inter story drift error percentage by different procedures. It could be observed that with increasing the numbers of stories from 5 to 15, the errors of inter story estimation by FEMA356 will increase in all cases. The percentages of errors also increase with increasing base acceleration from 0.3 g to 0.6 g. The percentages of errors by MPA procedure is less than FEMA356, especially for the cases of 10 and 15 stories in which the effects of higher modes are more significant.

8.3 Plastic hinge rotations

Fig. 13 shows plastic hinge rotation estimated by FEMA356, MPA and NLTH for each building with two different base accelerations 0.3 g and 0.6 g. The figure shows that generally none of both

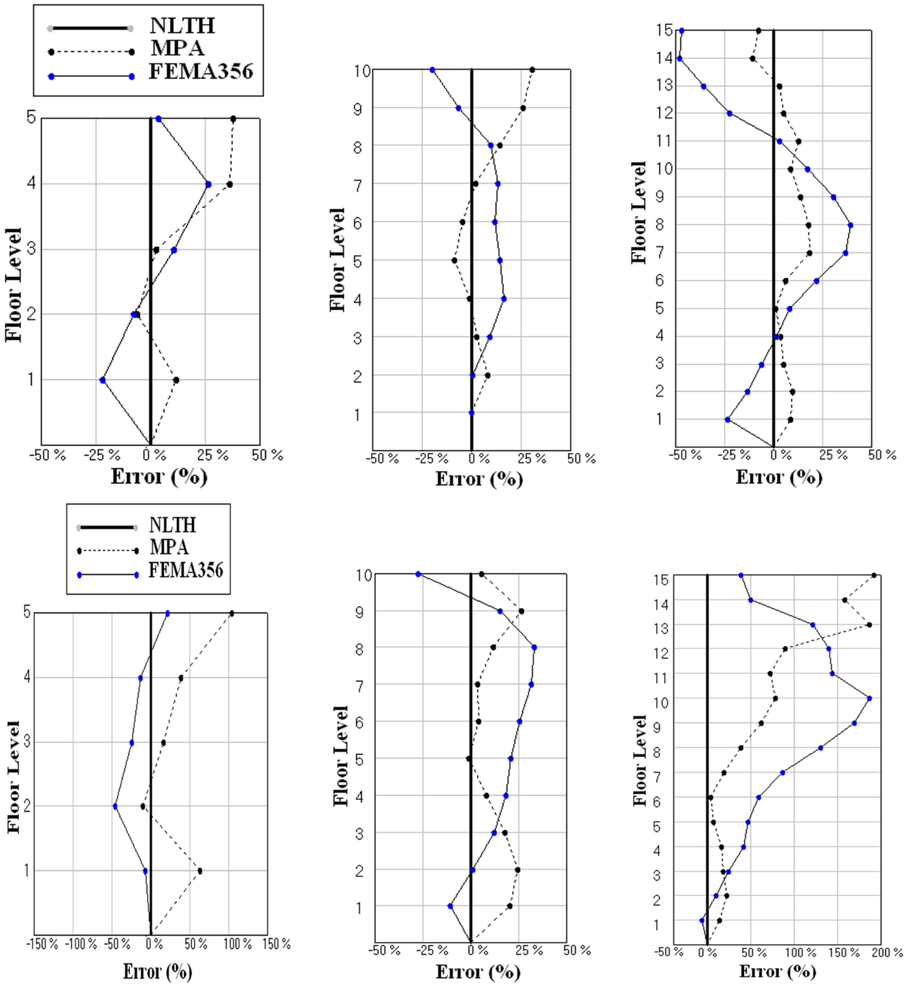

Fig. 12 Inter story Drift Error percentage for 5, 10 and 15 story building with design base acceleration of 0.3 g (first row) and 0.6 g (second row)

pushover procedures are accurate enough for capturing good results for evaluating plastic rotation. FEMA356 pushover procedure yields better estimations for lower stories and MPA procedure for upper stories in case of 0.3 g. The MPA procedure commonly gives better estimate for plastic rotation in case of 0.6 g. Fig. 14 shows the errors of plastic hinge rotation estimated by FEMA356, MPA and NLTH. The amounts of errors of FEMA356 will increase for upper stories in all cases. Although the MPA procedure have large errors in estimating hinges plastic rotations but its errors is less than FEMA356 procedure.

