Evaluating the Seismic Performance of Multi-Story Buildings with Steel CBFs of Various Placements by Using Nonlinear Time History Analyses

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SUMMARY:
This paper investigates the potentialities of the pushover analysis (POA) to estimate the seismic deformation demands of concentrically braced steel frames (CBFs). Reliability of the pushover analysis has been verified by conducting nonlinear dynamic time history analysis (THA) on 3, 6 and 10 story buildings subjected to 9 earthquake records representing a design spectrum. The drift ratios and plastic hinge rotations compare well with those from ground motions in steel frames with various numbers of stories and bays. It is shown that pushover analysis with predetermined lateral load pattern provides questionable estimates of inter-story drift. Results showed that the columns which act a member of CBFs are under-designed, particularly in taller buildings. Relatively good agreement was observed between the results of POA and THA in case of 3-story buildings, while in case of 10-story buildings the drift values obtained by THA are generally larger than those obtained by POA, particularly in upper stories.

Keywords: Pushover analysis, Nonlinear time history analysis, Performance level.

1. INTRODUCTION

Recent advances in the field of earthquake engineering are quickly paving the way towards the implementation of more rationale seismic-resistant design procedures. Performance-based earthquake engineering (PBEE) states the methodology in which structural design criteria are expressed in terms of achieving a set of different performance objectives [3].

These performance objectives are defined for different levels of excitations and they can be related to the level of structural damage, which in turn can be related to displacements and drift ratios [3]. To limit lateral deformations, an adequate estimation of peak lateral deformation demands on structures subjected to different ground motions is needed. Structures subjected to such ground motions are likely to undergo inelastic deformations [6]. The estimation of seismic performance with nonlinear dynamic analyses entails significant uncertainty due to the inherent randomness in ground shaking, and it needs advanced knowledge and engineering judgment.Therefore seismic codes recommend techniques that are simple enough so that they can be applied effectively according to the knowledge of the professionals involved. However, the simplicity should not compromise the reliability of such procedures. To accomplish this goal, new reliable procedures that can be developed according to the state of the art in seismic engineering should be developed [7].

Both structural and nonstructural damage sustained during earthquake ground motions are primarily produced by lateral displacements. Thus, the estimation of lateral displacement demands is of primary importance in performance-based earthquake-resistant design, especially, when damage control is the main quantity of interest.

Nonlinear time history analysis of a detailed analytical model is perhaps the best option for the estimation of deformation demands. However, there are many uncertainties associated with the generation of site-specific input and with the analytical models presently employed to represent structural behavior. [2]

2. MODELING AND ASSUMPTIONS

In the present study, a set of 3-, 6-, and 10-story steel building structures were considered in three different plan sizes of 20m x 15m, 25m x 20m, and 35m x 25m, totally nine different building systems, as shown in Fig. 1, 2, 3. and were designed according to the latest version of Iranian Standard 2800, released in 2005, which is very similar to IBC 2003 The buildings are assumed to be located on a soil type B and in a seismically active area for the earthquake with probability of 10% in 50 years (earthquake hazard level 1) with PGA of 0.35g.

Frames in the x directions are CBFs and all connections in that direction are considered to be simple and the frames in Y direction are ordinary moment resistant frame. The frame members were sized to support gravity and lateral loads. IPE and UNP sections, according to DIN standard, are chosen for columns, beams and bracings, respectively. To eliminate the over-strength effect, auxiliary sections have been artificially developed by assuming a continuous variation of section properties. In the code type design, once the members were seized, the entire design was checked for code drift limitations and if necessary refined to meet the requirements.

For the static and nonlinear dynamic analyses, the computer programs ETABS Nonlinear version 9.7.0 [11] and SAP2000 Advanced 14.2.2 [12] was used to predict the frame responses. To investigate the accuracy of different methods to predict the seismic response of concentrically braced steel frames, nine pairs of accelerograms, recorded on soil type of class B, used and normalized to 0.35g.

