SEISMIC EVALUATION OF STEEL DUAL SYSTEMS AGAINST ARTIFICIAL EARTHQUAKES

S.MojtabaALIAN AMIRI
Ms.c. Student, Semnan University, Semnan, Iran
mojtabaalian@yahoo.com

ElmiraSHOUSHTARI REZAVNI
Ms.c Student, Amirkabir University of Technology, Tehran, Iran
elmira.shoushtari@aut.ac.ir

RezaVAHDANI
Assisstant Professor, Semnan University, Semnan, Iran
reza.vahdani2001@gmail.com

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ABSTRACT

Dual systems in form of steel moment frames accompanied with chevron bracings are among commonly used systems in high seismicity regions in the world. Therefore, the recognition of their seismic behaviour is of high importance. Except a few parts of the world that have sufficient earthquake records, artificial accelerograms can be used in other regions. One method for generating artificial accelerograms is using wavelet transform. In this paper, 5-, 10-, 15- and 20-story structures are designed according to ASCE7 requirements. Spectra matching process is performed for 44 earthquake records. Pushover and nonlinear time history analysis is done in order to investigate seismic behaviour of this system against real and artificial earthquake records. It can be concluded that responses against artificial records has far less dispersion, while the average responses in these two sets of records are approximately the same.Incremental dynamic analysis is done, and adjusted collapse margin ratios are calculated, in order to check whether frames can meet the FEMA-P695 criterion. Fragility curves are obtained to know the probability of collapse for these earthquakes.

INTRODUCTION

Special concentrically braced frame systems (SCBF) allow yield and buckling of braced frames, and yield of joints for energy dissipation. SCBF's are considered as one of the most widely used bracing systems in seismic regions. Structural engineers have shown greater tendency to utilize these systems in the last decades as special moment resisting frames systems (SMRF) proved inefficient in Northridge earthquake of 1994. Prior to this earthquake, SMRF systems were considered as one the best structural systems for seismic regions. But the above mentioned earthquake as well as other recent earthquakes created brittle fractures in the joints of these systems, ultimately breaking experts' trust in these system. SCBF systems are intrinsically hard systems that can absorb seismic waves by structural and non-structural formations that are significantly small. As a result, considerable volume of research has been conducted on SCBF system with the main objective of system improvement and joint efficiency.

Requirements for dynamic analysis in special cases such as design irregularities or uneven distribution of mass and stiffness in the height of buildings are mentioned in most of seismic regulations. Due to its simplicity and the special attention paid by guidelines to designing spectrums, the response spectral analysis method is widely used in most of the linear analyses conducted for structural designing. However, the method suffers from its inability to provide information on the time from the response of the structure.

The final design of important structures such as nuclear plants, dams, high-rise structures, and suspension bridges is executed based on the linear and nonlinear time history analysis. To perform this analysis, accelerograms are needed for earth motions in the construction site of these structures. The Iranian Strong Motion Network was established in 1973; thus, accessibility to the seismic history of different regions of the country for the long past years is difficult. Upon inaccessibility of accelerograms, one solution can be finding another region sharing the same physical properties as those of the construction site and using the captured accelerograms of that region directly. Identification of such similarities with similar properties proves quite difficult. Considering the exclusive characteristics of seismic forces in structural design, no two earthquakes are similar; therefore, utilization of this method is of great difficulty and complexity.

Apart from specific regions in the world where proper accelerograms have been captured, artificial accelerograms can be used for execution of this analysis. Utilization of real and modified accelerograms is recommended for artificial accelerograms to have reasonable and close-to-reality seismic parameters. The required modification is carried out through synchronization of the response spectrum of the real accelerograms with spectrum of guidelines.

ARTIFICIAL EARTHQUAKES

Different methods have so far been recommended for producing artificial accelerograms. Considering the fact that accelerograms are intrinsically of two time and frequency domains, their production can be divided into three main types with regard to the utilized domain during production process: 1. Time domain methods, 2. Frequency domain methods, 3. Time-frequency domain methods among which utility of wavelet transformation during accelerogram production is the most significant. Wavelet transformation is a new mathematical transformation widely used in various science fields. It is of proper functionality for non-stationary phenomena in both time and frequency domains. Wavelet transformation method is used in this study for conformity of accelerogram response spectrum to regulatory standard design spectrum.

