IJST, Transactions of Civil Engineering, Vol. 37, No. C+, pp 395-407
Printed in The Islamic Republic of Iran, 2013

SEISMIC BEHAVIOR OF STEEL MOMENT RESISTING FRAMES ASSOCIATED WITH RC SHEAR WALLS*

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Abstract– In this paper, the influence of increasing the height of building on the seismic behavior of dual structural systems in the form of steel moment resisting frames accompanied with reinforced concrete shear walls has been investigated. Common structures experience inelastic stage of behavior encountering the seismic loads and the applied energy will be dissipated. The nonlinear responses of the structural models have been evaluated in this research. As a result, some parameters such as ductility factor of structure (μ), over-strength factor (R_s) and response modification factor (R) for the mentioned structures have been studied. To achieve these objectives, the buildings have 10 and 20 stories and contain such structural systems used to perform the pushover analyses having different load patterns. Regarding the results, it seems that the response modification factor (R) for the mentioned structural system is assumed to be higher than the value which is used in Iranian Code of Practice for Seismic Resistant Design of Buildings [Standard No.2800]. Analytical results showed that the ductility factor and the response modification factor increased as the structure height increased. In contrast, the over-strength factors increased by decreasing the height of the structure.

Keywords– Dual system, steel moment resisting frame, shear wall, steel bracing, reinforced concrete, seismic behavior

1. INTRODUCTION

Studying the behavior of building structures as subjected to severe earthquake ground motions reveals that these types of structures can exhibit enough strength, due to the nonlinear behavior of materials and possibility of the sufficient deformations of the structures. These structures absorb the applied energy and will dissipate it via tolerating the great displacements in nonlinear seismic behavior [1-4].

Nonlinear time history analysis of a detailed analytical model is perhaps the best option for the estimation of deformation demands. However, due to many uncertainties associated with the site-specific excitation as well as uncertainties in the parameters of analytical models, in many cases, the effort associated with detailed modeling and analysis may not be justified and feasible [5-7].

In recent years, nonlinear static analysis has received a great deal of research attention within the earthquake engineering community. Their main goal is to describe the nonlinear capacity of a structure when subjected to horizontal loading with a reduced computational effort with respect to nonlinear dynamic analyses. Pushover methods are particularly indicated for assessing existing structures (commonly not originally designed with seismic criteria in mind), when the employment of linear elastic methods, typical in new design situations, tends to be inappropriate. For these reasons, many codes and guidelines (e.g. [8-10]) recommend the use of nonlinear static methodologies to evaluate structural behavior under seismic action [11]. To assess the seismic performance of the structures, three various

*Received by the editors October 30, 2011; Accepted February 17, 2013.
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nonlinear static analyses are used, each of which contains a constant load pattern. These approaches are pushover analyses with load patterns proportionate to uniform and reverse triangular displacements of structures, and modal pushover analysis.

2. RESEARCH BACKGROUND

Numerous studies in the form of analytical and experimental works have been implemented on the mentioned structural systems. The most important results of which are as follows:

One of the most important points of composite structural systems is the connection between steel and concrete components of such systems. According to implemented tests, sufficient inner connectors such as shear studs can be used for uniform transfer of shear forces between steel frames and infilling walls and thus assure perfect composite action [12, 13].

The shaking table tests are the best method for simulation of behavior of the structure during earthquakes. But, this method is very expensive, especially for large scale models. The shaking table tests also implemented on steel moment frames infilled with light weight reinforced concrete walls have been reported. The tests contain the $\frac{1}{3}$ scale specimens having one bay and four-storey while linked to the reinforced concrete slabs [14].

A series of experimental programs including two-story specimens having larger scales in comparison to previous tests were developed to recognize the cyclic behavior of the composite structural systems. This study shows the lateral shear force tolerate via compressive strut of wall and shear studs [15].

Also, in another research about cyclic behavior of a composite structural system consisting of partially-restrained (PR) steel frames with reinforced concrete infill walls, it was found that this system has the potential to offer strength appropriate for resisting the forces from earthquakes and stiffness adequate for controlling drift for low- to moderate-rise buildings located in earthquake-prone regions [16].

Infills are commonly used in buildings for architectural reasons. It is proved that they have significant effects on both the strength and stiffness and they should not be ignored in the analysis and design of structures [17, 18]. Structural frames with infill panels typically provide an efficient method for bracing buildings [19]. The presence of infills can also have a significant effect on the energy dissipation capacity [20].

For the purpose of preliminary design and analysis of structures, many studies have been carried out to construct reduced nonlinear models that feature both accuracy and low computational cost. Miranda [21, 22] and Miranda and Reyes [23] have incorporated a simplified model of a building based on an equivalent continuum structure consisting of a series of flexural and shear cantilever beams to estimate deformation demands in multi-story buildings subjected to earthquakes. Although in that method the effect of non-linear behavior is considered by using some amplification factors, the flexural and shear cantilever beams can only behave in elastic range of vibration. Some researchers [24-26] have attempted to develop analytical models to predict the inelastic seismic response of reinforced concrete shear-wall buildings, including both the flexural and shear failure modes.