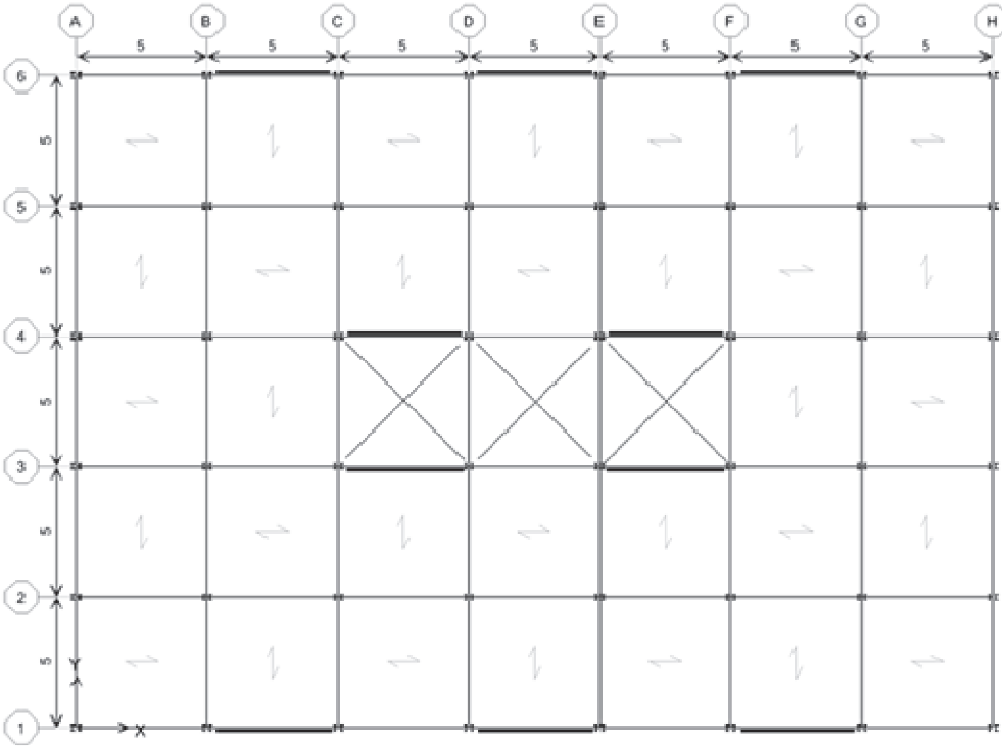

Figure 1. Large plan (35m x 25m) for 3, 6 and 10 story buildings and locating of braces.

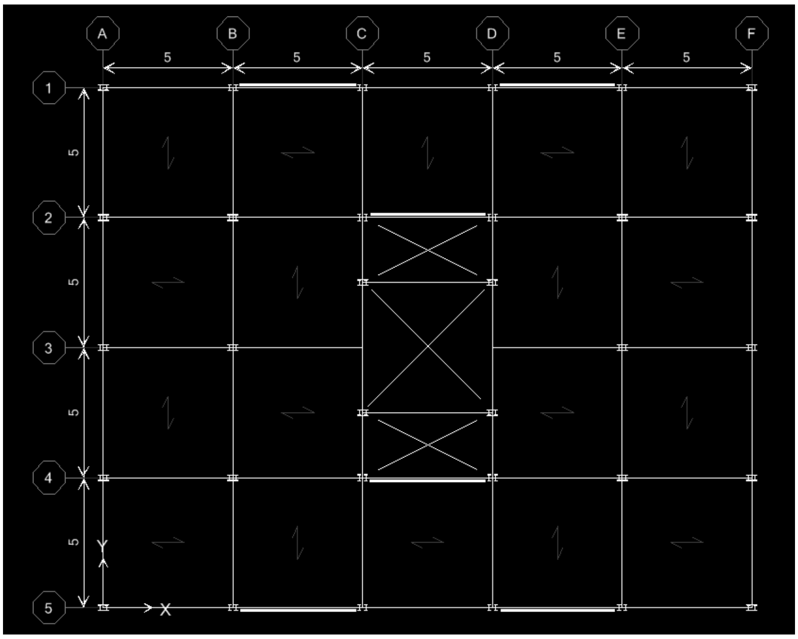

Figure 2. Medium plan (25m x 20m) for 3, 6 and 10 story buildings and locating of braces.

