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Assessment of Performance of Buildings with High Importance Factor through Nonlinear Static and Dynamic Procedures

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***Abstract*— The main objective of this paper is to appraise the performance level of building with very high importance factor like hospitals and emergency centers which have been analyzed and designed based on linear static procedure (LSP) with nonlinear static pushover analysis (NSPA) and nonlinear response history analysis (NRHA). To achieve this goal a steel moment frame building, first based on equivalent static procedure have been analyzed and designed. After applying drift limitations of code2800 and finding sections of members, the NSPA have been conducted based on FEMA356 and modal pushover analysis (MPA) while the NRHA are treated as benchmark results. Regarding the results it can be concluded that the performance level of very high importance buildings is about immediate occupancy. Furthermore the MPA is more accurate in comparison to other nonlinear static pushover analysis procedures. Finally, the NSPA procedures considering its abilities to take into account nonlinear behavior of building are an efficient suggestion of LSP at practical level especially for important buildings.**

***Index Terms*- Nonlinear static analysis, Modal Pushover Analysis, Performance based design**

I. INTRODUCTION

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 most accurate 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. Thus there is a major trend toward using the nonlinear static procedures (NSP). The NSP as an essential part of performance based design is now widely used especially at practical propose because of its simplicity and ability to predict seismic demands on inelastic response of buildings. One of the most popular static nonlinear procedures is pushover analysis which included in several seismic codes like Eurocode8 [1,2], ATC40 [3], FEMA356 [4]. The Pushover analysis is a series of incremental linear analyzes that in each step, a portion of lateral load is applied to the structure [5]. For monitoring the material nonlinear behavior of elements especially for yielding and post-yielding behavior, plastic hinges or 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 the 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 [6] 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 [7], the pushover analysis of MDOF system is combined with the response spectrum of equivalent SDOF system. In the modal pushover analysis (MPA) [8] 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 modes (normally two or three) provides an evaluation of the total seismic demand on inelastic systems. In modified modal pushover analysis (MMPA) [9] 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. In the adaptive pushover analysis (APA), load vectors are progressively updated to consider the change in system modal attributes during inelastic phase [10]. 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 [11]. 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 [12], [13]. The main objectives of current paper is

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appraise of linear static procedure (LSP) with nonlinear static pushover procedure using in FEMA356 (which are base code for Iranian seismic retrofitting code) and modal pushover analysis (MPA) and nonlinear response history analysis (NRHA).

II. MODAL PUSHOVER ANALYSIS (MPA)

In the modal pushover analysis (MPA), which has been developed by Chopra and Goel [8], 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 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 et. al. [8]. 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:

$$mu''+cu'+ku=-miu''_g(t) \qquad (1)$$

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 (1) is as shown below:

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

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

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

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

Equation (3) is the governing equation for the nth mode inelastic SDOF system with natural frequency ω_n and damping ζ_n and modal coordinate D_n. Equation (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 (5):

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

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

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:

$$\frac{F_{sn}}{L_n}=\omega_n^2 D_{ny} \qquad (7)$$

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

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

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

1. Compute ω_n and modes $\varnothing_n$ for linear elastic vibration of the building.
2. For the nth-mode, develop the base shear-roof displacement, V_{bn}-u_m pushover curve for force distribution $s^*_n=m\varnothing_n$
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 Equation (6), $\Gamma_n=\varnothing^T_n m1/\tilde{O}^T_n m\varnothing_n$
5. Compute peak deformation D_n of the nth-mode inelastic SDOF system define by the force-deformation relation and damping ratio ζ_n and the elastic vibration period T_n by Equation (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 nth mode inelastic SDOF system from $u_m=\Gamma_n\ \varnothing^T_m\ D_n$.
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:

$$r=\sqrt{\sum_n r_n^2}$$

III. DESCRIPTION OF MODEL

For evaluation of afore mentioned procedures, a 8- story steel moment resisting frame have been analyzed and