3. SEISMIC BEHAVIOR OF STRUCTURES

a) The ductility of structures

As a general rule it is possible to replace the ideal bilinear elasto-plastic diagrams with the base shear-displacement curves of structures (Fig. 1). The ductility factor in the SDOF systems is a proportion of maximum lateral displacement to the yielding lateral displacement of structures.

$$\mu = \frac{\Delta_{\max}}{\Delta y} \tag{1}$$

In fact, the ductility factor explains to what extent the structure enters the nonlinear state. There is no accurate definition for the ductility factor of MDOF structures. In some provisions, yielding is assumed to have been simultaneous, although not precise [27]. Meanwhile the relation between the base shear and displacement isn't an elastic-perfectly plastic equation. Taking Fig. 1 into consideration, an idealization in definition of the ductility factor is accepted.

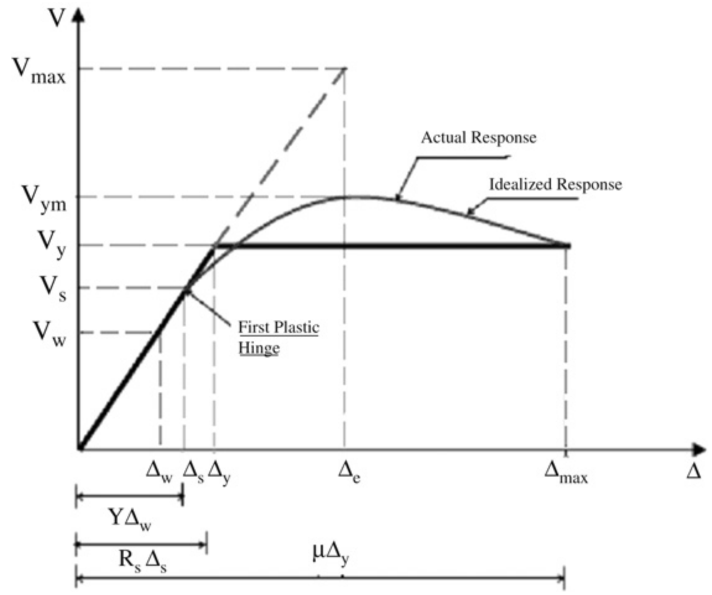

Fig. 1. General structure response [23]

b) Response modification factor

Seismic codes consider a reduction in design loads, taking advantage of the fact that the structures possess significant reserve strength (over-strength) and capacity to dissipate energy (ductility). The over-strength and the ductility are incorporated in structural design through a force reduction or a response modification factor. This factor represents ratio of maximum seismic force on a structure during specified ground motion if it was to remain elastic to the design seismic force. Thus, actual seismic forces are reduced by the factor "R" to obtain design forces. The basic flaw in code procedures is that they use linear methods but rely on nonlinear behavior [28].

As it was shown in Fig. 1, usually real nonlinear behavior is idealized by a bilinear elasto- perfectly plastic relation. The yield force of structure is shown by V_y and the yield displacement is Δ_y. In this figure V_e or V_{max} correspond to the elastic response strength of the structure. The maximum base shear in an elasto perfect behavior is V_y [29]. The ratio of maximum base shear considering elastic behavior V_e to maximum base shear in elasto perfect behavior V_y is called force reduction factor,

$$R_\mu = \frac{V_e}{V_y} \tag{2}$$

The over-strength factor is defined as the ratio of maximum base shear in actual behavior V_y to first significant yield strength in structure V_s,

$$R_S = \frac{V_y}{V_s} \tag{3}$$

To design for allowable stress method, the design codes decrease design loads from V_s to V_w. This decrease is done by allowable stress factor which is defined as [30]:

$$Y = \frac{V_S}{V_W} \tag{4}$$

The response modification factor, therefore accounts for the ductility and over-strength of the structure and the difference in the level of stresses considered in its design. It is generally expressed in the following form, taking into account the above mentioned conceptions [30],

$$R = \frac{V_e}{V_W} = \frac{V_e}{V_y} \times \frac{V_y}{V_S} \times \frac{V_S}{V_W} = R_\mu \times R_S \times Y \tag{5}$$

c) The relation between the force reduction factor, the ductility factor and the period of structure

The force reduction factor (R_μ) is related to several parameters many of which are correlated to characteristics of the structural system and some of them are independent from the structure and are related to the other parameters such as respected loading (the time history of earthquake). The R_μ will be correlated to a set of factors, especially the ductility factor of structure and its performance characteristics in the nonlinear state, if we consider a specific earthquake for a particular place. Therefore, the first step in determining the force reduction factor is specifying the relation between it and the capacity of the ductility of structure.

Multiple factors are known that affluence on the relation between R_μ and μ, such as materials, period of system, damping, P-Δ effects, the load-deformation model in the hysteresis loops and the type of soil that exists in the site. If we consider the assumption that the ductility in the structures with short period is the same as those that have longer periods, then the smaller R_μ is obtained.