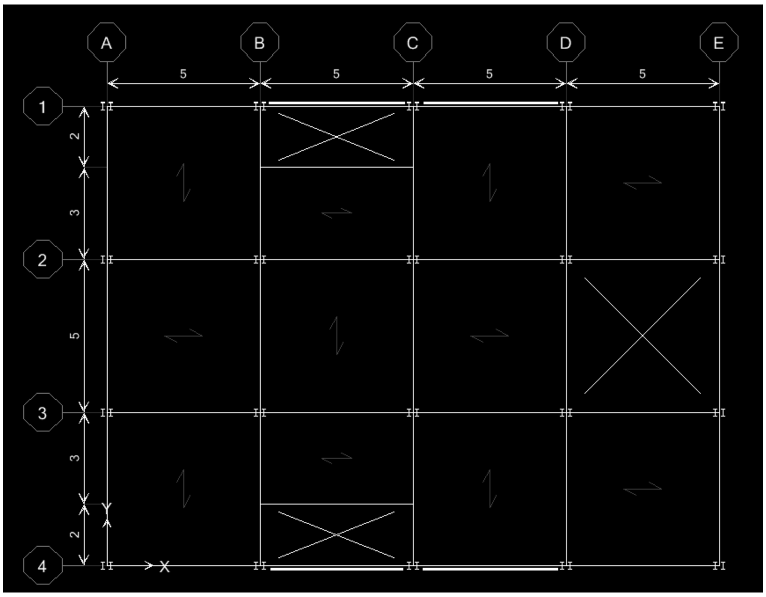

Figure 3. Small plan (20m x 15m) for 3, 6 and 10 story buildings and locating of braces.

3. NONLINEAR PROCEDURE

In the POA, or pushover analysis, monotonically increasing lateral forces are applied to a nonlinear mathematical model of the building until the displacement of the control node at the roof level exceeds the target displacement. The lateral forces should be applied to the building using distributions or profiles that bound, albeit approximately, the likely distribution of inertial forces in the design earthquake. To perform Pushover analysis two loading Patterns (uniform and spectral) are used. To find the target point in different buildings, coefficient method is used. The NEHRP guidelines [8, 9] indicate that, for a specific earthquake, the building should have enough capacity to withstand a specified roof displacement. This is called the target displacement and is defined as an estimate of the likely building roof displacement in the design earthquake. The guidelines give an indication of how to estimate the target displacement using the following expression:

$$\delta t = C_0 C_1 C_2 C_3 Sa T_e^2 / 4\pi^2 \quad (1)$$

Where C_0 = modification factor to relate the spectral displacement and expected maximum elastic displacement at the roof level; C_1 = modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C_2 = modification factor to represent the effects of stiffness degradation, strength deterioration, and pinching on the maximum Fig. 4. Idealized force–displacement curves. displacement response; C_3 = modification factor to represent increased displacements due to dynamic second order effects; *Te* = effective fundamental period of the building in the direction under consideration calculated using the secant stiffness at a base shear force equal to 60% of the yield force; and *Sa* = response spectrum acceleration at the effective fundamental period and damping ratio of the building. The factors C_1, C_2, and C_3 serve to modify the relation between mean elastic and mean inelastic displacements where the inelastic displacements correspond to those of a bilinear elastic–plastic system. The effective stiffness, *Ke*, the elastic stiffness, *Ki*, and the secant stiffness at maximum displacement, *Ks*, are identified in Fig. 4. To calculate the effective stiffness, *Ke*, and yield strength, *Vy*, line segments on the force–displacement curve were located using an iterative procedure that approximately balanced the area above and below the curve [8, 9]. A nonlinear static procedure was used to evaluate the seismic performance of 3-, 6- and 10-story concentrically braced frames shown in Fig. 1,2,3. To accomplish this, target displacement corresponding to the UBC 1997 [10] design spectra was estimated in accordance with Eq. (1). Subsequently, the pushover analysis was performed under a predetermined load pattern to achieve the target displacement. Story demands computed at this stage are considered as estimates of the maximum demands experienced by the structure in the design earthquake. For all pushover analyses, two vertical distributions of lateral load are considered: a uniform distribution proportional to the total mass at each level; and a vertical distribution proportional to the lateral force from spectral linear dynamic analysis.