Transformations are appropriate tools for providing additional and hidden information on the main function. It is also easier to use the transformed function rather than the initial function. One of these transformations, in order for non-stationary phenomena analysis, is time window Fourier transformation on the basis of which, signals are separated into distinct parts using time windows. Then, Fourier analysis is done on each part on the assumption that all parts are stationary. Width of windows is fixed upon transformation of window Fourier. Thus, proper time and frequency information cannot be achieved simultaneously.

To resolve the issues window functions of different sizes are usedin order for assessment of different frequency domains of signal or main function. The preliminary function which is used as a window function is called the mother wavelet. The other window functions are obtained through scaling the mother wavelet and replacement along the main signal time. The rest of the operation resembles time window Fourier transformation method. This transformation is called wavelet transformation as they are, like waves, a frequency function of time and place and concentrates their energy at a certain time and their average must be 0 as per their definition.

The smaller the proportion of the window, the more concentrated and more revealed the main function details will be. In the analysis of low frequencies, wavelet transformation uses windows of more width for a better time display; while slimmer windows are used in high frequencies for the same purpose. According to what has been discussed thus far, wavelet transformation is a powerful and proper tool for analysis of time variable non-stationary phenomena and shows the function properties both in time and frequency domains.

It is vital to choose an appropriate wavelet for achieving a successful wavelet transformation. Many researchers have attempted to simulate artificial earthquakes. A new wavelet was recommended by Suarez and Montejo (2005) based on impulse response of an oscillator with a sub critical damping whose mother wavelet is as follows:

$$\psi(t) = e^{-\xi\Omega|t|} \sin \Omega t \tag{1}$$

Where ξ and Ω are parameters indicating the time decrease and changes of the wavelet. They can be considered respectively as the damping ratio and natural frequency of a single degree of freedom oscillator. The wavelet should also be defined for $t < 0$ while it tends toward 0 in $t \to \pm\infty$. Thus, the absolute value of t is used which leads to the asymmetric wavelet depicted in figure 1.

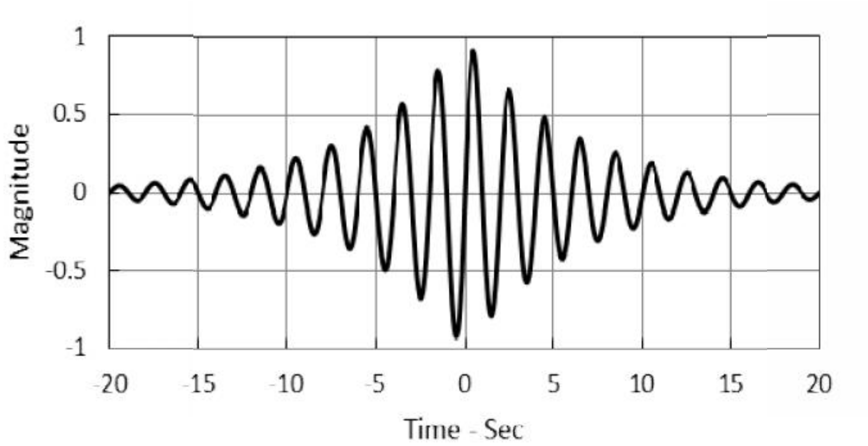

Figure 1. Impulse Response for $\Omega = \pi$ and $\xi = 0.005$

DESIGN AND MODELLING

Four structures (5-, 10-, 15-, and 20-story) were designed in ETABS as 3D structures. The structures were modelled with symmetric designs and in 5 spans of 5 meters and story height of 3.2 meters. Special moment resisting steel frames and braced frames (Chevron) served as their lateral force resisting system. The frames were designed by LRFD method and as per AISC360-05regulations. A36 steel with the yield strength of $f_y = 2530\ \mathrm{kgf/cm^2}$ and ultimate strength of $f_u = 4080\ \mathrm{kgf/cm^2}$was used for the design of beam and column steel parts, and A500 grade B steel with the yield strength of $f_y = 3230\ \mathrm{kgf/cm^2}$ and ultimate strength of $f_u = 4080\ \mathrm{kgf/cm^2}$ was used for the bracing parts. The dead and live load of the stories were $550\ \mathrm{kg/m^2}$ and $200\ \mathrm{kg/m^2}$ respectively; and those of the roof were $500\ \mathrm{kg/m^2}$ and $150\ \mathrm{kg/m^2}$ respectively. Also, the design spectrum of ASCE7 regulations was utilized for dynamic analysis and lateral loading. The structures have been assumed to be of groups 1 and 2 (the structures of no special application) with a significance coefficient of 1. Figure 2 shows the plan of structures, the frame in question, and the design of frames.