Also, New Mark and Hall [31] suggested the following equations for calculation of the force reduction factor of structures:

$$R_\mu = 1 \qquad T < 0.125 \ Sec \tag{6}$$

$$R_\mu = \sqrt{2\mu - 1} \qquad 0.125 \ Sec < T < 0.5 \ Sec \tag{7}$$

$$R_\mu = \mu \qquad 0.5 \ Sec < T \tag{8}$$

d) The conversion coefficient of linear to nonlinear displacement (C_d)

It's clear that the structural damages are normally originated from excessive deformations of the structure. Therefore, regarding the effective parameters on seismic design of a structure, the discussion about assessment and accurate prediction of displacement and monitoring of them are the most important aims in seismic design of a structure. The C_d coefficient can be calculated as follows:

$$C_d = \frac{\Delta_{\max}}{\Delta_S} = \frac{\Delta_{\max}}{\Delta_y} \times \frac{\Delta_y}{\Delta_s} = \mu \times R_s \tag{9}$$

4. DESIGN OF THE STRUCTURAL MODELS

In this study, two structural models are used for specifying the trend of this research and are defined as follows:

(1) A 10-storey building in the form of steel moment resisting frame accompanied with reinforced concrete shear wall, (Model A).

(2) A 20-storey building in the form of steel moment resisting frame accompanied with reinforced concrete shear wall, (Model B).

The height of all the stories is 3.5 m. Both of them have a residential application. Therefore, a floor dead load, an equivalent partition load and a live load for each story are applied 200, 650, 150 kg/m^2 respectively. Also, the structural system of the floor is a composite of reinforced concrete slabs and steel secondary beams. The steel material used in the sections of the structural members is of ST37 type with yielding strength of 2400 kg/cm^2 and ultimate strength of 3700 kg/cm^2. The compressive strength of concrete material, f'c, used in the shear walls is 300 kg/cm^2. American Institute of Steel Construction Specification (AISC-ASD 2005) [32] and American Concrete Institute Requirements (ACI 318-05) [33] were used to design steel members and shear wall respectively. In order to calculate earthquake load, the spectrum dynamic method was used based on reference Standard No. 2800-05 [34]. The equation suggested by Kheyroddin was used to determine the thickness and the number of required shear walls [35].

$$\rho_{\min} = \frac{\left(\frac{h_w}{l_w}\right)^2}{835 + 205\,\frac{h_w}{l_w}} \tag{10}$$

In which ρ_{min} is the minimum wall area to story area ratio, h_w is the total wall height and l_w is wall length (average shear wall lengths present in building plan).

The plans of the structures, the direction of the girders and secondary beams and the location of shear walls are shown in Fig. 2 and Fig. 3. In the design process of these structures, an attempt was made for moment frame members to tolerate 25 percent of earthquake forces in addition to bearing gravity load. The thicknesses of the shear walls for each storey are shown in Table 1 and Table 2. Regarding the design of the structures, box-shaped and I- shaped sections are obtained for the section area of columns and beams, respectively. Also, Table 3 indicates the some gained properties of these structures.

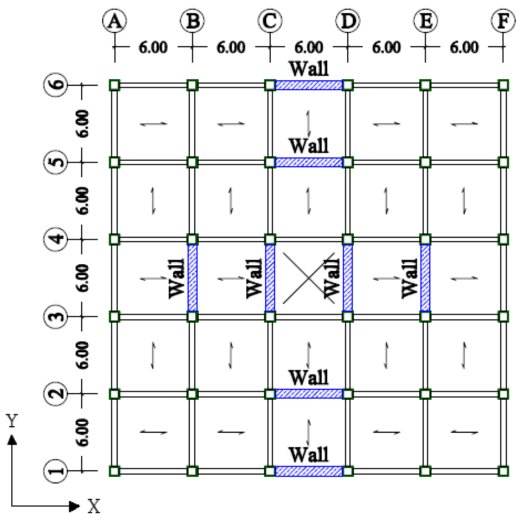

Fig. 2. The structural plan of the model A

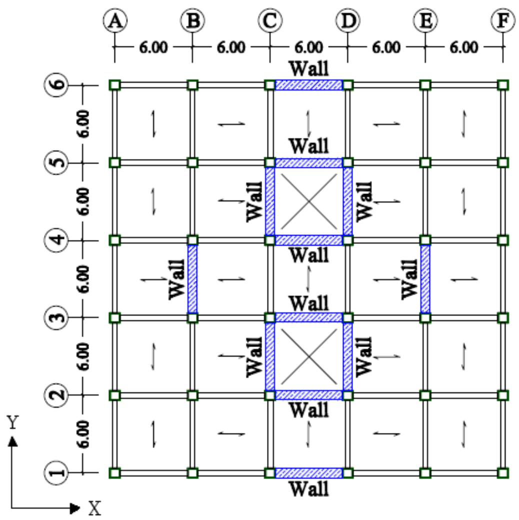

Fig. 3. The structural plan of the model B

Table 1. The thickness of the shear walls in model A

No. of the Storey	1-4	5-7	8-10
Thickness (cm)	35	30	20

Table 2. The thickness of the shear walls in model B

No. of the story	1-4	5-8	9-12	13-16	17-20
Thickness (cm)	50	45	40	30	20

Table 3. The structural properties of model A in linear design stage

Type of Model	T_1 (sec)	R	V (ton)	Δx (cm)	Δy (cm)	Max drift X	Max drift Y
Model A	1.215	8	800	6.98	9.09	0.0028	0.0036
Model B	1.915	8	1145	16.17	13.17	0.003	0.0024

T_1 is the natural period of the structure.
R is the response modification factor of the structure.
V is the base shear of the structure.
Δx and Δy are the displacements at roof level of the structure in X and Y direction respectively.
Max drift X and Max drift Y are the maximum drifts of the structure in X and Y direction respectively.