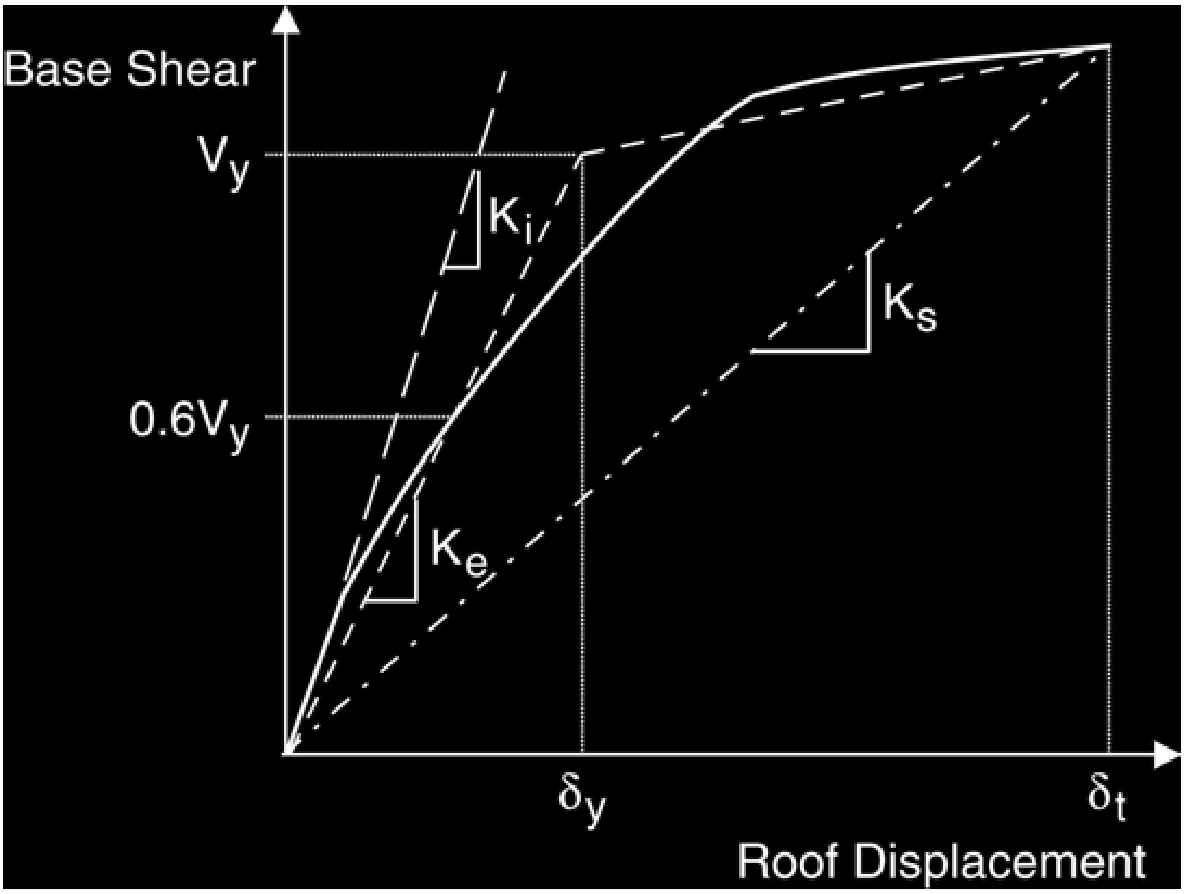

Figure 4. Idealized force–displacement curves.

Values of target displacements δ_t obtained from POA and the maximum displacement of roof δ_{MAX} obtained from THA is shown in Table.1.

Table.1. Coefficients and target displacements through POA and maximum roof displacement through THA.

Analysis		Coefficients	3story buildings			6story buildings			10story buildings		
			Large plan	Medium plan	Small plan	Large plan	Medium plan	Small plan	Large plan	Medium plan	Small plan
Nonlinear static	Spectral load pattern	C_0	1.06	1.03	1.04	1.2	1.2	1.07	1.43	1.22	1.17
		C_1	1.22	1.24	1.27	1	1	1.02	1	1	1
		C_2	1.18	1.19	1.2	1.1	1.1	1.11	1.1	1.1	1.1
		C_3	1	1	1	1	1	1	1.03	1.02	1
		S_a	0.87	0.87	0.87	0.73	0.73	0.87	0.51	0.58	0.64
		T_e	0.32	0.30	0.28	0.65	0.65	0.48	1.12	0.92	0.79
		δ_t(cm)	3.49	3.06	2.79	10.20	10.26	6.13	26.2	16.8	13.03
	Uniform load pattern	C_0	1.19	1.20	1.18	1.31	1.27	1.24	1.39	1.37	1.30
		C_1	1.25	1.22	1.29	1	1	1.01	1	1	1
		C_2	1.19	1.18	1.2	1.1	1.1	1.11	1.1	1.1	1.1
		C_3	1	1	1	1	1.04	1.04	1	1	1
		S_a	0.87	0.87	0.87	0.8	0.8	0.87	0.56	0.58	0.65
		T_e	0.3	0.32	0.26	0.56	0.56	0.48	0.96	0.91	0.78
		δ_t(cm)	3.5	3.8	2.8	9.19	9.55	7.65	20	18.05	14.13
Nonlinear dynamic		δ_{MAX}(cm)	2.8	3.3	2.7	9.64	9.56	9.4	35.6	34.9	29.7

In order to demonstrate the validity of the nonlinear static procedure to predict the displacement demands of concentrically braced frames, a set of time history analysis (THA) were conducted by using nine pairs of accelerograms, recorded on soil type of class B, and normalized and scaled with UBC spectrum to 0.35g Fig.5,6,7. Nonlinear dynamic analyses have been performed for all 9 earthquakes records Table.2.

