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ABSTRACT

In the past two decades, many investigations have been made on the methods related to the seismic retrofitting of the structures. As the real nonlinear behaviors of the elements are considered in these methods, the values of internal forces determined in the members are equal to the real ones. In the structural seismic linear analysis methods the elements behaviors are still assumed linear. In this regard, the passage of structure through yield point borders and the displacement increasing are correspondent to the forces elevation. In FEMA codes, the factors such as demand modifier (m) or increasing capacity ones are presented applying the effects of elements nonlinear behaviors in linear method analysis.

In this research the factors recommended in ASCE 41-06 are studied focusing on several symmetrical and asymmetrical concrete moment resisting frame moldings (5, 7 and 10 stories structures) and linear & non linear time history analysis (7 records). In this regard, the method used for scaling earthquake records is the current method presented in UBC97, “nonlinear dynamic analysis (RHA)”. According to this research there is direct relation between irregularity and increasing the amount of distribution among the results. In this way by increasing the eccentricity, difference between obtained results and recommended values will be more than the case with less irregularity.

INTRODUCTION

Recent studies show that the displacements represent better explanations of structural responses against forces as the structures enter the nonlinear behavior areas because of earthquake effects and the structural failure are controlled more effectively constraining the displacements other than forces[Albanesi,2002]. Now, the most logical method in the evaluation and retrofitting of available structures and designing new structures would be inelastic nonlinear analyses. The goal of applying these methods is to predict the structural behaviors in the future earthquakes. The importance of these methods will be bolded by performance based engineering development as a modern and new method in seismic designing and evaluating.

Performance based design focuses on the deformation because damage in structures is usually incorporate with deformation. Although the deformations are proportion to the forces, after material yielding submission or cracks formation, many deformations are formed as per marginal increase in forces level. Therefore, the deformations explain the structures statuses better than forces in nonlinear areas. Therefore, in design principle based on performance and in linear method analyses, the lateral forces because of earthquake are assessed in such a way that the deformations be close to the structure real deformation at maximum possible level. If the structural behavior is still linear, under such loading, forces and deformations of the analyses will be about reality. However, in the case of nonlinear material behavior, the forces are assessed over the real amounts [FEMA-274, 2004].

In the special situations, using simplified methods like linear static ones can result in accurate evaluation of forces and deformation amounts; the linear analyses advantage is the linear relation between forces and deformations. Therefore, it is simply possible to assess the forces and deformations in the different statuses of local combination. Fig. 1 shows the differences between two methods, linear and nonlinear. There is no significant difference between linear and nonlinear methods in "a" zone, but the lateral forces should be increased in "b" zone in order to achieve the deformation gained in linear analyses which is similar to those of nonlinear.

If the structures behave linearly because of loading, the gained forces for the structural elements are close to the earthquake predicted amounts as well. However, in case of the structural nonlinear behaviors, the forces calculated in this way will be greater than the material yielding amounts. Therefore, in the acceptance criteria assessment, the results of linear analyses should be revised for the structures having nonlinear behavior in the earthquakes.

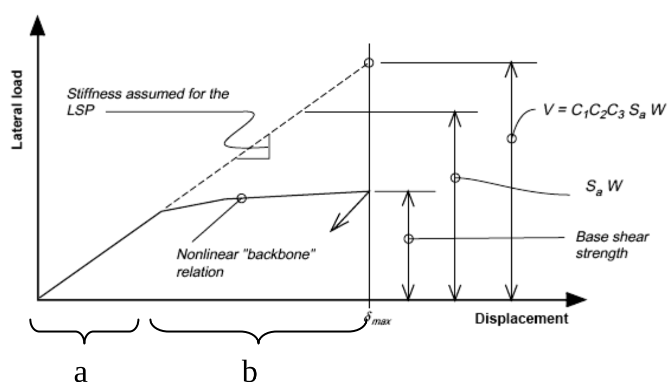

Fig. 1. linear and nonlinear methods differences [FEMA-356,2000]

LINEAR METHOD ANALYSIS ACCEPTANCE CRITERIA

Acceptance criteria basis is to classify all actions into two groups controlled by displacement and forces. For moment frames, the actions controlled by deformation are limited to the beams bending, even though the bending yield in columns is sometimes inevitable (at least in the foundation level). When using linear methods to design, the actions gained directly by these methods could be used only for determining the values equivalent to yield point. Design actions in other parts of structures should be determined using limited analysis method and concerning gravity loads plus yield actions applied on the free body diaphragm [FEMA-356,2000].

In ductile elements, showing great deformation in the linear areas after yielding, the actions determined by linear analyses are greater than their real amount in the elements. As the elements nonlinear behaviors are neglected in linear analyses, the energy dissipated because of elements nonlinear behavior are not concerned. Regarding the linearity of force-displacement relation, the displacement increasing in nonlinear zone will be proportion to the forces elevation. While parts of energy formed in the structures are absorbed and dissipated due to the structures entering nonlinear areas. Therefore, increasing displacement level does not necessarily mean force increasing.