5. NONLINEAR ANALYSIS

a) Existing pushover analysis methods

During a pushover analysis, a frame structure is subjected to gravitational loads and horizontal loads applied at each storey, with the latter being incremented up to failure. Ideally, the distribution of horizontal loads should approximate the inertia forces that are generated in the structure during an earthquake. Conventional pushover procedures adopt an invariant load pattern during the analysis, and according to a number of codes and guidelines, at least two different force distributions must be considered; uniform and proportional to the first modal shape. The invariant load pattern is one of the most significant limitations of traditional methods, because the actual inertia force distribution changes continuously during seismic events due to higher mode contribution and structural degradation, which modifies the stiffness of individual structural elements and consequently of the structure as a whole [36].

A procedure proposed by Chopra and Goel [37], is the Modal Pushover Analysis (MPA), whereby a series of independent pushover analyses are carried out, considering different horizontal load patterns for each modal shape. According to the authors, it is sufficient to consider the first two or three modal shapes. Results in terms of capacity curves for various modal shapes are transformed in capacity curves for equivalent SDOFs, one for each mode. Seismic demands are separately evaluated for each SDOF and finally combined by the SRSS method. A common drawback of this method is the mode superposition of results obtained from nonlinear pushover analyses carried out separately, for various modes. The method neglects the interaction amongst the modes, with modal superposition being performed just as in elastic modal analysis. Accordingly, capacity curves typically overestimate base shear values [38].

b) Modeling

In order to assess the seismic behavior of selected buildings we have conducted a series of nonlinear static analyses. After a preliminary design of the structures, the nonlinear model of the following elements, force-deformation relationship and deformation capacities has been developed. There are a number of different ways to model inelastic beams and columns. At one extreme are finite element models using fiber sections. At the other are chord rotation models that consider the member as a whole and essentially require one to specify only the relationship between end moment and end rotation. In between these extremes are a number of other models. In this study the chord rotation model for beams and columns has been selected. The basic model is shown in Fig. 4. This is a symmetrical beam with equal and opposite end moments and no loads along the beam length. To use this model one has to specify the nonlinear relationship between the end moment and end rotation. An advantage of this model is that FEMA-356 gives specific properties, including end rotation capacities [39].

The F-D relationship shown in Fig. 5 is utilized for all beams and columns components. For beam components, F is end moment and D is end rotation. For column elements, F is force and D is axial displacement and end rotation. Also, for modeling beams and columns, the FEMA beam and column components are used in both models. Hence, the modeling parameters used in the F-D relationship are accessible in FEMA-356.

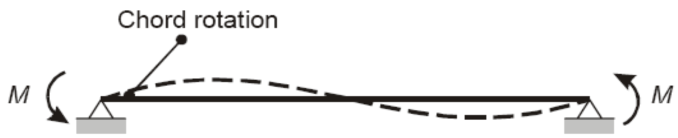

Fig. 4. Chord rotation model [39]

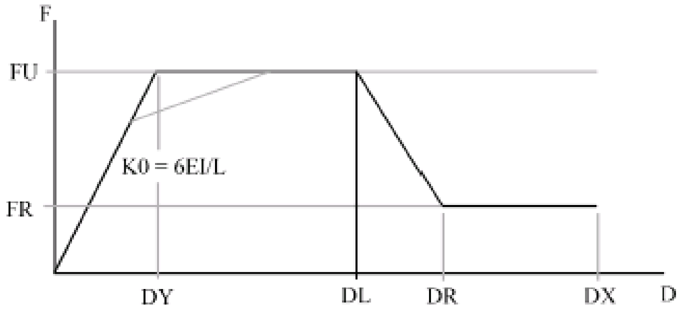

Fig. 5. Force-deformation relationship [39]

To make the reinforced concrete shear wall sections, defining the linear and nonlinear characteristics of its materials (concrete and steel bar) are necessary.

As it is shown in Fig. 6, the stress-strain curve of concrete is selected in the form of trilinear with strain hardening; and its tension strength is ignored. To calculate the modulus of elasticity of concrete, the following equation is used.

$$E_C = 15100 \sqrt{f'_C} \tag{11}$$

Hence, the modulus of elasticity, E_C, is assumed to be 261540 kg/cm^2.

The strain of ultimate strength of concrete, ε_l, is taken 0.003 [34], and the strain of crushing limit of concrete, ε_{cu}, is taken 0.005. As it is seen in Fig. 6, the strain of yielding strength of concrete, ε_y, is taken 0.002; and then the ratio of initial modulus of elasticity to secondary modulus of elasticity is specified to 0.402.

As it is shown in Fig. 7, the stress-strain relationship of steel bar is supposed to be bilinear (elastic-perfectly plastic). The modulus of elasticity, E_s, is taken 2100000 kg/cm^2 and the ultimate strain, ε_{su}, is taken 0.05. Also, its yielding strength, F_y, is 4000 kg/cm^2.