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designed with 4 spans of 5 meters with the height of 3.2m for each story, based on equivalent static analysis according to Iranian code of practice [14] and AISC/ASD 2001 for steel design (Fig. 1). The column sections of stories 1-3, 4-6 and 7-8 are C1, C2 and C3 respectively. The beam sections of stories 1-4, 5, 6 and 7-8 are B1, B2, B3 and B4 respectively. The dead and live load is considered *4.25t/m*, *1.25t/m* for stories and 3.5 *t/m* and 1*t/m* for roof respectively. All type of frames are special steel moment resisting frame with behavior factor $R=10$ and importance factor $I=1.4$. It is assumed that all buildings are located in a high level of seismic zone with a design base acceleration A=*0.3g* and soil profile type III (*180-360m/s*, $T_0=0.15$, $Ts=0.7sec$, $S=1.75$) therefore the behavior factor B=2.066. The self weight, weight due to loads and total weight of structure are 53, 742 and 795 ton respectively. The yield stress of steel is assumed $fy=2400kg/cm^2$ The fundamental period of vibration of all buildings is calculated based on dynamic analysis instead of using empirical formula ($T=0.08H^{0.75}$).

TABLE I.
PARAMETERS OF EQUIVALENT STATIC ANALYSIS

Fundamental Period (sec)	C=ABI/R	V (ton)	Ft (ton)	$C=(V-F_t)/W$
1.081	0.08675	69	5.1	0.0804

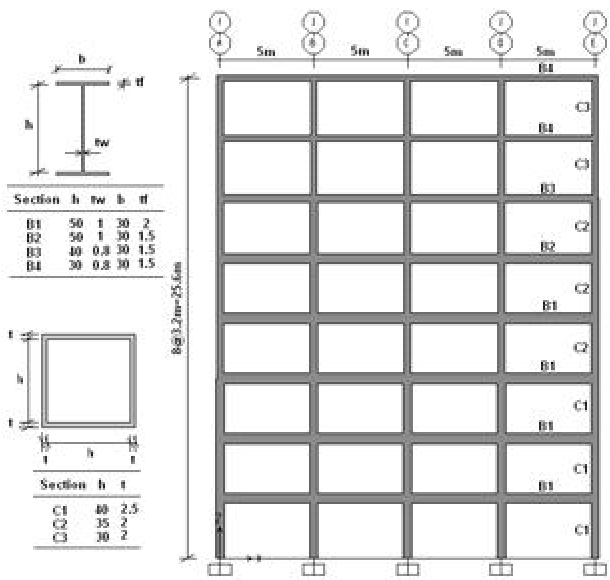

Figure 1. 8-Story steel moment resisting frame specifications

Fundamental period, weight, base shear and other equivalent static analysis parameters are shown in Table I. In this study, a concentrated uncoupled moment hinges (*M*3) and a concentrated coupled *P-M*3 hinges are used for modeling of plastic zone of beams and columns respectively. To perform nonlinear static and dynamic analysis for MDOF buildings, the SAP2000 NL version [15] was employed and for nonlinear time history analysis for equivalent SDOF system a program was developed by the authors.

IV. GROUND MOTION ENSEMBLE

Three ground motions were intended to be far 5 to 20 *km*, for a set of fault rupture with strike-slip mechanism at magnitude range 6.9 to 7.8. The specifications of the used records are given in Table II. 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 (Fig 2).

TABLE II.
LIST OF USED GROUND MOTIONS

	Duzce, Turkey	Imperial Valley	Kocaeli Turkey
Date	1999/11/12	1979/10/15	1999/08/17
Magnitude	Ms=7.3	Ms=6.9	Ms=7.8
Record	Duzce/Bol090	Elcentro/5165	Duzce/270ERD
Dist.[b](km)	17.6	5.3	12.7
PGA(g)	0.822	0.707	0.358
PGV(cm/s)	62.1	20.7	46.4
PGD(cm)	13.55	11.55	17.61

[a] Data Source: PEER (http://peer.berkeley.edu.smcat)
[b] Closest distance to fault