In ASCE 41-06, modifying the action values in the element determined by linear methods to their values in nonlinear methods, the demand modifier factors or capacity modifier factors are used. (EQ.20-3 in FEMA356)

$$m\kappa\ Q_{CE} \geq Q_{UD} \tag{1}$$

Where, m is modifier factor based on element nonlinear behavior, κ is knowledge factor of structural details and characteristics and QCE is element expected capacity concerning any actions applying the elements simultaneously. The structural model of linear elastic behavior is placed subjected to the lateral load in linear analyses methods. The lateral load value is selected in such a way that the structural deformation be equal to the value predicted under design earthquake. Therefore, the internal forces are equal to the design earthquake ones as well. However, if the structural behavior is nonlinear in the earthquake, what usually happens, and then the forces obtained in the analyses will be more than the ones in the earthquake. The difference value depends on the element nonlinearity. In this regard, to compare internal forces with the element capacity, m factor has been involved [FEMA-274, 2004].

Fig.2. shows an element behavior controlled by displacement. In this fig. the results of linear analyses for the structure of nonlinear behavior is in accordance with the straight line. However, real nonlinear behavior of the structure is in accordance with the curved line; therefore, to compare forces with capacities, the element capacity is multiplied by m factor, artificially.

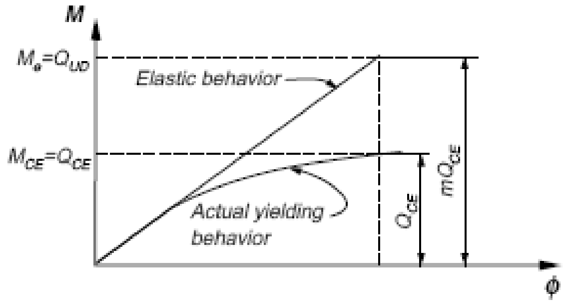

Fig. 2. m factor determination principles [ATC-40,1996]

STATIC NONLINEAR ANALYSIS AND SCALED NDP

Nonlinear methods are consisted of displacement factor and capacity spectrum methods presented in ASCE 41-06 [FEMA-356,2000] and ATC [ATC-40,1996], respectively. These methods practically lead to totally different estimations of maximum structural displacement during earthquake in unique structure. Comprehensive studies have been done to find the reasons of different results of the two methods and revising them to reach a unique response. The results of the investigation taken place in ATC55 project by the name of FEMA440 [FEMA 440,2004].

The approaches suggested in this study for revising the methods lead to the increasing accuracy in estimating target displacement. One of the shortages of nonlinear static methods is the disability to show the changes in nonlinear dynamic response of structures. This weak point can lead to inaccurate estimations of local demand forces and plastic deformations especially when through the structural yielding the role of higher modes is more emphasized. In Pushover analysis distribution of forces and target displacement determination is based on these two assumptions that the structural response is affected by the building principle mode and the form of this mode remains unchanged after structural yielding [Tjhin,2006].

It is obvious that after structural yielding, these two assumptions are both approximate. In order to reduce the errors and involve the effects of higher modes in seismic demands assessment, several researches have been made by Chopra and Goel., the results of them are presenting multi-mode pushover analyses(MPA) and revised MPA methods. All these methods show reliable results in special situations; therefore, the correctness of these methods in designing and assessment of structures is questionable [Chopra,2005].

Regarding the necessity of revising these inelastic analyses methods for concerning MDOF effects, few nonlinear dynamic analyses by scaled earthquake records are performed in such a way that peak roof displacement is equal to the target displacement predetermined by pushover analysis.

In this method, named scaled nonlinear dynamic analysis or scaled NDP, seismic hazard is determined by maximum inelastic roof displacement. Target displacement of structures can be determined using static nonlinear analysis methods and the ones presented in ASCE 41-06, ATC40 and revised relations in FEMA440. In the followings, the applicability of the mentioned methods in demand modifier factor determination and its comparison with current time history analysis method (RHA) will be assessed.

APPLIED STRUCTURAL MODELS

To determine the demand modifier factor values by two methods, scaled NDP and RHA, and their comparisons with the values presented in ASCE 41-06, 9 symmetric and symmetrical concrete structures are studied in the maps. These structures are consist of 5, 7 and 10 concrete stories in each of which two 10% and 20% eccentrics are used to study their effects on the dynamic behaviors of structural systems. The height of stories concerned in the structural models is 3 meters. These structures are designed based on the ACI 318-90 and in the form of special moment frame using ETABS software. For seismic loading of the structure, UBC97 principles are used for the zone of high seismicity.

Local soil profile is SD based on the UBC97 principles (shear wave velocity is between 180- 360 m/sec). The applied materials in this structure are the concrete (f_c= 210 kg/cm2) and Steel (f_y= 4200 kg/cm2). SAP2000 software is used for nonlinear dynamic and pushover analyses to model the structural components. The straight element with flexure behavior is used for beams and the interaction between axial and bending is used for columns. Regarding the concrete section behaviors, the cracked section modules are used in the analysis steps. According to the ACI code, 35% moment of inertia is used in cracked concrete sections of beams and 70% in the columns. The elements nonlinear properties in this software are in the form of point plastic hinges. These properties are selected based on ASCE 41-06 code according to the principles determining their brittle or ductile behaviors. Some of the structural plans are presented in Figs.3.