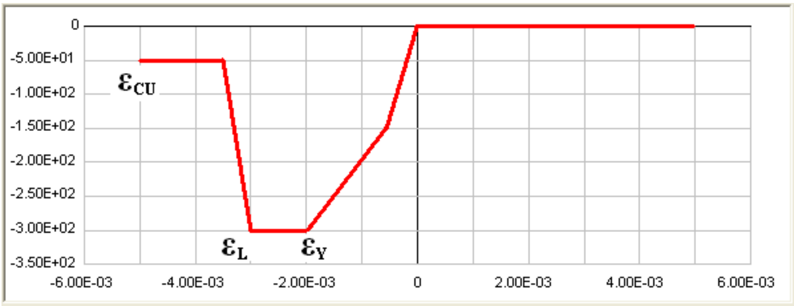

Fig. 6. Nonlinear properties of concrete material

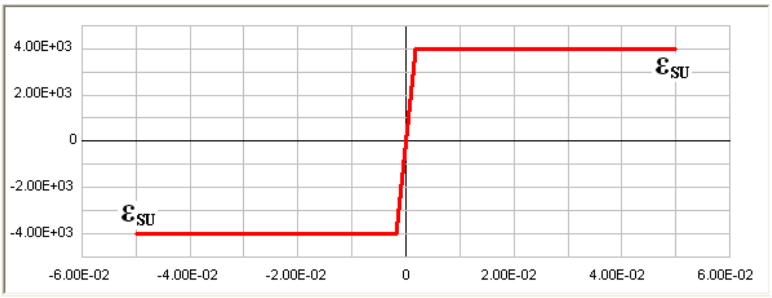

Fig. 7. Nonlinear properties of steel bar

c) Nonlinear analysis of the models

In this research, three nonlinear static analysis approaches are used for each model, which are described in the following. So 9 pushover analyses have been performed. The center of mass at the roof level is selected as a control point of the displacement of structure in all analysis. Since the relative lateral displacement (drift) of roof is used as a reference relative lateral displacement, for platting the capacity curves of the structures and for interpretation of the results obtained from these analyses.

During application of these analyses, two approaches have been used to regulate the drift of structure. The first criterion is the limitation of reference drift and inter-story drift for the structure, which is 2% based on Table C1-3 of FEMA-356 and Standard No.2800 [34]. Consequently, the analysis will be stopped when these drifts exceed from the mentioned limit. The second criterion for finishing the analysis is when the deformation capacity of each element is reached.

1. Uniform nonlinear static procedure (UNSP): To perform a static pushover analysis you must specify the distribution of horizontal loads over the structure height.

One of the most difficult issues for push-over analysis is choosing the push-over load distribution. During an actual earthquake, the effective loads on a structure change continuously in magnitude, distribution and direction. The distribution of story shears over the height of a building can thus change substantially with time, especially for taller buildings where higher modes of vibration can have significant effects. In a static pushover analysis the distribution and direction of the loads are fixed, and only the magnitude varies. Hence, the distribution of story shears stays constant. To account for different story shear distributions it is necessary to consider a number of different push-over load distributions. One option in FEMA-356 is to use uniform and triangular distributions over the building height. Note that a uniform distribution usually corresponds to a uniform acceleration over the building height, so that the load at any floor level is proportional to the mass at the floor.

2. Triangular nonlinear static procedure (TNSP): The difference between this procedure and the previous one is in their load pattern, while in this procedure the inverted triangular profile is used for displacement based load pattern of storey masses, based on FEMA-356. Therefore, the imposed displacement and hence the acceleration will not be uniform over the building height.

3. Modal pushover analysis (MPA): Load distributions can be based on the structure mode shapes. For a low-rise structure that is dominated by its first mode response, a load distribution based on the first mode may be reasonable. Also, considering the higher modes is important for a structure with significant higher mode responses.

In this study, the three first mode shapes in the X-direction of structural plan (see Fig. 2 and Fig. 3) are selected to perform modal pushover analyses.

The capacity curves of the Models A and B which are obtained from mentioned pushover procedures are shown in Figs. 8 and 10. The ideal bilinear diagrams of the curves are shown in Figs. 9 and 11 respectively. Also, the values of V_S, V_Y, V_U, Δ_Y and Δ_U which are obtained from the analyses and the value of V_W which is specified in preliminary design stage, are represented in Table 4. Therefore, seismic parameters have been calculated by using the equations which are defined in the section 3, as it indicated in Table 5. Also, the average of the above values that are shown in the following tables are determined.

