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Analysis of Different Performance Levels by Comparing Fragility Curves and Behavioral Parameters in Zipper Bracing Structures Under Increasing Dynamic Analysis

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Abstract

In this paper, the Analysis of different performance levels by comparing fragility curves and behavioral parameters in zipper bracing structures under increasing dynamic analysis. This adverse effect can be mitigated by adding zipper columns or vertical members connecting the intersection points of the braces above the first floor. This paper critically evaluates over strength, ductility and response modification factors of these structures3, 5 and 7 story structures were modeled and analyzed. Static pushover analysis, linear dynamic analysis and nonlinear incremental dynamic analysis are performed by OpenSees software concerning 15 records of past earthquakes. In this paper, Analysis of different performance levels by comparing fragility curves and behavioral parameters in zipper bracing structures under increasing dynamic analysis. Seismic analysis of 3rd, 5th and 7th floor structures was performed with Zipper bracing structural system .Fragility curves were obtained for functional levels IO, LS and CP. The fragility curves have been plotted for the zipper bracings for the first time. These curves can be used as the bases for estimating seismic demands and performance based design of the structures for such bracings. As a general conclusion, the results showed that the RLRFD parameter increases with increasing altitude. The RASD parameter also increases with increasing altitude. As the height increases, the parameter Rμ increases. But for the parameters Rso and Rs the results showed that the maximum value is obtained in a 5-story structure. As the height and floors of the structure increase and decrease, it decreases to 5 and 7 floors of two parameters Rso and Rs

Keywords: Seismic behavior, zipper-braced frames, Fragility Curve, IDA analyze.

1. Introduction

The divergent braced frame with zipper can be considered as a bracing system consisting of horizontal beam elements connected to the overall shear and bending behavior. If the design of the structure is based on capacity, the presence of zipper elements significantly improves the seismic behavior of the braced structure. In fact, the submission of one or more bonding beams does not create a large concentration of large plastic cycles in the overall behavior of the structure. Zipped bracing panels create such a feature due to the stiffness of the floor due to the axial stiffness of the braces, columns and zipper elements compared to the flexural and shear stiffness of the beams. Although the stiffness of the floor is fully maintained until the beam is surrendered, the overall stiffness of the structure is greatly reduced. Accordingly, zipped divergent braced structural panels gradually tend to behave with the appearance of rigid deformation. This behavior causes the bond to have a relatively high plastic capacity and stiffness in other structural components and braces. The presence of zipper elements in structures with divergent bracing system avoids a sharp and rapid reduction in floor stiffness when yielding bonded beams.

Wu and Lu [32] suggested light-weight energy-dissipa- tion rocking core frame as a novel lateral load resisting sys- tem that has improved seismic performance in terms oflow residual drifts due to utilizing self-centering energy-dissi- pation braces. Blebo and Roke proposed seismic-resistant self-centering rocking core system to provide consider- able drift capacity while limiting residual drift and struc- rural damage [2, 3). Grigorian and Griogorian [4] pro- posed structural design of rocking wall-frame which have uniform drift distribution and lead to prevention of soft story mechanism formation. Mortier et al. [5], use rock- ing systems in the retrofitting of existing steel structures. Jia et al. [6] suggested rocking dual-steel has trilinear hysteretic behavior and lead to mitigation of residual drift as a result of early re-yielding of low-yield point steel. The results of Moradi and Burton [7] showed that the seis- mic response of the controlled rocking steel braced frames do not affect considerably by changing design parame- ters such as post-tensioned bars modulus of elasticity and strain hardening ratio of the fuses.

Rocking Concentrically Braced Frame (RCBF) is one type of the rocking system that has been proposed in the past decade as high performance lateral load resisting sys- tem for steel structures. The rocking behavior at the base causes the larger lateral displacement and limited mem- ber force demand as a result of softening mechanism of the RCBFs [8]. Also. RCBF experiences first damage at higher drifts comparing with CBF system. resulting in better seismic performance in terms of drift ratio and resid- ual drift as presented in different experimental and analyt- ical studies [9, 10]. Recent studies presented that RCBFs like rocking concentrically braced frame [9. 11, 12], dual rocking frames [13, 14], using multiple rocking joint through the height of the frame [15], rocking braced frames with energy dissipating elements and various story num- bers [16] and tension-only concentrically braced frames with rocking core [17] are able to decrease damage under severe earthquakes and enhance the seismic performance of ordinary CBFs in terms of uniform distribution of inter story drifts. Huang et al. showed that RCBFs are eco- nomically better systems comparing to CBFs for low and mid-rise building under severe earthquakes [18].