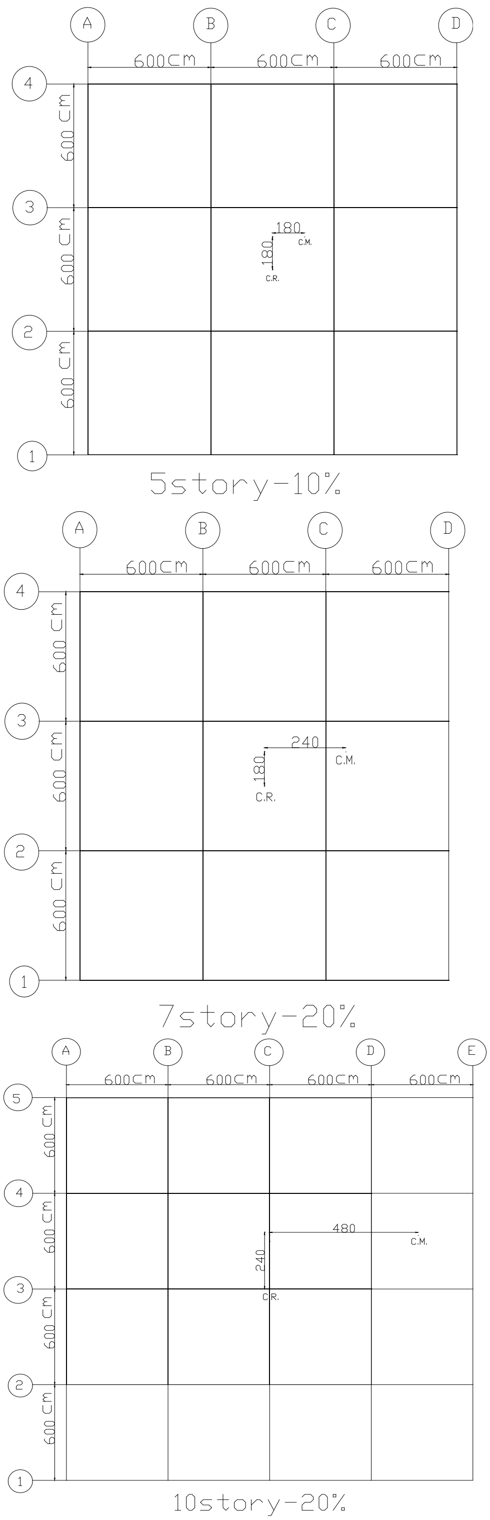

Fig. 3. model plans with the eccentrics 10% and 20%

APPLIED GROUND MOTIONS

7 earthquake records are used in nonlinear dynamic analyses. As the structure is designed for the soil type SD, according to UBC97 principles, the selected earthquakes are recorded in this soil type and scaled for that using ASCE 41-06 design spectrum. These 7 records are also used in scaled NDP method. However, the earthquake records are scaled as the peak roof displacement in nonlinear dynamic analyses be equal to the target displacement determined by static nonlinear analysis. The earthquake records characteristics are presented in table 1.

Table1: The accelerograms applied in nonlinear dynamic analyses

Earthquake	Date	Ms	Station Location (Number)	Component	PGA (g)	PGV (cm/sec)	Source Distance (km)	Data Source
Loma Prieta(G02)	1989/10/18 00:05	7.1	47380 Gilroy Array #2(47380)	000 090	0.367 0.322	32.9 39.1	12.7	CDMG
Imperial Valley(11)	1979/10/15 23:16	6.9	El Centro Array #11(5058)	140 230	0.364 0.38	34.5 42.1	12.6	USGS
Chi-Chi Taiwan(TCU122)	1989/09/20	7.6	(TCU122)	N W	0.261 0.22	34 42.5	10.03	CWB
Northridge (CNP)	1994/01/17 12:31	6.7	Canoga Park - Topanga(90053)	106 196	0.356 0.42	32.1 60.8	15.8	USC
Superstint Hill(ICC)	1987/11/24 13:16	6.6	El Centro Imp. Co. Cent(01335)	000 090	0.358 0.258	46.4 40.9	13.9	CDMG
Whittier Narrows	1987/10/01 14:42	5.7	Compton-Castlegate St(90078)	000 270	0.332 0.333	27.1 14.1	16.9	USC
Morgan Hill	1984/04/24 21:15	6.1	Gilroy Array #2(47380)	000 090	0.162 0.212	5.1 12.6	15.1	CDMG

DEMAND MODIFIER FACTORS (m)

In order to determine the demand modifier parameters, all models by applications of presented record are analyzed once by nonlinear time history and once by linear time history to determine the elements internal action values more accurately and near to reality. As the demand modifier factors are somehow connectors between the elements real internal demands determined by nonlinear and linear dynamic analyses, in this way to determine these factors in each acceptance criteria, the proportion of internal actions determined by linear dynamic analyses assuming the elements linear behaviors and the values gained by nonlinear dynamic analyses assuming elements nonlinear behaviors are used. The average and average plus standard deviation values are schemed for both linear dynamic analyses (RHA) and scaled NDP analyses and compared with their corresponding values presented in ASCE 41-06.

Concrete Beams

The demand modifier factors gained by RHA analyses and SNDP method are plotted in Figs. 4-6 and compared with their corresponding values presented in ASCE 41-06 in the relevant criteria zone. The curves show that the results distribution are increased with eccentric increasing in both scaling methods UBC97 (the current time history analysis) and FEMA440 (Scaled NDP) methods. This increasing depends on the selected earthquake records and in case of different selections; the distribution may decrease in some cases. The distribution in results of scaled NDP method is lower than the ones in current RHA method anyway.