Maximum story`s drift obtained from both analysis are compared in Fig.8. The other parameters investigated in these analyses are plastic deformation in column and bracing. These results are compared with acceptance criteria and the performance level of building and elements that are determined by FEMA356 [9].It is shown that the results obtained by this method are slight underestimates. However, the accuracy of the nonlinear static procedure to predict the maximum roof displacement caused by the design ground motion seems to be acceptable for practical applications. Similar conclusions are reported by Gupta and Krawinkler [13] for regular SMRF structures. In order to evaluate the relative accuracy of pushover analysis for prediction of maximum story drift demands in individual stories, for a given target roof displacement, the results are compared with the average of those of 9 earthquakes. As shown in Fig. 8, the nonlinear static procedure provides questionable estimates of interstory drift demands for the concentrically braced frames investigated in this study. The results illustrated in this figure were obtained by using a vertical distribution of lateral loads using different distribution patterns; the effects of pre-assumed lateral load on the results of pushover analysis have been investigated. One can clearly observe from this figure that the results are very sensitive to the choice of lateral load pattern and that there is a very large scatter in the observations, particularly for the maximum drift distribution. In 3- story buildings POA with both lateral pattern and THA estimate the maximum story`s drift almost the same but in 6- and 10 story buildings below three story , POA with uniform load pattern and then spectral load pattern and then THA respectively estimate larger maximum story drift and for higher than 3 story the opposite is right. Accordingly, an acceptable estimation of story drift demands over the height of the structure is difficult to accomplish by using the nonlinear static procedure because of the dependence on different factors such as the relative strength and stiffness of the stories, effects of higher mode, pre-assumed lateral load pattern and characteristics of the ground motions.

Table.2. Earthquakes used for THA, PGA is maximum acceleration and T is corresponding period of maximum Pseudo-velocity.

NO.	Earthquake Name	Station	PGA(g)	T(s)
1	WHITTIER NARROWS 1987	PASADENA LURA ST	0.36	0.28
2	NORTHRIDGE 1974	TOPAGANA-FIRE STA	0.275	0.24
3	SAN FERNANDO 1971	CASTAIC OLD RIDGE ROUTE	0.339	0.34
4	NORTHRIDGE 1974	LA - N WESTMORELAND	0.355	0.4
5	MAMMOTH LAKES 1980	CONVICT CREEK	0.438	0.58
6	WHITTIER NARROWS 1987	PASADENA LURA ST	0.339	0.7
7	NORTHRIDGE 1974	ARLETA	0.349	0.84
8	LOMA PRIETA 1989	HOLLISTER - South & Pine	0.377	1.04
9	CAPE MENDOCINO 1992	RIO DELL OVERPASS FF	0.357	1.07

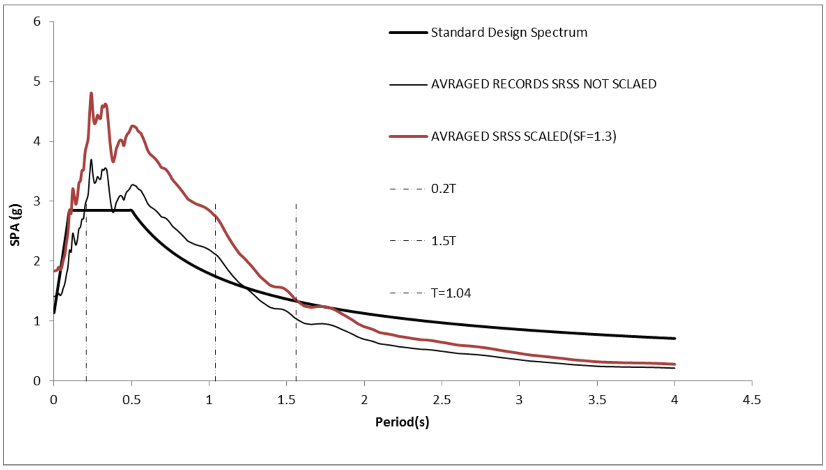

Fig.5. Tidal mean sum of squares of response spectrum chosen for 3-story buildings with T=0.32s scaled with UBC design spectrum and the scale factor is 1.3.