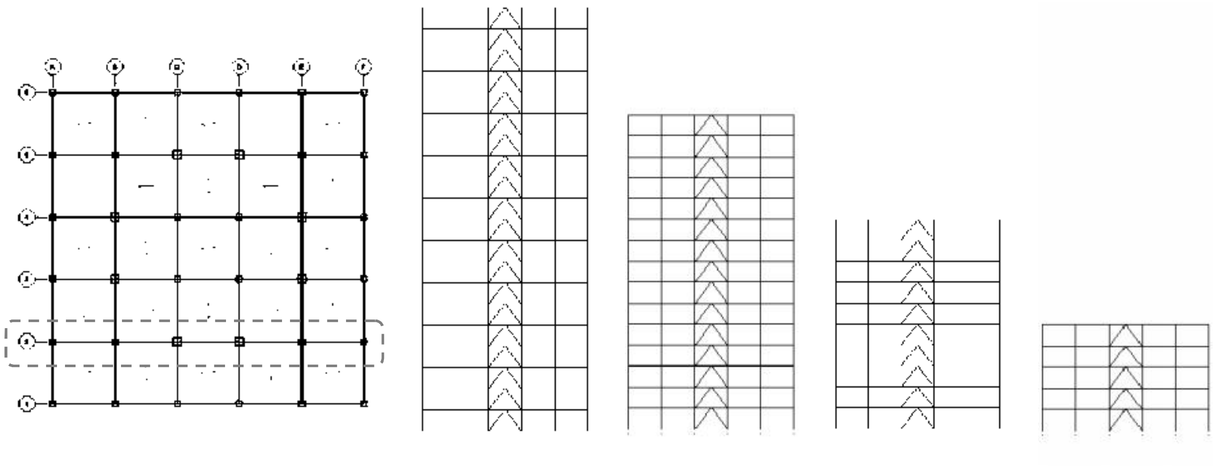

Figure 2. Plan of structures, and the design of frames selected for modelling in the software

In addition to strength aspects of the structures, elements such as story drift and weak-beam/strong-column principle have been taken into consideration as per regulatory constraints. Also, the braced frames are able to absorb 25 percent of earthquake force. According to the note 1-4-9-12 of ASCE7-05, the base shear calculated through spectral method should not be lower than 90 percent of the static base shear; otherwise, the spectral base shear must be modified.

Considering table 1-12 in ASCE7-05, the ductility, over-strength factor, and Displacement Amplification Factor coefficients were obtained as shown in table 1.

Table 1. Design-related coefficients

Ductility Coefficient - R	Over-strength Factor - Ω	Displacement Amplification Factor - C_d
7	2.5	5.5

Soil type is chosen to be D, since according to ASCE7 regulations, for circumstances that no information is available on the properties of the soils of the region. The maximum values of spectral acceleration for SDC D ($S_s = 1.5g$ and $S_1 = 0.6g$) is determined based on the effective border between probabilistic and deterministic areas (close areas) of MCE earthquake (According to the note 21.2 of ASCE). F_a and F_v coefficients are obtained as per the soil type and from tables 1-4-11 and 2-4-11 of ASCE. S_{MS} and S_{M1} are the maximum spectral acceleration from the maximum earthquake spectrum (MCE) in short periods and 1 second period respectively. S_{DS} and S_{D1} represent the same factors for designing earthquake (DBE) which is obtained by dividing S_{MS} and S_{M1} by 1.5 (table 2).