The demand modifier factor values in scaled NDP method are in appropriate accordance with the average values gained by current RHA method; however, the results distribution is very high in RHA method. This means that in case of selecting 7 records which is the least numbers recommended in UBC97 or IBC2000 codes, the responses may be assessed very low or very high. In other words, by scaling 7 records in this method, the responses values strongly depend on the kind of the records. Therefore, to ensure of the results correctness, more earthquake records should be used. While in scaled NDP the results distribution is very low and it can be concluded that the dependence rate of demand modifier factor on the selected earthquake records in this method is lower than the ones in current RHA method. Therefore, the proper accuracy can be achieved by fewer records. As it can be seen in the curves, in **IO** acceptance criteria, scaled NDP gives lower evaluation of demand modifier factor in comparison with RHA method and its results are closer to the values presented by ASCE 41-06.

In **LS** acceptance criteria, the scaled NDP estimation of demand modifier factor are higher comparing with RHA method; however, these values are closer the ones presented in ASCE 41-06. The scaled NDP estimations are higher than RHA in **CP** excluding the 7 story model and closer to ASCE 41-06 values of demand modifier factors for concrete beams controlled by bending. This difference shows that in RHA method using 7 earthquake records can not present accurate evaluation of elements action rate in the estimation of their internal demands. Therefore, more earthquake records should be used in RHA method for analysis to reduce the results distribution and decrease the difference between values gained by ASCE 41-06 corresponding factors and the ones of scaled NDP analysis.

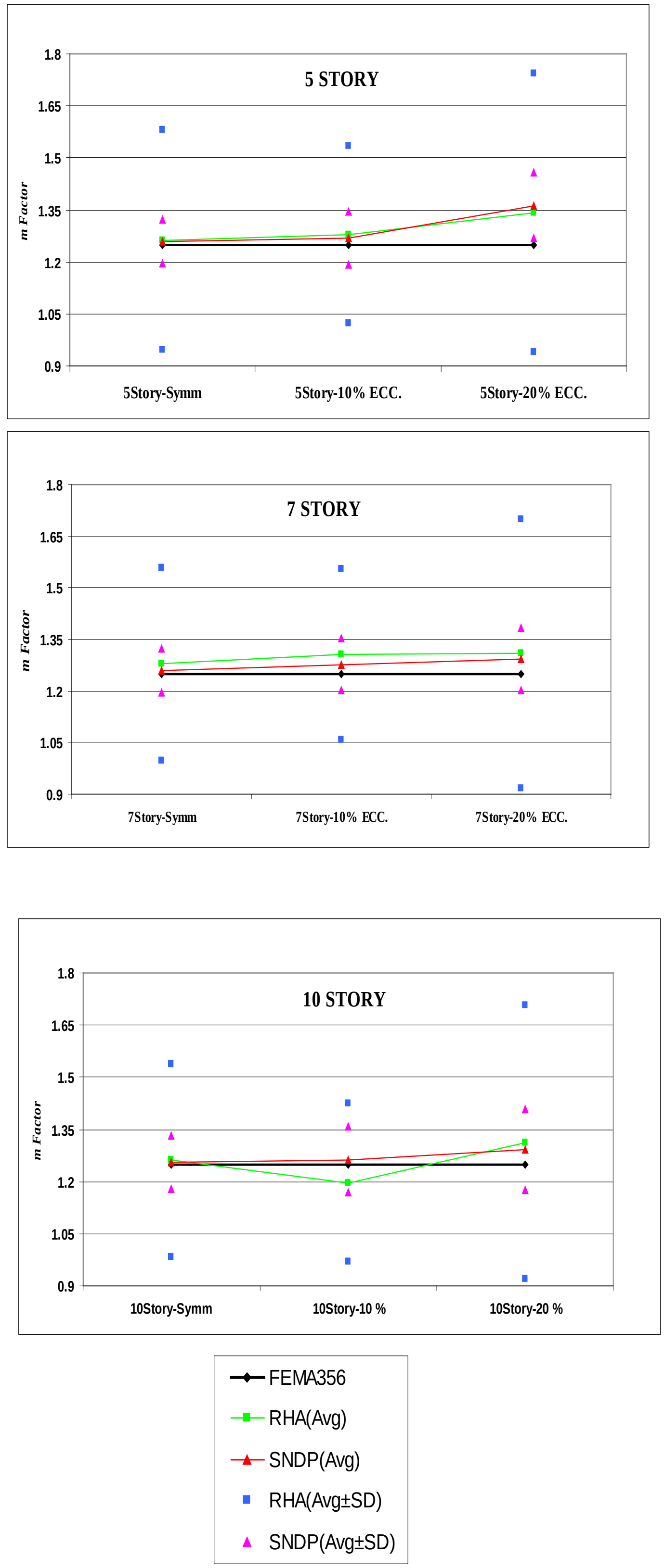

Fig. 4. m factor values for concrete beams in scaled NDP analyses and comparing with RHA method values in IO acceptance criteria zone