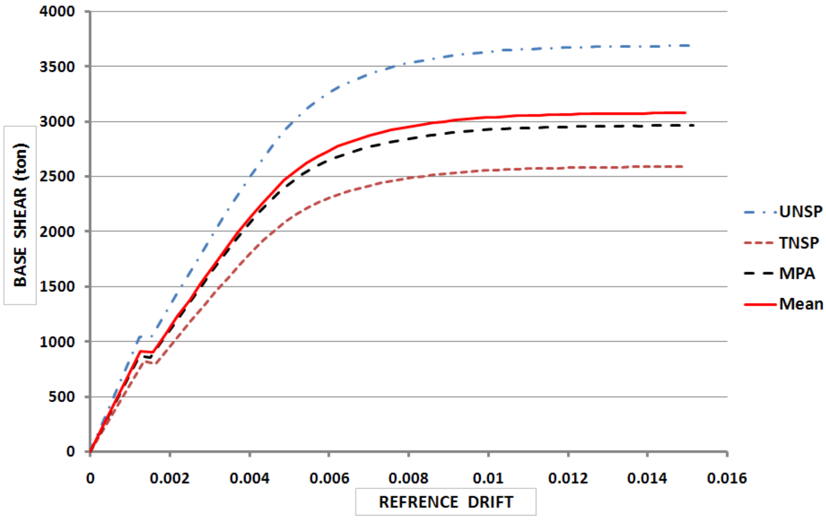

Fig. 8. The capacity curves of various pushover procedures for model A

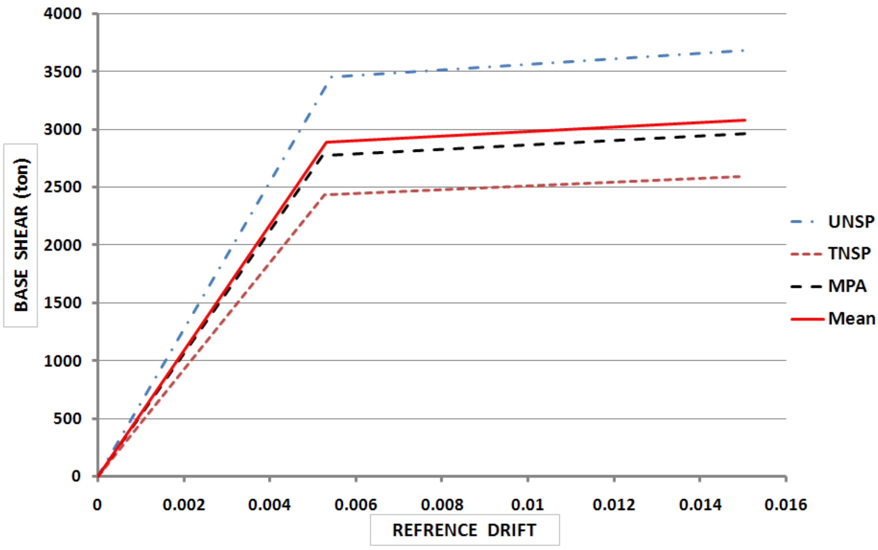

Fig. 9. The ideal bilinear diagrams of the pushover analyses for model A

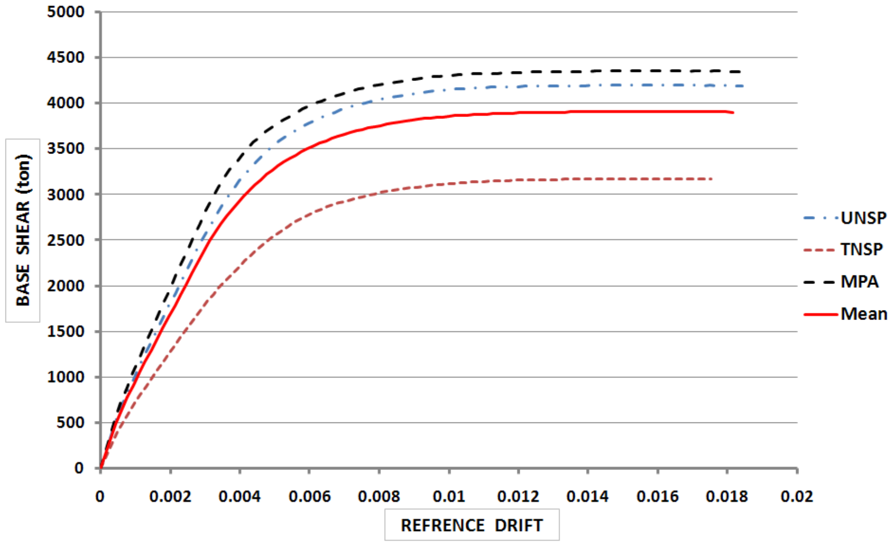

Fig. 10. The capacity curves of various pushover procedures for model B

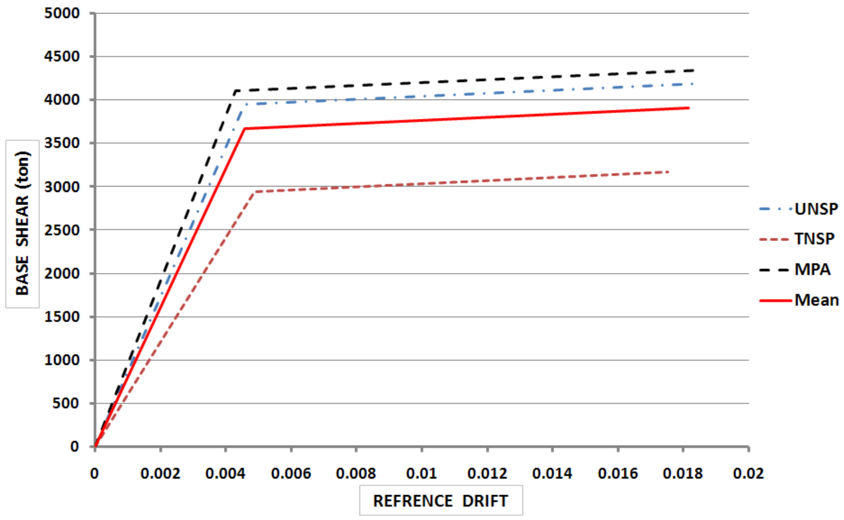

Fig. 11. The ideal bilinear diagrams of the pushover analyses for model B

Table 4. The structural properties of models A and B in nonlinear analysis stage

Type of model	Type of analysis	V_W (ton)	V_S (ton)	V_Y (ton)	V_U (ton)	Δ_Y	Δ_U
Model A	UNSP	800	1440	3455	3690	0.00542	0.0151
	TNSP		1070	2435	2596	0.00527	0.0149
	MPA		1177	2780	2967	0.00522	0.0151
	Mean		1229	2890	3084	0.0053	0.01503
Model B	UNSP	1145	2429	3954	4193	0.00456	0.01843
	TNSP		1688	2940	3171	0.00487	0.01753
	MPA		2690	4110	4351	0.00431	0.01854
	Mean		2269	3668	3905	0.00458	0.01816