9. Conclusions

This paper has evaluated nonlinear static procedure offered by FEMA356 and modal pushover

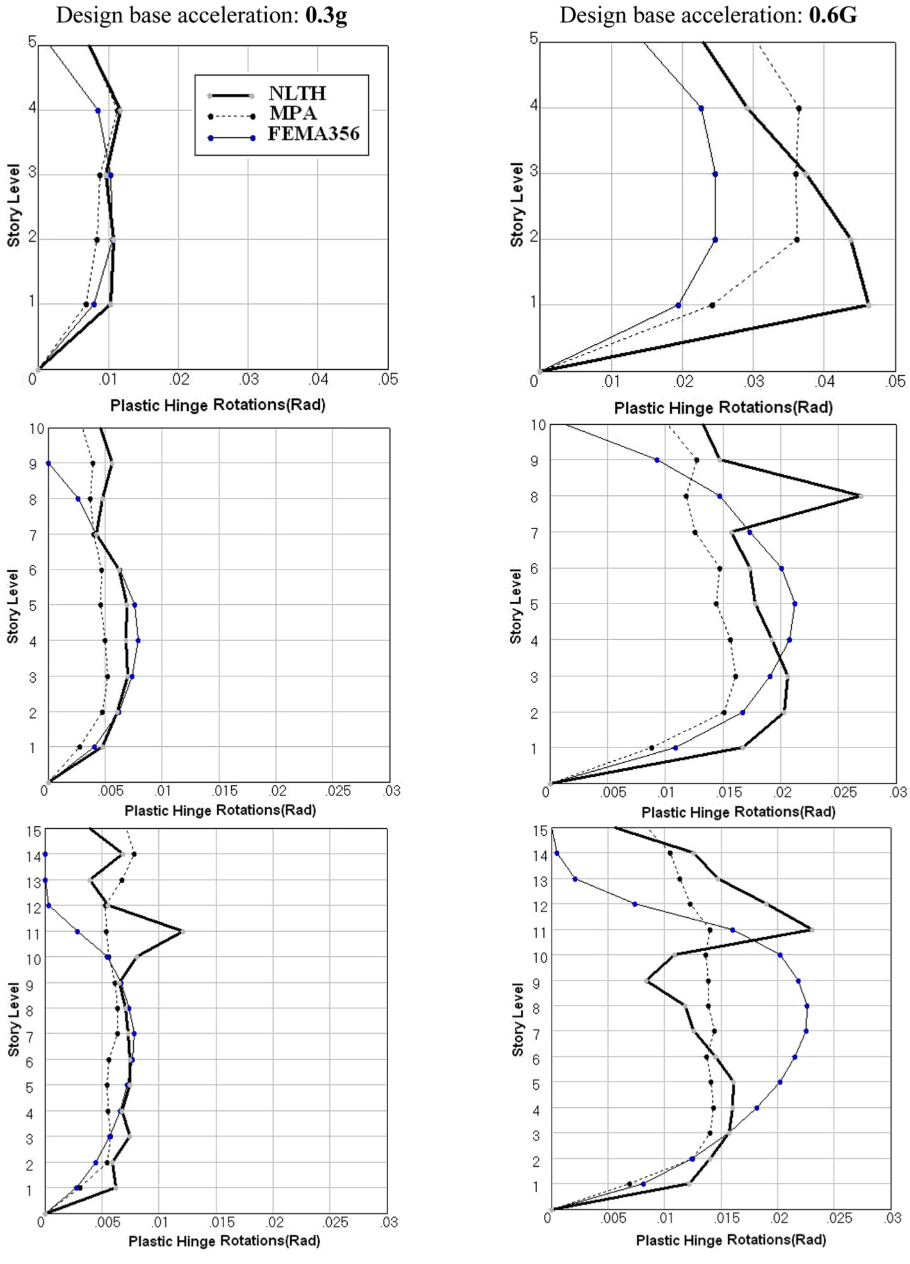

Fig. 13 Plastic hinge rotation for 5,10 and 15 story building with design base acceleration of 0.3 g (first row) and 0.6 g (second row)