2. Research methods

For modeling the members in nonlinear range of deformation, the following assumptions were made. Nonlinear beam-column element is used in the software to model the columns, beams and bracings. This element can consider the effects of P-Δ and large deformations to account for geometric nonlinear effect of the model. In order to model wide plasticity in the member length and nonlinear buckling in the program, each element including beam, column and bracing, is divided into several fibers along their sections and several segments along their lengths. The

OpenSees was used for modeling and conducting incremental dynamic analysis. This software, produced by Berkley University of California, is one of the most effective software in nonlinear and dynamic analysis. In this research, nonlinear time history analysis procedure was adopted to perform the seismic analyses of the frames. The parameters of these events are very similar to those of the site on which the structure has been built. Accordingly, 15 records are chosen from known worldwide earthquakes including two big earthquakes in Iran, Bam and Tabas as listed in Table 1. Shear wave velocities of all these sites correspond to the velocity in soil type II (375-750 m/s) as presented by Iranian code for seismic design (Code 2007) and site category B of USGS classification[19]. According IM scaling, hunt and fill algorithm was used to optimize the number of scaling of each record. They are applied in nonlinear dynamic analysis and drawing IDA Plots with enough accuracy and speed. In the first step of IM scaling, a very low level of seismic intensity parameter (0.005 g) is selected (spectral acceleration of the first mode) which guarantees the linear response of structure. A minimum number of points are used in the hunt stage in order to achieve the range of spectral acceleration of the first mode where the damage has happened. Based on Eq. (1), the IM values increase sequentially with the seismic intensity in each step. The value of Sa (T1) in each step is calculated as follows:

$$S_a\,(T_1)_i = S_a\,(T_1)_{i-1} + \alpha * (i-1) \tag{1}$$

Where, Sa (T1) is spectral acceleration corresponding to first mode; i is the number of steps; and α is a factor. In this research α = 0.05. Fill step is started after analyzing each step and finding the interval of spectral acceleration where the damage limit state happens. As the seismic stress parameter increases progressively, it is showed that it increases in accordance with the above relation. The spectral acceleration which accurately corresponds to the considered damage limit state is determined by increasing the analysis points between two spectral acceleration values where the considered damage limit state happens. The accuracy of IDA Plot can also increase by increasing the analysis points in other intervals of spectral acceleration of the first mode.

Table 1. Earthquake characteristics used in the present study

Earthquake	Source	PGA (g)
Earthquake 1	Santa Monica	0.88
Earthquake 2	Anderson Dam	0.42
Earthquake 3	Cool Water	0.42
Earthquake 4	KJMA	0.82
Earthquake 5	Bam	0.77
Earthquake 6	CHY080	0.9
Earthquake 7	Tabas	0.92
Earthquake 8	Gilroy	0.43
Earthquake 9	Corralitos	0.64
Earthquake 10	N.Palms	0.69
Earthquake 11	Manjil	0.84
Earthquake 12	ChiChi	0.698
Earthquake 13	Landers	0.78
Earthquake 14	Kobe	0.7
Earthquake 15	Coyot	0.48

2-1- Incremental dynamic analysis

Incremental Dynamic Analysis (IDA) is an emerging analysis method that offers thorough seismic demand and capacity prediction capability by using a series of nonlinear dynamic analyses under a multiply scaled suite of ground motion records. Realization of its opportunities requires several innovations, such as choosing suitable ground motion Intensity Measures (IMs) and representative Damage Measures (DMs). In addition, proper interpolation and summarization techniques for multiple records need to be employed, providing the means for estimating the probability distribution of the structural demand given the seismic intensity. Limit-states, such as the dynamic global system instability, can be naturally defined in the context of IDA, thus allowing annual rates of exceedance to be calculated. Finally, the data gathered through IDA can provide intuition for the behavior of structures and shed new light on the connection between the Static Pushover (SPO) and the dynamic response [20].