Table 5. The seismic parameters of models A and B

Type of model	Type of analysis	μ , R_μ	R_S	Y	R	C_d
Model A	UNSP	2.78	2.4	1.8	12	6.67
	TNSP	2.83	2.28	1.34	8.65	6.45
	MPA	2.9	2.36	1.46	10	6.84
	Mean	2.84	2.35	1.54	10.28	6.67
Model B	UNSP	4.03	1.63	2.12	13.93	6.57
	TNSP	3.6	1.74	1.47	9.2	6.26
	MPA	4.3	1.53	2.35	15.46	6.58
	Mean	3.96	1.62	1.98	12.7	6.42

As it observed from analytical results, the parameters obtained from modal pushover analysis such as ductility factor (μ), force reduction factor (R_μ) and response modification factor (R) for the mentioned structures have been increased as the structure height increased and will be greater than the same parameters obtained from other pushover approaches. This behavior can reveal the impact of increasing the number of stories and hence the effect of higher mode shapes of the structure.

6. CONCLUSION

Some of the key results obtained from the present analytical work are as summarized as:

- The mean value of the μ and R_μ factors for 10-story structure (Model A) is 2.84, whereas the value of mentioned factors is 3.96 for 20-story structure (Model B).
- The mean value of the over-strength factor, R_S, for Model A and Model B are 2.35 and 1.62, respectively.
- The mean value of the response modification factor, R, for Model A and Model B in allowable stress design method are evaluated as 10.28 and 12.7, respectively. These values are greater than the same factors for both of the above structural systems in Standard No.2800.
- The mean value of the increasing coefficient of linear to nonlinear displacement, C_d, for Model A and Model B is evaluated as 6.67 and 6.42, respectively.
- Analytical results show that the ductility factor and the response modification factor increased as the structure height increased. However, the over-strength factors increased as the structure height decreased.
- With regard to the results, it seems that the C_d factor for the mentioned structural systems is more than the values which are in Standard No.2800. The C_d factor is suggested as 0.7 times the response modification factor, R, in this code.

REFERENCES

1. Memarzadeh, P., Saadatpour, M. M. & Azhari, M. (2010). Nonlinear dynamic response and ductility requirements of a typical steel plate shear wall subjected to El Centro earthquake. *Iranian Journal of Science and Technology, Transaction B: Engineering*, Vol. 34, No. B4.
2. Kaplan, H., Gönen, H., Nohutcu, H., Çetinkaya N. & Yilmaz, S. (2009). A new strong floor-reaction wall system without gallery for experimental studies in structural mechanics, *Iranian Journal of Science and Technology, Transaction B: Engineering*, Vol. 33, No. B4.