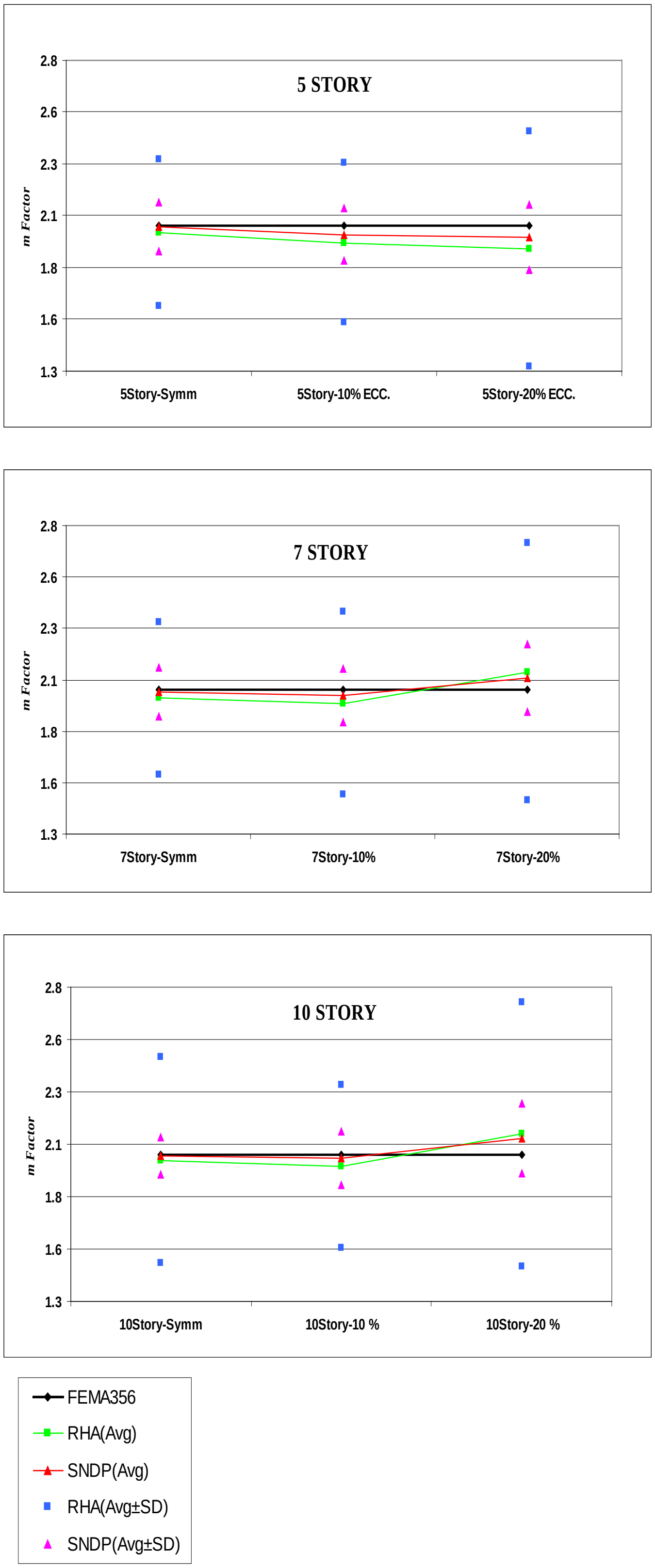

Fig. 5. m factor values for concrete beams in scaled NDP analyses and comparing with RHA method values in LS acceptance criteria zone