Table 2.Calculated Seismic Load Coefficients

F_a	F_v	S_s	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}
1	1.5	1.5	0.6	1.5	0.9	1	0.6

OpenSees software was utilized in this study for structural analyses. This software is able to carry out various linear and nonlinear analyses statically and dynamically on steel and concrete structures. The below general points are worth to note with regard to modelling of structures:

- Modelling was carried out in 2D format and each node is of 3 degrees of freedom.
- All columns are fixed at the base.
- Structural mass was considered in seismic behaviour analysis as$1.05D + 0.25L$. Structural mass is only upon nodes.

To model this behaviour in the members, an elastic element with concentrated plasticity was used. The nonlinear behaviour was considered in the two ends of the member and the other parts of the member were considered as being elastic. Therefore, the members are modelled by elastic elements (elasticBeamColumn) and all their nonlinear behaviour at both ends of the member were modelled using Plastic Rotary Springs. Hinges are considered at half of the column dimension from the cross-section. The distance of the hinge from the joint has been modelled by rigid elements. Therefore, the structural properties of each element are a combination of subsidiary elements which are placed as a series.

Rotspring2dmodIKmodel was utilized for simulation of plastic braced hinges of beams and columns which considers an element with a length of 0 in which Bilin has been used. This element simulates the Ibarra and Krawinkler Deterioration Model (2005) modified by Lignos (2008) with Bilinear Hysteretic behaviour.

The reported damages imposed by near-field earthquakes on bracing frames and lack of sufficient information on modelling and behaviour analysis of these types of structures led Uriz (2008) to run long-term test projects on different types of bracing frames and calibrate the obtained results by different types of structural models.

Considering the above points, fibers with Steel02 or Giuffré-Menegotto-Pinto material were used in OpenSees software for modelling the bracing parts whose strain hardening was considered as 0.003 as per studies conducted by Uriz. Two elements were used along the bracing and 3 integration points were considered. Also, initial eccentricity in the middle of the bracing was considered as 0.005 of the effective length of bracing so the bracing buckling can be modelled in OpenSees. Bracing fracture was also considered by Fatigue material, introduced by Uriz. Gusset plate behaviour was also simulated using the model introduced by Hsiao (2012) and by rotary springs at the end of two elements of the bracing (Figure 3).

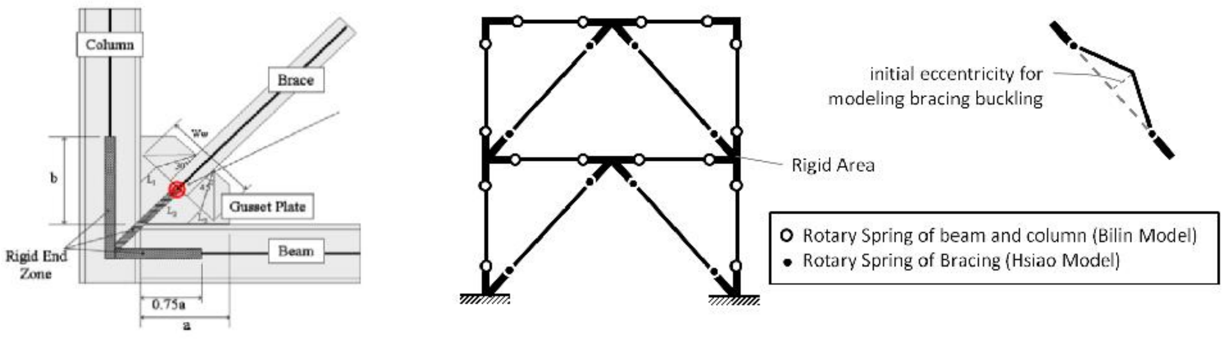

Figure 3. The bracing end hinge model introduced by Hsiao (2012)

RESULTS OF ANALYSES

Pushover analysis was conducted in this study to determine the maximum strength of the frame, ductility, over-strength factor, and frame failure mechanism. The frames' first modal shape pattern has been used in this diagram for lateral loading in the height.