3. Kheyroddin A., Naderpour, H., Ghodrati Amiri, G. & Hoseini Vaez, S. R. (2011). Influence of carbon fiber reinforced polymers on upgrading shear behavior of RC coupling beams. *Iranian Journal of Science & Technology*, *Transactions of Civil Engineering*, Vol. 35, No. C2.
4. Kheyroddin A. & Naderpour, H. (2008). Nonlinear finite element analysis of composite RC shear walls. *Iranian Journal of Science and Technology, Transaction B: Engineering*, Vol. 32, No. B2.
5. Seren Akavci, S. (2007). Nonlinear analysis of semi-rigid frames with rigid end sections. *Iranian Journal of Science and Technology*, *Transaction B: Engineering*, Vol. 31, No. B5.
6. Arslan, H. M., Aksogan, O. & Choo, B. S. (2004). Free vibrations of flexibly connected elastically supported stiffened coupled shear walls with stepwise changes in width. *Iranian Journal of Science and Technology*, *Transaction B: Engineering*, Volume 28, No. B5.
7. Hajirasouliha. I. & Doostan. A. (2010). A simplified model for seismic response prediction of concentrically braced frames. *Advances in Engineering Software*, Vol. 41, pp. 497–505.
8. Comité Europeen de Normalization, (2005). *Eurocode 8-design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings*. CEN. Brussels (Belgium).
9. Applied technology council, ATC-40, (1996). *Seismic evaluation and retrofit of concrete buildings*. ATC. Redwood City (USA).
10. Federal emergency management agency, FEMA-356, (2000). *Prestandard and commentary for the seismic rehabilitation of buildings*. FEMA, Washington (USA).
11. Ferracuti. B., Pinho. R., Savoia. M. & Francia. R., (2009). Verification of displacement-based adaptive pushover through multi-ground motion incremental dynamic analyses. *Engineering Structures*, Vol. 31, pp. 1789-1799.
12. Makino. M., Kawano. A., Kurobane. Y., Saisho. M. & Yoshinaga. K. (1980). An investigation for the design of framed structures with infill walls. *Proceedings of the Seventh World Conference on Earthquake Engineering*, Vol. 4, pp. 369–372.
13. Makino. M. (1985). Design of framed steel structures with infilled reinforced concrete walls. In: *Roeder CW, Editor. Composite and Mixed Construction*. New York (NY): ASCE; pp. 279–287.
14. Kwan. A. K. H. & Xia. J. Q. (1995). Shake-table tests of large-scale shear wall and infilled frame models. *Proceedings of the Institution of Civil Engineers: Structures and Buildings*, Vol. 110, No 1, p. 66–77.
15. Tong, X. (2001). Seismic behavior of composite steel frame–reinforced concrete infill wall structural system. *Ph.D. Thesis*, Department of Civil Engineering, University of Minnesota, Minneapolis.
16. Tong, X., Hajjar, J. F., Schultz, A. E. & Shield, C. K. (2005). Cyclic behavior of steel frame structures with composite reinforced concrete infill walls and partially-restrained connections. *Journal of Constructional Steel Research*, Vol. 61, pp. 531–552.
17. Abdel El Razik. M., Asran, A. & Abdel Hafiz. A. (2006). Effect of infill walls on the performance of multi-storey building during earthquakes. *1st International Structural Specialty Conference*.
18. FEMA 306, (1998). Evaluation of earthquake damaged concrete and masonry wall buildings. *Applied technology council*, ATC-43 Project.
19. Naderpour, H., Kheyroddin A. & Ghodrati Amiri, G. (2010). Prediction of FRP-confined compressive strength of concrete using artificial neural networks. *Composite Structures (Elsevier)*, IF=2.007, Vol. 92, pp. 2817–2829.
20. Decanini. L. D., Liberatore. L. & Mollaioli. F. (2002). Response of bare and infilled RC frames under the effect of horizontal and vertical seismic excitation. *12th European Conference on Earthquake Engineering*.
21. Miranda. E. (1997). Estimation of maximum inter-story drift demands in displacement based besign. In: Fajfar P, Krawinkler H, editors, *Proc. workshop on seismic design methodologies for the next generation of codes*, Rotterdam: Balkema, pp. 253–64.
22. Miranda. E. (1999). Approximate seismic lateral deformation demands in multistory buildings. *J Struct Eng*, Vol. 125, No. 4, pp. 417–25.

23. Miranda. E. & Reyes. C. J. (2002). Approximate lateral drift demands in multistory buildings with non-uniform stiffness. *J Struct Eng*, Vol. 128, No. 7, pp. 840–9.
24. Fajfar. P. & Gaspersic. P. (1996). The N2 method for the seismic damage analysis of RC buildings. *Earthquake Eng Struct Dynam*, Vol. 25, pp. 31–46.
25. Saiidi. M. & Sozen. M. A. (1981). Simple nonlinear seismic analysis of RC structures. *J. Struct. Div.* ASCE, Vol. 107, No. 5, pp. 937–52.
26. Hidalgo. P. A., Jordan. R. M. & Martinez. M. P. (2002). An analytical model to predict the inelastic seismic behavior of shear-wall, reinforced concrete structures. *Eng. Struct.*, Vol. 24, No. 1, pp. 85–98.
27. Wakabayashi, M. (1986). *Design of earthquake-resistant buildings*. Mc Graw-Hill.
28. Kim. J. & Choi. H. (2005). Response modification factors of chevron-braced frames. *Engineering Structures*, Vol. 27, pp. 285–300.
29. Uang, C. (1991). Establishing R (or R_w) and C_d factors for building seismic provisions. *Jornal of Structural Engineering*, ASCE, Vol. 117, No. 1, pp. 19-28.
30. Asgarian. B. & Shokrgozar, H. R. (2009). BRBF response modification factor. *Journal of Constructional Steel Research*, Vol. 65, pp. 290-298.
31. New Mark. N. M. & Hall. W. J. (1982). *Earthquake spectra and design*. Engineering Monograph Earthquake Engineering Research Institute, Berkeley, California.
32. American Institute for Steel Construction (AISC), (2005). *Specification for structural steel buildings, ANSI/AISC 360-05*. Chicago (IL): *American Institute for Steel Construction.*
33. American Concrete Institute, (2005). *Building code requirements for structural concrete* (ACI 318-05).
34. Building and Housing Research Center, (2007). *Iranian code of practice for seismic resistant design of buildings [Standard no.2800, 3th edition]*, Tehran, Iran.
35. Kheyroddin. A. (2006). *Analysis and design of shear walls*. Semnan University.
36. Ferracuti. B., Pinho, R., Savoia. M. & Francia. R. (2009). Verification of displacement-based adaptive pushover through multi-ground motion incremental dynamic analyses. *Engineering Structures*, Vol. 31, pp. 1789-1799.
37. Chopra. A. K. & Goel. R. K. (2003). A modal pushover analysis procedure for estimating seismic demands for buildings. *Earthq Eng Struct Dyn*, Vol. 31, pp. 561-82.
38. Hernandez-Montes. E., Kwon. O. S. & Aschheim. M. A. (2004). An energy-based formulation for first- and multiple-mode nonlinear static (pushover) analyses. *J Earthq Eng*, Vol. 8, No. 1, pp. 69-88.
39. Mohammadjafari, A. & Jalali, A. (2009). Assessment of performance based parameters in near fault tall buildings. *Journal of Applied Sciences*, Vol. 9, No. 22, pp. 4044-4049.