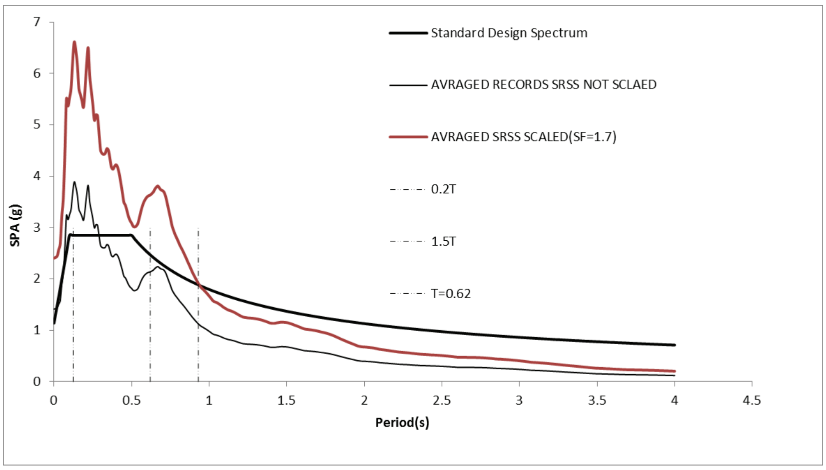

Fig.6. Tidal mean sum of squares of response spectrum chosen for 6-story buildings with T=0.62s scaled with UBC design spectrum and the scale factor is 1.7.

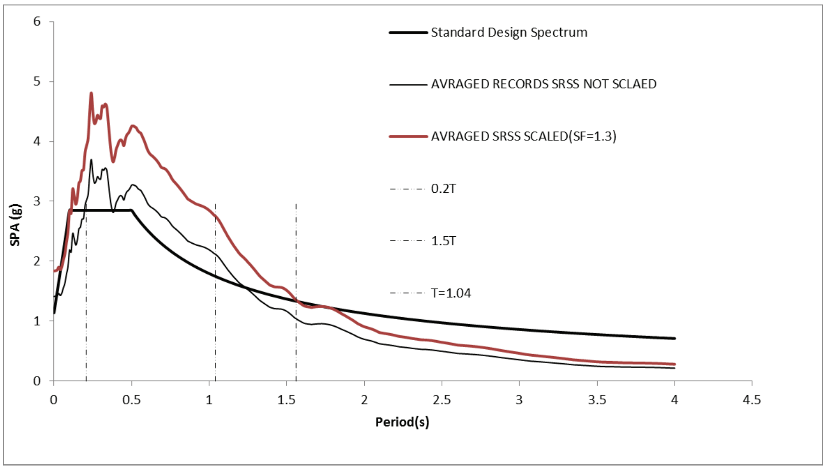

Fig.7. Tidal mean sum of squares of response spectrum chosen for 10-story buildings with T=1.04s scaled with UBC design spectrum and the scale factor is 1.3.

4. OVERALL PERFORMANCE OF BUILDINGS

All the studied buildings are stable at the target displacements Table.1. and the base shear at this displacement is more than the yield base shear. Through maximum story drift Fig.8.for 3stories buildings there is immediate occupancy performance level and for 6 stories buildings there is life safety performance level with low risk, performance level for10 stories buildings is also life safety but with higher risk than the others. Acceptance criteria are from FEMA356 [9].

5. PERFORMANCE OF BUILDINGS COMPONENTS

In this study we have two kinds of components for braced steel frames, braces and braced columns, as shown in Table.3, 4. Performance level for both tensile and compression braces is life safety. Braced columns that are under compression have force controlling behavior so the ratio of demand force to capacity should be smaller than 1. As shown in table.5. These compression braced columns do not have a good performance especially in lower stories. Finally almost all the tensile braced columns performance level is immediate occupancy.