analysis to predict seismic demands in a set of designed concrete buildings by code of practice (CODE2800). Each building is subjected to seven ground motions with special characteristics to the site specifications. The mean of results served as benchmark responses in comparison to FEMA356 and MPA procedures results. The consideration of lateral displacements, inter story drifts, hinges plastic rotations of beams and their errors are the bases for the following conclusions:

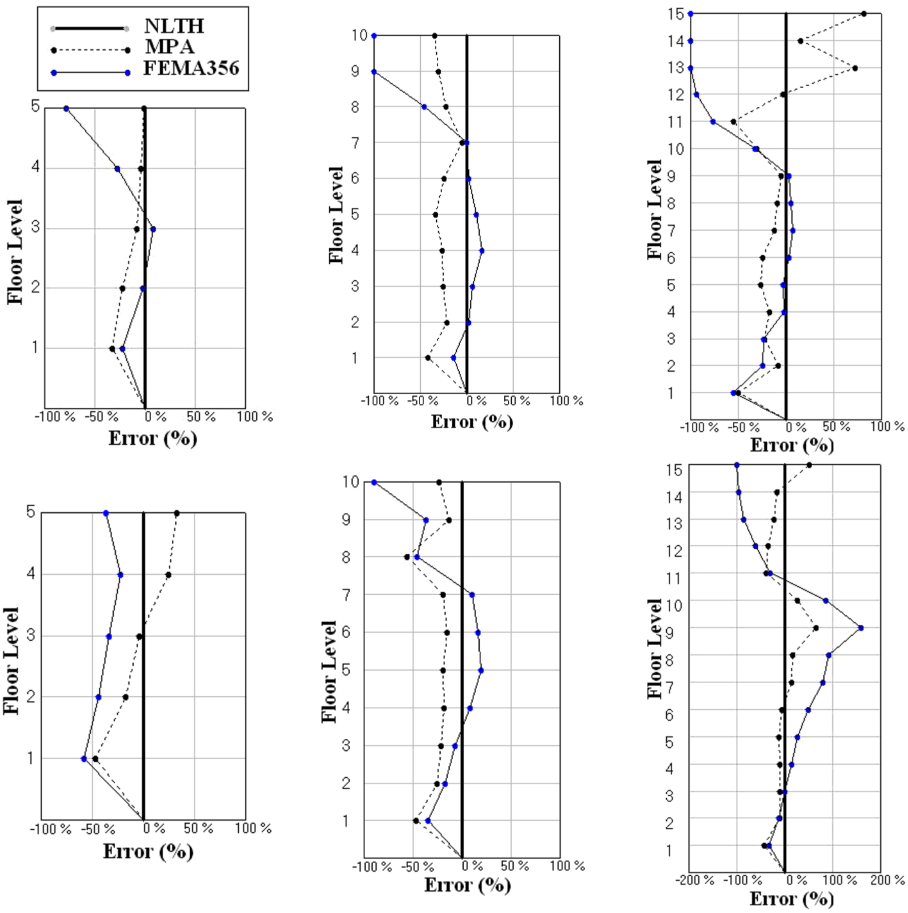

Fig. 14 Errors of plastic hinge rotation estimated by FEMA356, MPA and NLTH for 5,10 and 15 story building with design base acceleration of 0.3 g (first row) and 0.6 g (second row)

1-The FEMA356 procedure can not predict accurate results in inter story drifts and hinges plastic rotations especially for middle and upper stories of higher buildings and for higher base accelerations when the effects of higher mode participation are more important.