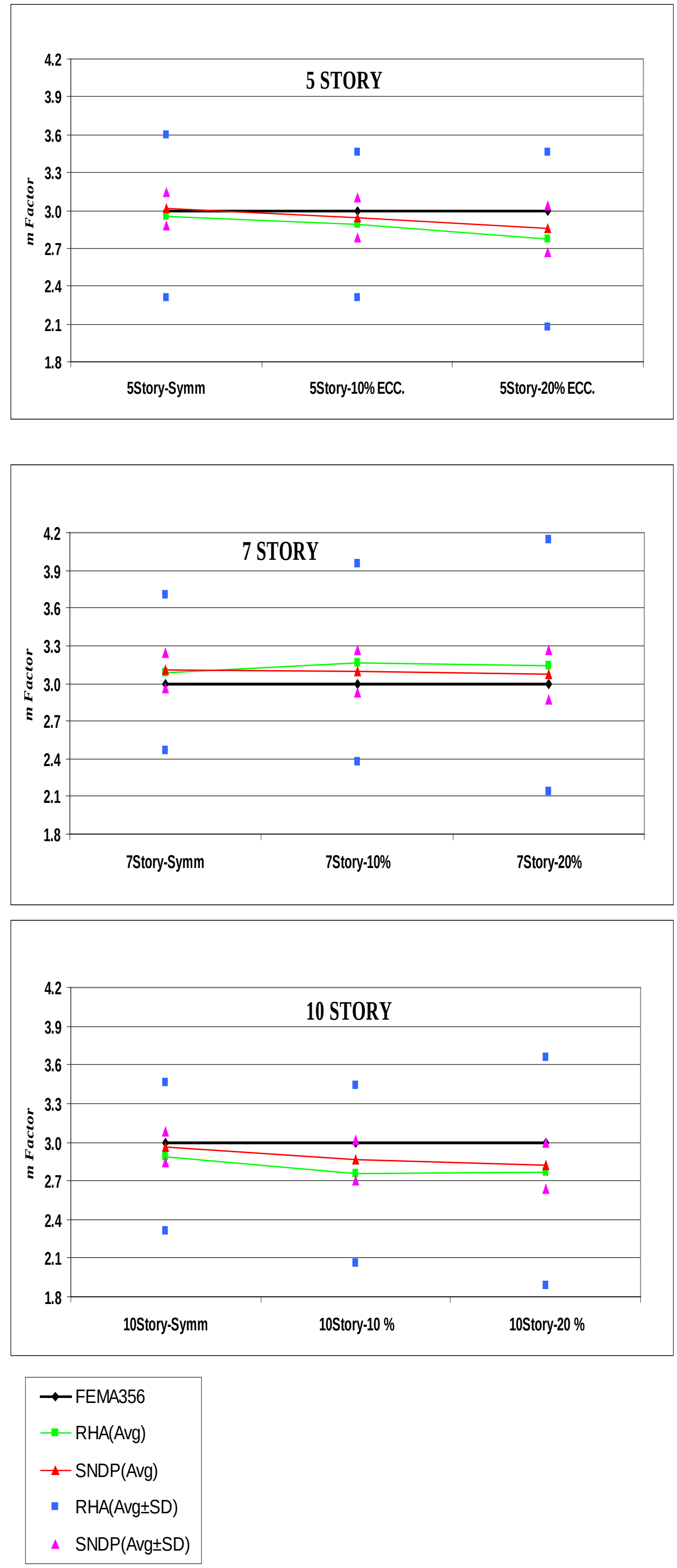

Fig. 6. m factor values for concrete beams in scaled NDP analyses and comparing with RHA method values in CP acceptance criteria zone

Concrete Columns

The demand modifier factors of columns gained by RHA analysis and SNDP are schemed in Figs.7-9 and compared with corresponding values presented in ASCE 41-06 in the relevant acceptance criteria. As it is obvious in the mentioned figures, the distribution of results for concrete columns, like beams, increases with eccentric increasing in both scaling methods, UBC97 (RHA) and FEMA440 (SNDP). The average value of error will increase with the eccentric increasing as well. The demand modifier factors values in scaled NDP method for columns, like beams, are in acceptable accordance with the average values gained by current RHA method; however, the results distribution is very high in RHA method.

In order to compare the results in RHA and scaled NDP analyses for columns, concerning the mentioned curves, it is observed that in IO acceptance criteria, scald NDP method gives lower estimation of demand modifier factor comparing with RHA one and its results are closer to the values presented in FMA356. The scaled NDP estimations in determining the demand modifier factors are lower in comparison with RHA method in LS acceptance criteria as well; while these values are closer to the ones presented in ASCE 41-06. In CP zone, excluding 5-story model, scaled NDP estimations are lower than the ones of RHA method and closer to FEMA values of demand modifier factors for concrete columns controlled by bending.

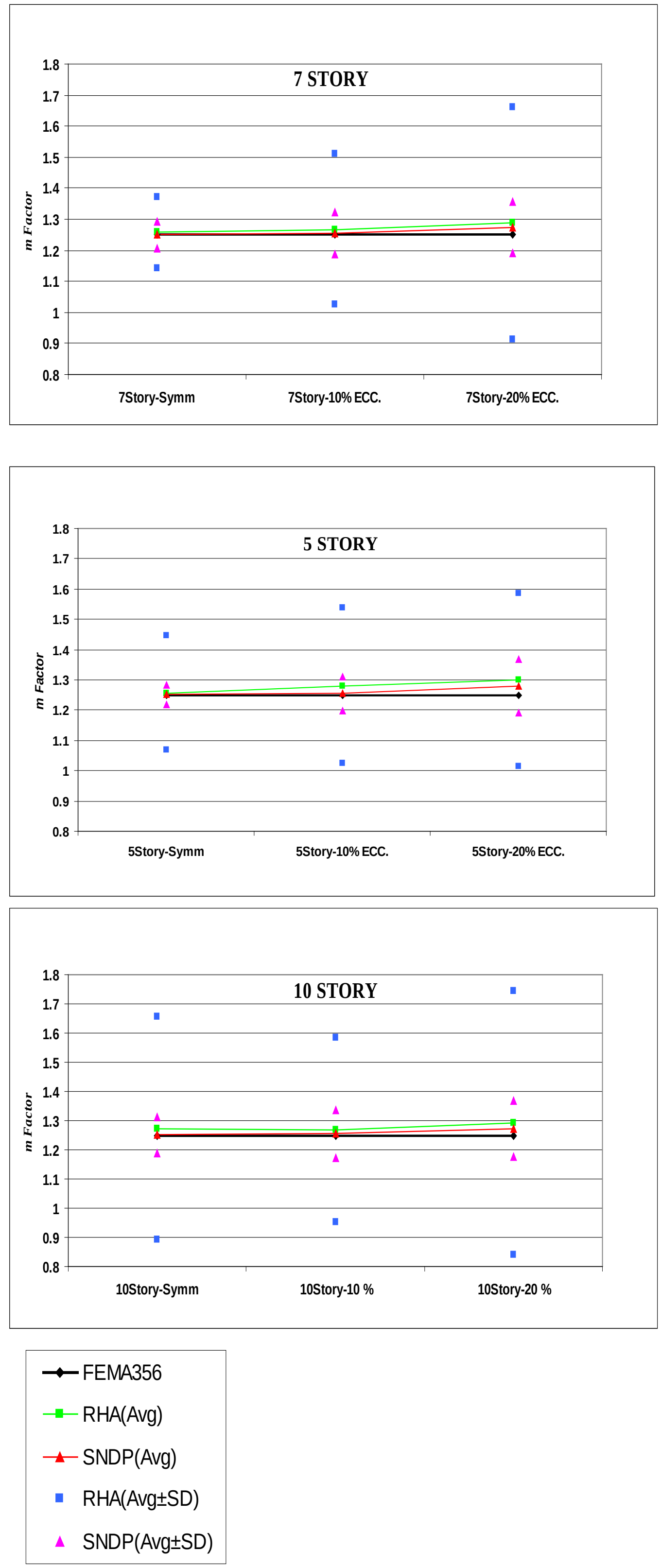

Fig. 7. m factor values for concrete columns in scaled NDP analyses and comparing them with RHA method values in IO acceptance criteria zone