6. COMPARATIVE STUDY

Through Fig.8. We can compare nonlinear dynamic and static analyses; for 3 story buildings there are almost the same results for maximum drift of stories due to each analysis. For 6 and 10 story buildings, below three stories nonlinear static analysis estimates higher drifts than nonlinear dynamic analysis and for the rest opposite is right. We can also compare two analyses of estimating performance components of braced frames due to Table.3, 4, 5. Nonlinear dynamic analysis estimates weaker performance for compression braced columns than static analysis Tables.5. For tensile braces opposite is right it means that nonlinear static analysis estimate lower capacity for these components Table.3. Results of both analyses are the same for compression braces and tensile braced columns.

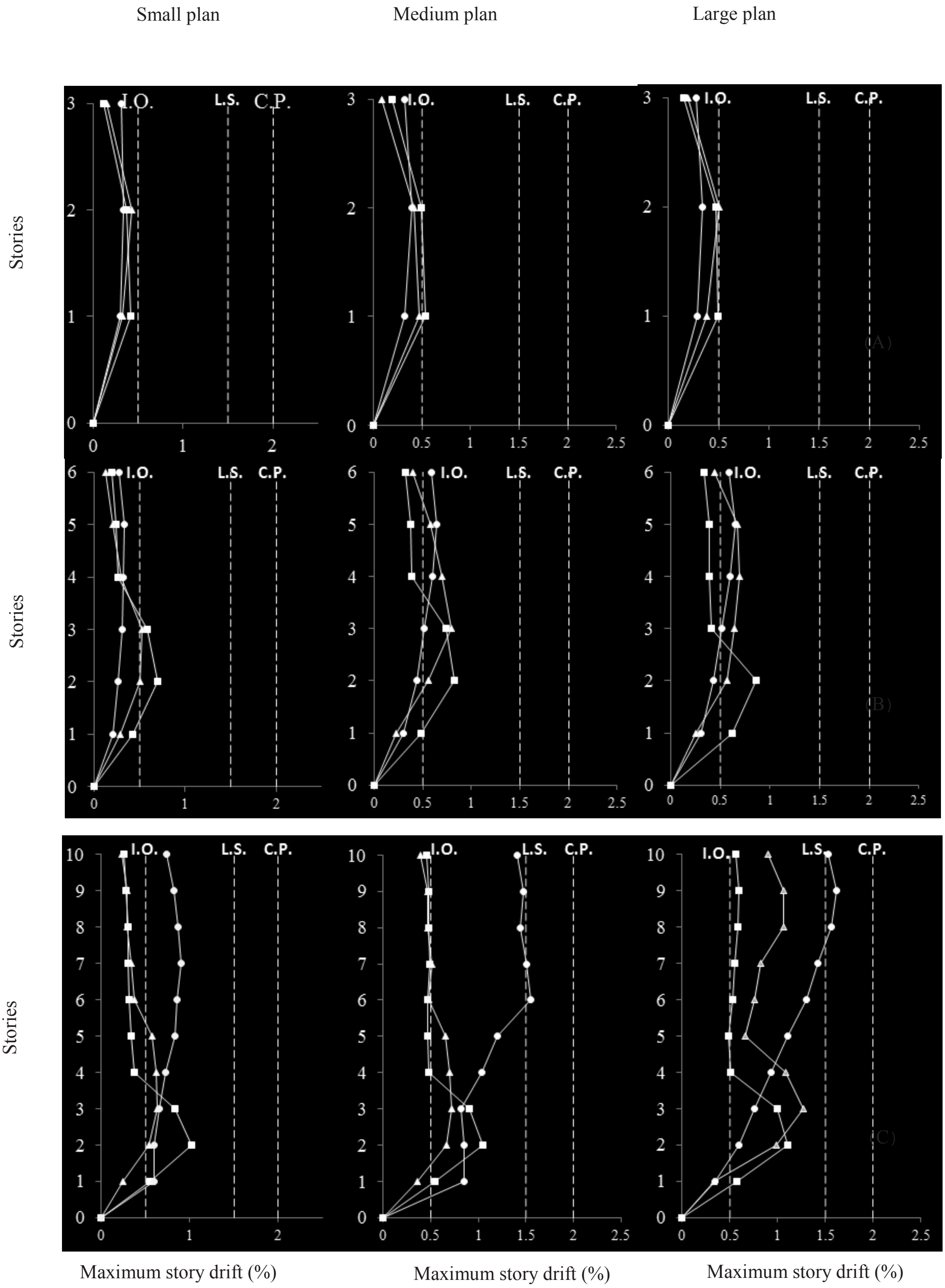

Fig.8. Maximum story drift results. Points ▬ show nonlinear static analysis results for uniform load pattern, points ▲ show nonlinear static analysis results for spectrum load pattern and points ● show nonlinear dynamic analyses. A, B and C for 3, 6 and 10 story buildings. Acceptance criteria are from FEMA356 [9].