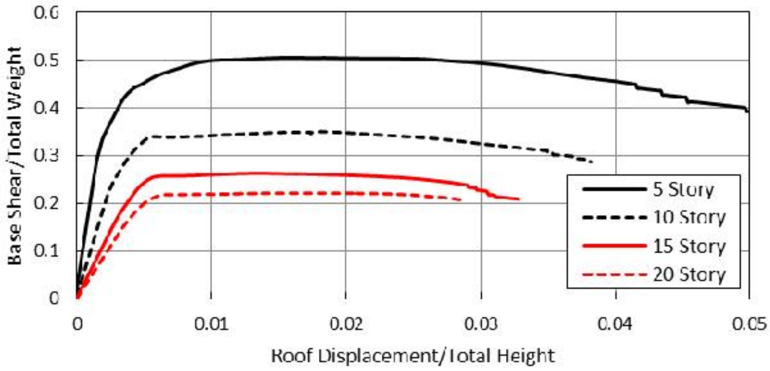

Figure 4. The nonlinear static analysis (pushover)

Ductility and over-strength factor values are obtained considering the pushover diagrams. These values are presented in table 3.

Table 3.Ductility and over-strength factor coefficients

	5-Story Frame	10-Story Frame	15-Story Frame	20-Story Frame
Ductility Coefficient	17.5	11.5	6.7	6.8
Over-strength Coefficient	1.9	2.0	1.8	2.1

As shown, ductility coefficients for 15- and 20-story frames are close to each other, and over-strength coefficients for all frames are almost equal.

The coefficient values for ductility and over-strength factor recommended by ASCE are 7 and 2.5 respectively; ductility coefficients obtained for 15- and 20-story frames and over-strength coefficients obtained for all frames are almost close to the values recommended by ASCE.

Time history analysis was also conducted for better understanding of frame behaviour under 44 far-field earthquakes proposed by FEMA-p695. The records once obtained naturally and normalized by the PGV medium were compared with those of DBE earthquake (earthquake with a 10% occurrence probability in 50 years). Once again the spectral fitting process was applied on these records and analyses were run under artificially induced earthquakes. Figure 5 shows the maximal distribution of interstory drift for 5-, 10-, 15-, and 20-story frames for different series of actual records, as well as the values of these responses obtained for artificial records. As illustrated, the dispersion of responses (standard deviation) under artificial earthquakes is almost half of that under real earthquakes, while the average values of responses for both modes are close to one another.

The main goal of performance evaluation is that collapse probability be reasonably small, but still rational. Performance of structures in this study is evaluated using FEMA_P695 methodology. In this method, collapse level ground motion is defined as an intensity which results in a median collapse. In other words, half of the structures exposed to this earthquake sustain kind of life-threatening damages.

According to this code, performance of a system is considered reasonable when its collapse probability is limited to a specific amount. Average of collapse probability for each group of structures should be limited to 10 percent, and for each individual model should be limited to 20 percent.

Probability collapse is calculated using CMR (Collapse Margin Ratio) which is the ratio of median collapse intensity (obtained by incremental dynamic analysis) to MCE spectral demand.

Collapse capacity and CMR calculation can be significantly influenced by frequency content (spectral shape) of ground motion record set. In order to take into account the effect of spectral shape, CMRs should be modified. ACMRs (Adjusted Collapse Margin Ratio) are obtained by multiplying CMRs into spectral shape factor (SSF).