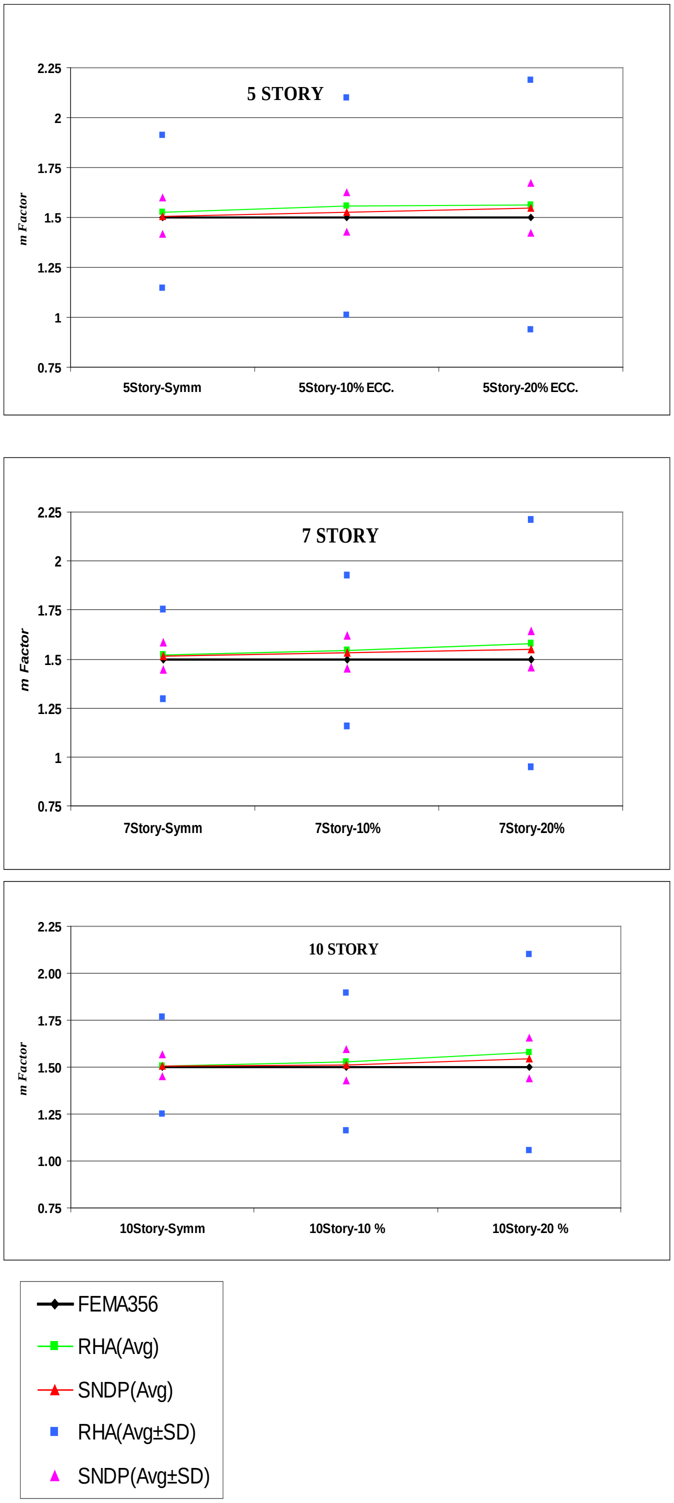

Fig. 8. m factor values for concrete columns in scaled NDP analyses and comparing them with RHA method values in LS acceptance criteria zone