2-In Comparison to FEMA356, the MPA procedure predicts more accurate results for inter story drifts and hinges plastic rotations because the MPA procedure is able to take into account the inelastic response of higher modes.

3-In Comparison to FEMA356, the MPA procedure requires more number steps to complete its procedures, although it takes a considerable operation time. As a result the MPA is simple enough to be used in design offices for practical purposes.

4-Since the considered nonlinear static procedures in this paper use invariant lateral load vectors, therefore they can not properly take into account the variation of dynamic specifications like changing stiffness, period and mode shapes of system during in inelastic responses. On the other hand, any try for achieving adaptive load vectors may cause missing the simplicities of above procedures which could be of significance at practice level.

References

American Concrete Institute (ACI) (2005), “Building code requirements for structural concrete (Committee 318, ACI 318-05)”, American Concrete Institute (ACI), Farmington Hills, MI.

American Society of Civil Engineering (ASCE) (2000), “Prestandard and commentary for seismic rehabilitation of buildings”, FEMA356, Washington D.C.

Applied Technology Council (ATC) (1996), “Seismic evaluation and retrofit of concrete buildings”, Rep. No. ATC40, Volume 1 and 2, Redwood City, California.

Applied Technology Council (ATC) (2005), “Improvement of nonlinear static seismic analysis procedures”, Rep. No. FEMA440, Redwood City, California.

Building and Housing Research Center (2007), Iranian Code of practice for seismic resistant design of buildings (CODE2800), (Standard No. 2800), 3rd Edition.

CEN (2004), European Standard EN 1998-1-2004. Eurocode 8: Design of structures for earthquake resistance - Part 1:General rules, seismic actions and rules for buildings, Bruxelles, Belgium.

CEN (2005), European Standard EN 1998-3-2005. Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings, Bruxelles, Belgium.

Chopra, A.K. and Goel, R.K. (2002), “A modal pushover analysis procedure for estimating seismic demands for buildings”, *Earthq. Eng. Struct. Dyn.*, **31**, 561-82.

Chopra, A.K., Goel, R.K. and Chintanapakdee, C. (2004), “Evaluation of a modified MPA procedure assuming higher modes as elastic to estimate seismic demands”, *Earthq. Struct*., **20**(3), 757-78.

Chopra, A.K. (2007), *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, Prentice-Hall, Englewood Cliffs, NJ.

CSI Analysis Reference Manual (2009), SAP2000, Ver. 14, integrated finite element analysis and design of structures, Computers and Structures ING, Berkeley.

Fajfar, P. (2000), “A nonlinear analysis method for performance based seismic design”, *Earthq. Spectra*, **16**, 573-592.

FEMA368 (2000), NERPH Recommended Provisions for Seismic Regulations for New Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, D.C.

Gupta, B. and Kunnath, S.K. (2000), “Adaptive spectra-based pushover procedure for seismic evaluation of structures”, *Earthq. Spectra*, **16**(2), 367-91.

Kalkan, E. and Kunnath, S.K. (2006), “Adaptive modal combination procedure for nonlinear static analysis of building structures”, *ASCE J. Struct. Eng*., **132**(11), 0733-9445.

Kalkan, E. and Kunnath, S.K. (2006), “Assessment of current nonlinear static procedures for seismic evaluation of buildings”, *Eng. Struct*., **29**(3), 305-316.

Krawilklar, H. and Seneviratna, G.D.P.K. (1998), “Pros and cons of a pushover analysis of seismic performance evaluation”, *J. Eng. Struct*., **20**(4-6), 452-462.

Marsono, A.K. and Khoshnoud, H.R. (2010), “Evaluating equivalent static analysis of Iranian code with nonlinear static pushover analysis”, *Proceedings of the First Makassar International Conference on Civil Engineering (MICCE2010)*, Makassar, Indonesia.

Poursha, M., Khoshnoudian, F. and Moghadam, A. (2008), “Assessment of conventional ststic procedures with FEMA load distribution and modal analysis for high-rise buildings”, *J. Civil Eng*., **6**(2), 142-157.