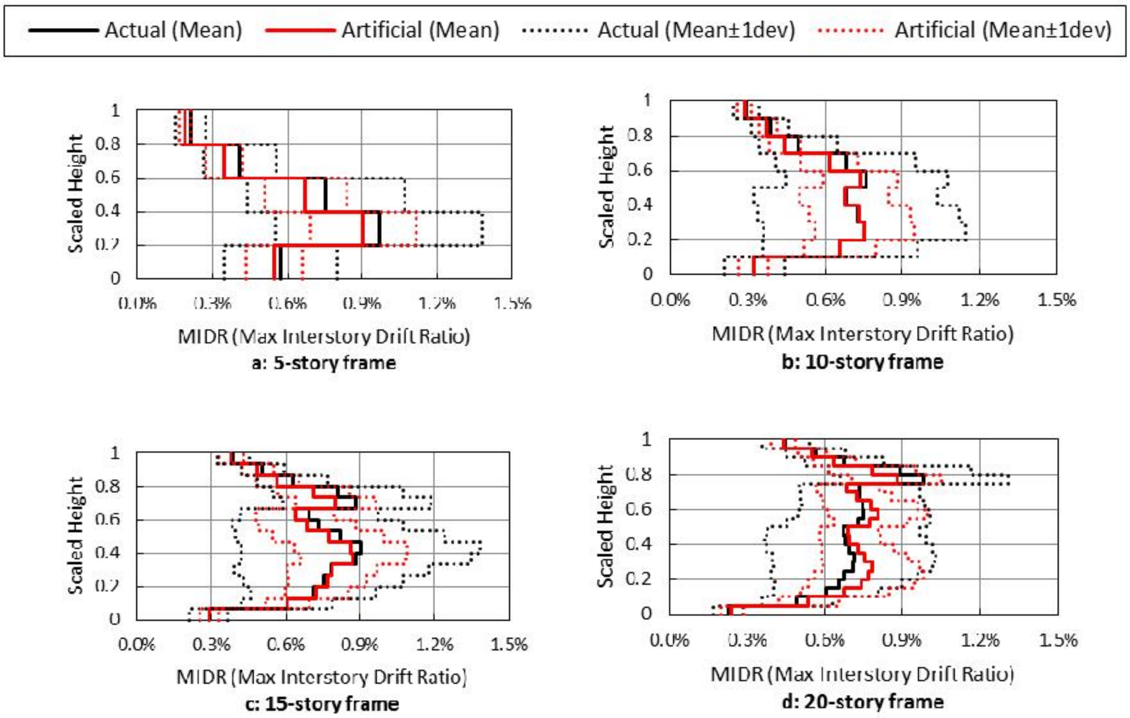

Figure 5. distribution of maximum interstory drift ratio (MIDR) in height

With respect to the time history analyses results, reduction in dispersion, and approximate adaption of average responses of frames of artificial earthquakes in comparison to real earthquakes, in order to save time fewer number of artificial earthquakes are used in IDA analysis.

Fragility curves using artificial earthquakes are shown for all frames in figure 6. MCE spectral acceleration for first mode period for each frame is shown on figures. It is noticed that probability collapse for 5-, 10-, 15- and 20-story frames is 8, 11, 14, and 16 percent respectively. Therefore, collapse probability is increased by increasing the height of structures.