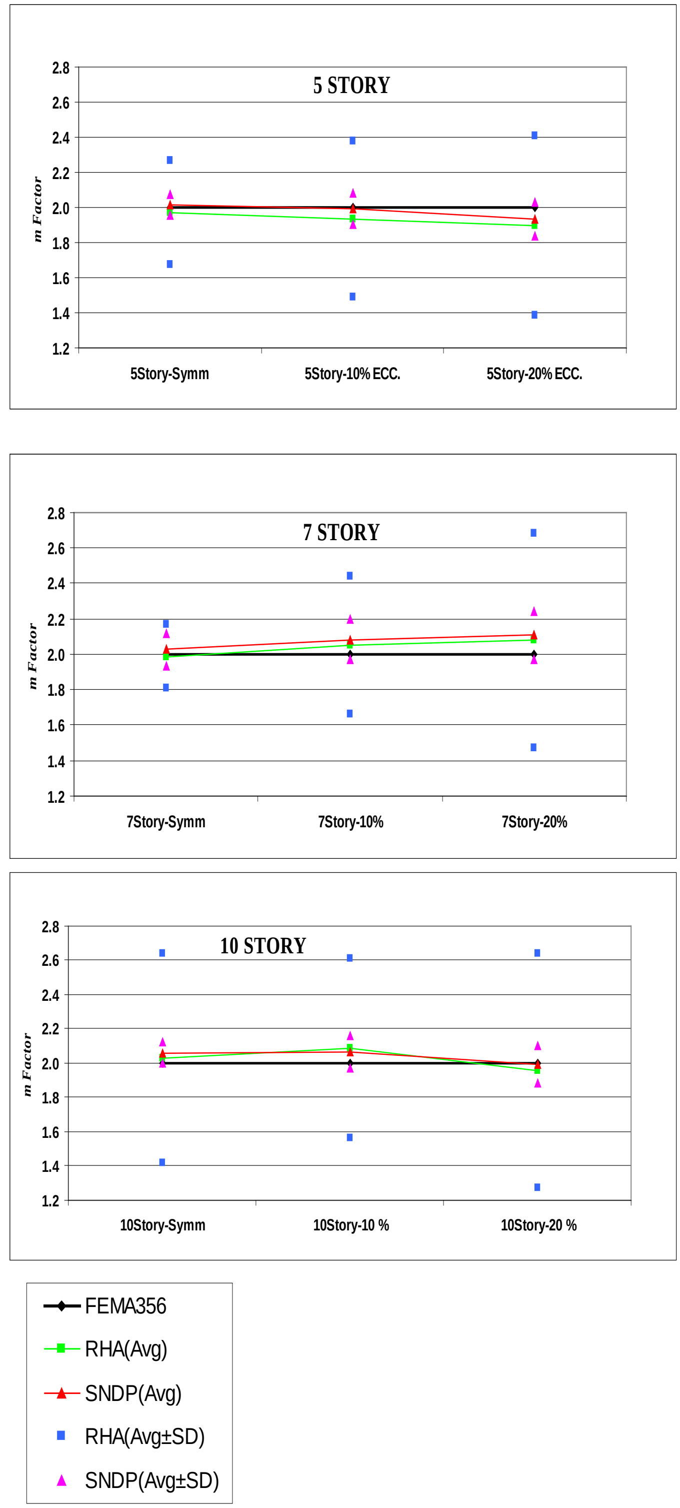

Fig. 9. m factor values for concrete columns in scaled NDP analyses and comparing them with RHA method values in CP acceptance criteria zone

CONCLUSION

In this article the demand modifier factor values presented in ASCE 41-06 are studied using two analytical methods, nonlinear dynamic analysis (RHA) and scaled nonlinear dynamic analysis (SNDP), and the gained results are compared. The most important results are:

1- The average values of demand modifier factors gained by current nonlinear dynamic analysis method (RHA) and scaled NDP method are in good accordance with the corresponding values presented in ASCE 41-06 for columns and beams in symmetric structures. However, the differences between these values increase with eccentric increasing showing that the eccentric effect has been less concerned in ASCE 41-06 and the presented values are better for symmetric structures.

2- The results distribution in RHA method is higher than the ones in scaled NDP. The distribution is because of many uncertainties existed in seismic nonlinear analysis. Therefore, to ensure the results correctness, more earthquake records should be applied; while the results distribution is much lower in scaled NDP method and the dependence of analysis results on the selected earthquake records is lower than the one in current RHA method. Therefore, the proper accuracy can be achieved by using fewer earthquake records.

3- The nonlinear dynamic analysis method (RHA) gives higher estimations of demand modifier factors in comparison with the ones presented in ASCE 41-06 in IO acceptance criteria zone. Scaled NDP method gives lower estimations of demand modifier factors comparing to RHA method and its values are closer to the ones presented in ASCE 41-06 codes. In these two methods, the difference between determined values and the ones presented in ASCE 41-06 increases with eccentric increasing.

4- For beams in LS acceptance criteria zone, the current nonlinear dynamic analysis method evaluation of the demand modifier factor is lower, excluding 7 and 10 story structures with 20% eccentric, comparing the values presented in ASCE 41-06. Scaled NDP method estimations are higher than the values of demand modifier factors in RHA method and the values are closer to the ones presented in ASCE 41-06.

5- The values of beams in CP acceptance criteria zone, determined by both two methods, scaled NDP and RHA, are lower than the ones presented in ASCE 41-06. However, the values gained by scaled NDP method has lower distribution and are closer to the values presented in ASCE 41-06.

6- The values of the two methods, scaled NDP and RHA, in IO acceptance criteria zone are higher than the ones of ASCE 41-06 for concrete columns controlled by bending. Comparing the results of the two methods, those of scaled NDP method are closer to the values presented in ASCE 41-06 and lower than the results of RHA method

7- In LS acceptance criteria zone, current nonlinear dynamic analysis (RHA) gives higher estimations of demand modifier factors for columns than the values presented in ASCE 41-06. Scaled NDP method gives lower estimations of demand modifier factors in comparison with RHA method, and the determined values are closer to the ones presented in ASCE 41-06.

8- Structural behaviors are different according to their columns height in CP acceptance criteria zone. Regarding 5 story structures, independent of eccentric, values determined by RHA method are lower than the values presented in ASCE 41-06. The values determined by scaled NDP method are lower than the ones presented in ASCE 41-06 as well; however, these results are higher in comparison with RHA method and closer to FEMA values. The values gained by RHA and scaled NDP methods for 7 story structures are slightly higher than the ones presented in ASCE 41-06. However, the values gained by scaled NDP method are closer to ASCE 41-06 ones. Regarding 10 story structures, excluding the 20% eccentric status, the values gained by RHA and scaled NDP methods are higher than those presented in ASCE 41-06, but the values gained by scaled NDP method are closer to ASCE 41-06 values.

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