Table.3. Performance levels for tensile braces and the corresponding hinges.

buildings	plan	Plastic hinges (%)	nonlinear static analysis with uniform load pattern	Plastic hinges (%)	nonlinear static analysis with spectral load pattern	Plastic hinges (%)	nonlinear dynamic analysis time history
			performance level		performance level		performance level
3 stories	small	33.3	immediate occupancy	12.5	life safety	33.3	immediate occupancy
	medium	16.7	life safety	16.7	life safety	16.7	immediate occupancy
	large	33.3	immediate occupancy	11.7	life safety	33.3	immediate occupancy
6 stories	small	16.7	life safety	8.3	life safety	8.3	immediate occupancy
	medium	25	life safety	13.9	life safety	25	immediate occupancy
	large	16.7	life safety	15	life safety	16.7	immediate occupancy
10 stories	small	15	life safety	10	life safety	2.5	life safety
	medium	15	life safety	11.7	life safety	10.8	life safety
	large	7.5	life safety	7.5	life safety	32	immediate occupancy

Table.4. Performance levels for compression braces and the corresponding hinges.

buildings	plan	Plastic hinges (%)	nonlinear static analysis with uniform load pattern	Plastic hinges (%)	nonlinear static analysis with spectral load pattern	Plastic hinges (%)	nonlinear dynamic analysis time history
			performance level		performance level		performance level
3 stories	small	33.3	life safety	33.3	life safety	39.6	life safety
	medium	33.3	life safety	33.3	life safety	33.33	life safety
	large	33.3	life safety	33.3	life safety	33.3	life safety
6 stories	small	25	life safety	25	life safety	33.3	life safety
	medium	26.4	life safety	36.1	life safety	30.2	life safety
	large	25	life safety	35	life safety	20	life safety
10 stories	small	22.5	life safety	21.9	life safety	43.8	life safety
	medium	21.7	life safety	25	life safety	21.7	life safety
	large	1	collapse prevention	0.5	collapse prevention	20	life safety

Table.5. Performance levels for compression braced columns with ratio of demand force to capacity (D/C).

buildings	plan	nonlinear static analysis with uniform load pattern	nonlinear static analysis with spectral load pattern	nonlinear dynamic analysis time history
		performance level	performance level	performance level
3 stories	small	D/C=0.51(OK)	D/C=0.53(OK)	D/C=0.76(OK)
	medium	D/C=0.74(OK)	D/C=0.64(OK)	D/C=1.07(NOT OK)
	large	D/C=0.51(OK)	D/C=0.54(OK)	D/C=0.74(OK)
6 stories	small	D/C=0.68(OK)	D/C=0.72(OK)	D/C=1.02(NOT OK)
	medium	D/C=0.71(OK)	D/C=0.76(OK)	D/C=1.03(NOT OK)
	large	D/C=0.8(OK)	D/C=0.82(OK)	D/C=1.07(NOT OK)
10 stories	small	D/C=0.72(OK)	D/C=0.74(OK)	D/C=1.47(NOT OK)
	medium	D/C=0.65(OK)	D/C=0.63(OK)	D/C=1.32(NOT OK)
	large	D/C=0.77(OK)	D/C=0.91(OK)	D/C=1.16(NOT OK)

6. CONCLUSION

In general, the results indicate that, for design earthquake, life safety performance is achieved. This result corresponds with the objective mentioned in Standard 2800.

In order to achieve the desired performance levels, it is necessary to eliminate some shortcoming observed in above-mentioned buildings, so that these performance levels can be maintained with a high degree of confidence. These shortcomings are as follows: A: Shortage in the capacity of braced columns and consequently, weak performance and high vulnerability in earthquake hazard level 1. B:Results show that by increasing the area of plan, period of the buildings increased and the performance level reduced but the Iranian code for seismic resistant design of buildings, Standard 2800 calculate the period of buildings only with height of buildings and do not mention the area of the buildings.

In comparison of nonlinear pushover analysis and dynamic time-history analysis, 6 and 10 story buildings have larger inter-story drift according to time-history analysis and for 3 story buildings both analysis have almost the same inter-story drifts. For taller than 3 story buildings pushover analysis with predetermined lateral load pattern are very sensitive to the choice of load pattern and provide questionable estimates of inter-story drift demands for concentrically braced steel frames.

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