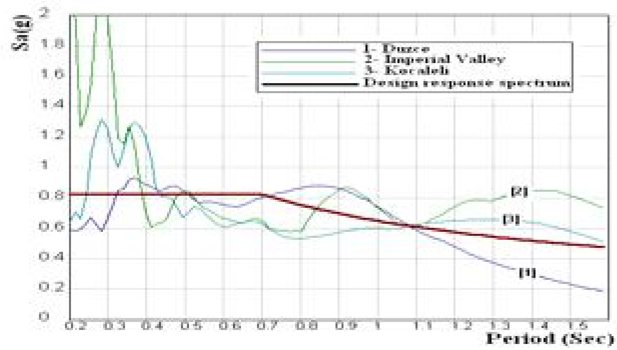

Figure 2. Standard response spectrum and 5%-damped response spectra of scaled motions

V. ANALYZING OF FRAMES BY MPA PROCEDURE

Base on the previous sections, first, a linear dynamic analysis performed for all frames to find dynamic characteristics like periods and modal mass participation (Table III). In table III $\alpha_n=L_n\Gamma_n/M$ or $\alpha_n=M^*/M$ and $M_n=\Sigma M_i\varphi_i^2$ or $M_n=L_n/\Gamma_n$. Then, a nonlinear static analysis conducted for model to develop the base shear-roof displacement, V_{bn}-u_m pushover curve and convert it to

TABLE III.
PARAMETERS OF EQUIVALENT STATIC ANALYSIS

Mode	1	2	3
T_n (Sec)	1.081	0.412	0.233
M (ton)	795	795	795
α_n	0.755	0.138	0.049
L_n (ton)	425.63	176.37	127.8
Γ_n	1.41	0.62	0.30
M_n	302	284.5	422.5
M^*_n	600	109.3	38.7
V_Y(ton)	283.6	156.9	98.8
U_{ny}(Cm)	19.5	4	1
F_{sn}/L_n	0.473	1.435	2.554
D_{ny}(Cm)	13.82	6.43	3.31
T_n(Sec)	1.085	0.424	0.228

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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 nth-mode inelastic SDOF system by the authors program. Fig. 3 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. For instance, the amounts of response for maximum lateral displacement of SDOF system are 18.823, 3.722 and 1.091Cm for three first modes respectively, for Imperial Valley record. After multiple this amounts to their transformation factors, $\Gamma_{n.}$ we have 26.54, 2.31 and 0.33Cm for MDOF system or actual frame. With using of SRSS (Square root of the sum of squares) maximum lateral displacement is 26.64Cm for Imperial Valley record. The same calculation can be done for other records and maximum response is chosen as the final results for MPA procedure. It is evident from above results that the effect of higher modes, in compare of first mode is negligible. It is because of regularity of frame in this study and participation of higher modes in seismic response is not significant. Fig. 4 shows the comparison of first mode response of SDOF by MPA procedure which is multiplied by $\Gamma_{1.=}1.41$ with nonlinear time history roof displacement response of 8 story building with base acceleration 0.3g. 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 analyzing of whole system is very small.

VI. EVALUATION OF LINEAR AND NONLINEAR PROCEDURES

According to the results of analysis, the linear equivalent static analysis and nonlinear static analysis of the FEMA356 and MPA nonlinear procedures are evaluated by comparing maximum story displacements, inter story drift and beam plastic rotations to nonlinear time history dynamic analysis (NLTH). The results of NLTH are considered as our exact solution. The target displacement is about 25.4cm, 26.6cm and 25.25 by FEMA356, MPA and NLTH respectively. Top row of Fig. 5 shows maximum displacement to height ratio evaluated by elastic analysis, FEMA356, MPA and NLTH and their errors for base accelerations of 0.3g. According to code2800 the actual design story drift is calculated from multiple 0.7R in design story drift that is result of linear analysis of building. The results show the elastic procedure is always overestimate and amount of error is more than 50%. The figure shows that FEMA356 pushover procedure underestimate lateral displacement and MPA procedure overestimates lateral displacement for all stories. In estimation of lateral displacement for this building the results of FEMA356 and MPA are the same. This is because the lack of participation of higher modes in the response of building. In fact as it can be seen in Fig. 3 just first mode of vibration has inelastic response and other modes are elastic.