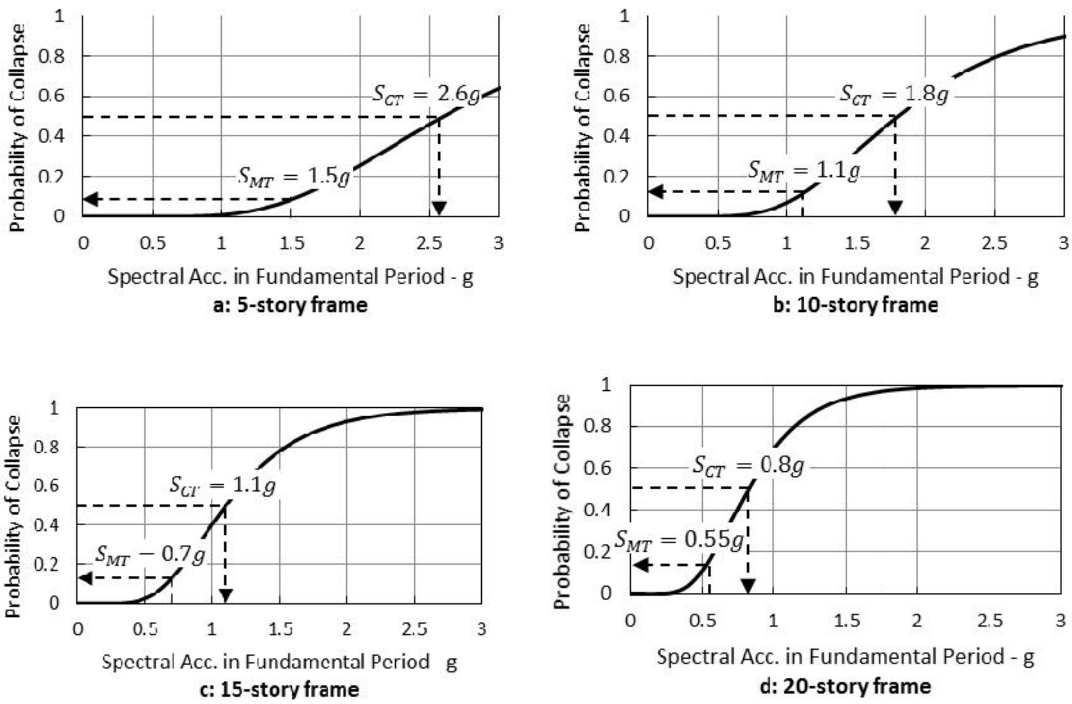

Figure 6. distribution of maximum interstory drift ratio (MIDR) in height

In order to obtain collapse capacity of structure, total system collapse uncertainty should be calculated. Four sources of uncertainty are evaluated using FEMA p695 methodology, and their effects are incorporated in the collapse assessment process. Record to record uncertainty (β_{RTR}), design requirements uncertainty (β_{DR}), data uncertainty (β_{TD}), and modelling uncertainty (β_{MDL}) are considered 0.4, 0.35, 0.2, and 0.35. Therefore, total uncertainty is equivalent to 0.65.ACMR values are obtained for these frames using the mentioned method and are shown in table 4.

Table 4.Adjusted collapse margin ratios

Story	SMT	SCT	CMR	SSF	ACMR	ACMR accp. (20%)	
5	1.5g	2.6g	1.67	1.33	2.32	1.73	PASS
10	1.1g	1.8g	1.63	1.41	2.30	1.73	PASS
15	0.7g	1.1g	1.57	1.46	2.29	1.73	PASS
20	0.5g	0.8g	1.50	1.51	2.27	1.73	PASS
			avg.ACMR		2.29	PASS	
			ACMR accp. (10%)		2.30		

CONCLUSION

Four frames (5-, 10-, 15-, and 20-Story) were modelled in OpenSees software; nonlinear static, nonlinear time history and incremental dynamic analyses were conducted on them. The obtained results were used for performance assessment and determining the weakness of the structures. The following points can be concluded:

Dominant period of actual and artificial records are respectively between 0.3 to 1.5 seconds, and between 0.51 to 0.91 seconds. Maximum velocities of actual and artificial records are between 12.8 to 115 cm/s, and between 41 to 94.2 cm/sand their maximum accelerations are between 0.15g to 0.82g, and 0.38g to 0.71g.

The pushover curve of the frames shows that the structure exhibits a relatively linear behaviour before buckling of bracings. But the structure strength deteriorates after bracing buckling which leads to failure by an increase in deformations and fractures in the bracings. Ductility coefficients obtained from nonlinear static analysis on 5-, 10-, 15- and 20-story frames were 17.5, 11.5, 6.7, and 6.8. The value recommended by regulations is 7 which is close to the value obtained for 15- and 20-story frames.

Over-strength coefficients were 1.9, 2, 1.8 and 2.1. The value recommended by the regulations is 2.5 which are close to the value obtained for 20-story frame. It seems that it is better to have different values of behaviour coefficient, over-strength coefficient and etc. for low-rise, mid-rise and high-rise systems. However, a fixed value is recommended by regulations for each of them.

The comparison of results obtained from time history analysis indicates that the maximal response average of frames for records transfers to lower stories by an increase in the number of stories. It can be justified by the fact that higher frames show greater sensitivity to P-delta effects which leads to a significant increase in the drift of lower stories. Thus, a modification in design principles is needed so that the formation of marginal mechanisms can be prevented (or at least postponed) in the lower stories of systems with great number of stories.

A review of time history analysis results under real and artificial earthquakes show that the reflection average values are close to one another in both modes and reflection distribution (standard deviation) in artificial earthquakes is almost half of that in real earthquakes. It can thus be concluded that utilization of artificial records for time history analysis for structures situated in regions which do not have suitable records will yield acceptable results.

According to the results, frames meet the criteria both individually or in a group. Additionally, with increasing the number of stories ACMR has decreased, which can be the result of P-Delta effects which are increased as the height of structure increases, and as a result can form partial mechanisms in lower stories which leads to sooner collapse of structures.

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