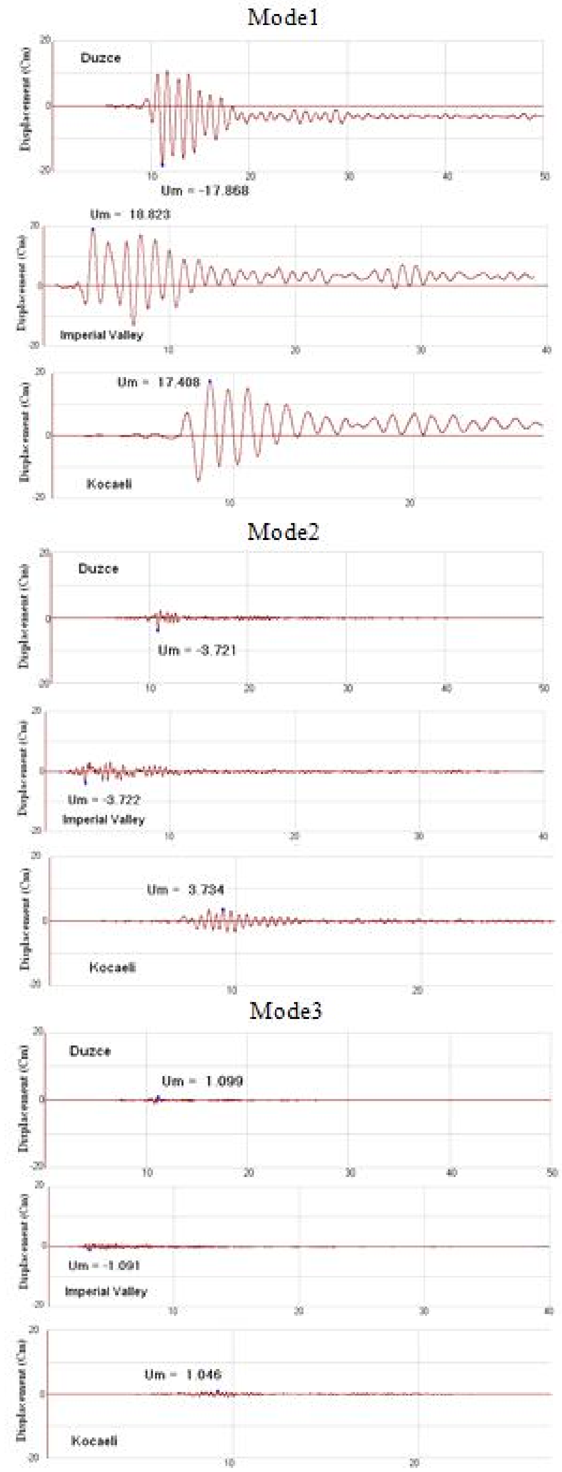

Figure 3 Response of inelastic SDOF in different modes to ground motion under NLTH analysis

Fig. 5 also shows inter story drift ratio evaluated by elastic analysis, FEMA356, MPA and NLTH. The figure shows that elastic analysis overestimate with large error percentage. The FEMA356 pushover procedure underestimates inter story drift ratio in lower and overestimates for upper stories with small error percentage in compare to elastic analysis. The MPA procedure generally overestimates drift ratio and yields better estimations of drift demands in comparison to FEMA356. Furthermore, Fig. 5 shows plastic hinge rotation estimated by FEMA356, MPA and NLTH. The figure shows

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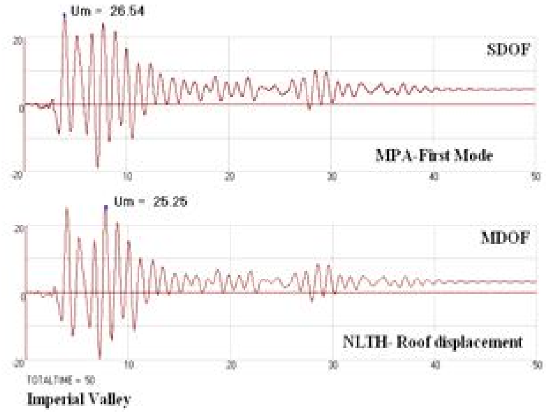

Figure 4. Compare first mode response of SDOF by MPA procedure with nonlinear time history roof displacement response.

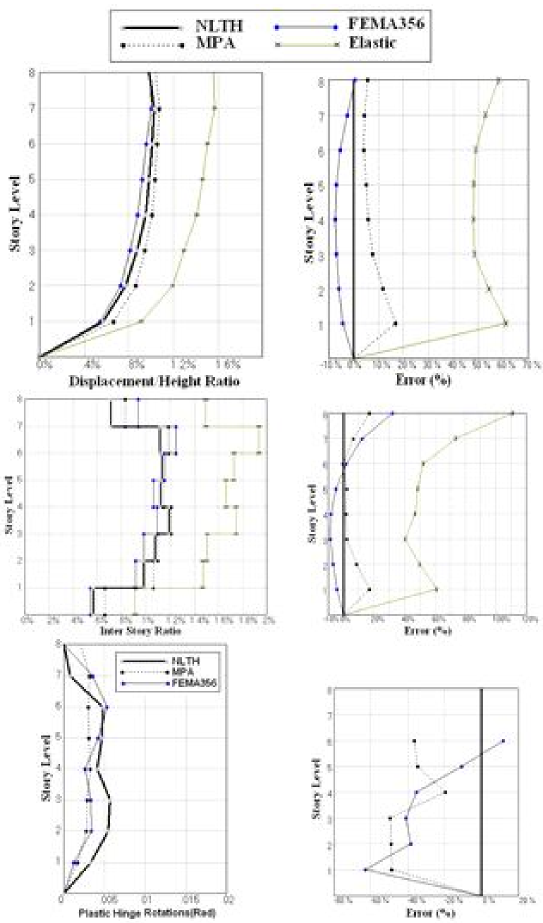

Figure5. Lateral displacement to height ratio, Inter story Drift Ratio, Plastic hinge rotation and their errors percentage evaluated by linear and nonlinear procedure

that generally none of both pushover procedures are accurate enough for capturing good results for evaluating plastic rotation. The amounts of plastic hinge rotations for roof story are zero and for 7th story are very small by NLTH but both FEMA356 and MPA estimate unrealistic amounts for these stories thus their estimations are eliminate from the error percentage graph.

CONCLUSION

This paper has evaluated equivalent linear static analysis of Code2800 and nonlinear static procedure offered by FEMA356 and modal pushover analysis to predict seismic demands in a sample designed steel building based on Iranian code of practice (CODE2800).

The building is subjected to three ground motions with special characteristics to the site specifications. The maximum results served as benchmark responses in comparison to elastic analysis, FEMA356 and MPA procedures results. The consideration of results is the bases for the following conclusions:

1. The results of nonlinear static and dynamic procedure show the state of designed building by equivalent linear static of code2800 is around immediate occupancy.

2. The equivalent linear static analysis of Code2800 can not predict accurate results in lateral displacements; inter story drifts and hinges plastic rotations and its results is too overestimate. Therefore analyzing and designing of high importance buildings with equivalent linear static procedures is over design.

3. The nonlinear static procedures can estimate reasonable results in lateral displacements, inter story drifts and hinges plastic rotations in comparison of linear procedure. These procedures are relatively fast and simple process for practical proposes especially for design offices.

4. In Comparison to FEMA356, the MPA procedure predicts more accurate results for inter story drifts and its advantage to FEMA356 is more important when the higher mode seismic effects are significant. It usually occurs for high or irregular buildings which participation of higher modes in seismic behavior of building is crucial.

5. In Comparison to FEMA356, the MPA procedure requires more steps to complete its procedures, but in compare to NLTH which has longtime process and intrinsic complexity, MPA procedure is very fast, simple to use and reasonable results to predict seismic behavior of building.

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