2-3- The Models used in research

In this article, three steel frames of 3, 5 and 7 stories with bracing systems have been investigated according to the Iranian standard No. 2800 (Code 2007). They have been assumed to be established in a zone with very high seismicity on the soil type II (based on the Iranian standard No. 2800) with an average shear wave velocity of 375–750 m/s in a depth of 30 m. These buildings have been designed in line with the requirements of Iranian earthquake resistance design code and Iranian National Building Code for steel structure design. The heights of all stories are 3 m, the spans are 6 m and the applied steel is of st-37-1 kind with a yield stress of 235 MPa. The dead and live loads are 4.5 KN/m^2 and 2 KN/m^2, respectively. All connections including beam to column, braces to each other and braces to beam-column joint are in the form of hinge. However, the connections of the three bracing members should have sufficient rigidity at their crossing point in order to resist against the movements normal to frame plane and prevent geometric instability. In this study, the plans of all stories have been considered the same and shown in Fig. 1. The locations of bracings are presented as dotted line in this figure. Due to the fact that the internal frames have totally hinged connections without bracings, they play no role in tolerating the lateral loads. A two-dimensional frame has been selected as the representative of the tri-dimensional structure for IDA analysis in order to reduce the time and calculation volume of dynamic analysis. The mentioned frame has been shown by dotted line in the above mentioned figure. The cross-sections of model members are shown in Table 2. In order to design the structures subjected to earthquake, equivalent lateral static forces were applied to all stories. These forces are calculated according to the Iranian Earthquake Code (Code 2007). The design base shear is computed as follows:

$$V = C*W \qquad (12)$$

$$C = ABI/R \qquad (13)$$

Where, V is base shear; C is seismic coefficient; W is equivalent weight of structure;

- ✓ A is design base acceleration;
- ✓ B is response factor; I is importance factor;
- ✓ And R is response modification factor.
- ✓ A × B is design spectral acceleration expressed as gravitational acceleration (g)

- ✓ Against fundamental period of structure (T)
- ✓ For soil type II

The importance factor (I) and design base acceleration (A) were assigned with the values of 1 and 0.35, respectively, for designing the frames. The value of primary response modification factor is taken as 5.5 in allowable stress design method. The frame members have been designed by allowable stress design method according to Iranian National Building Code. Figure 1- showed the plan of the structure studied in the present study.

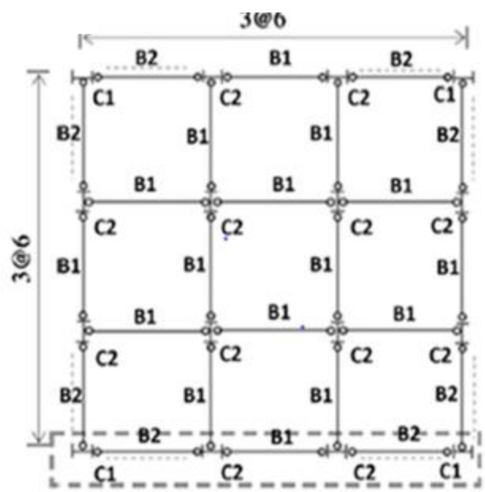


Figure 1- The plan of the structure studied in the present study

3- Result and discussion

In the present study, the behavior of structures with zipper bracing has been investigated. 3, 5 and 7 floor structures have been investigated with incremental dynamic analysis and nonlinear static analysis. Pushover curves of the frames with different stories were plotted for loading pattern of the first mode in terms of roof displacement-base share. The values of static base shear equivalent to the first plastic hinge formation in the structure have been derived from Fig. 2.
Figure 3 shows the cover diagram of different structures. The base shear for the three-story structure was 408 kN. For a 5-story structure, a base shear value of 791.2 kN was obtained. For a 7-story structure, the base shear value was 1027.4 kN and the base shear volume increased with increasing the number of floors of the structure.

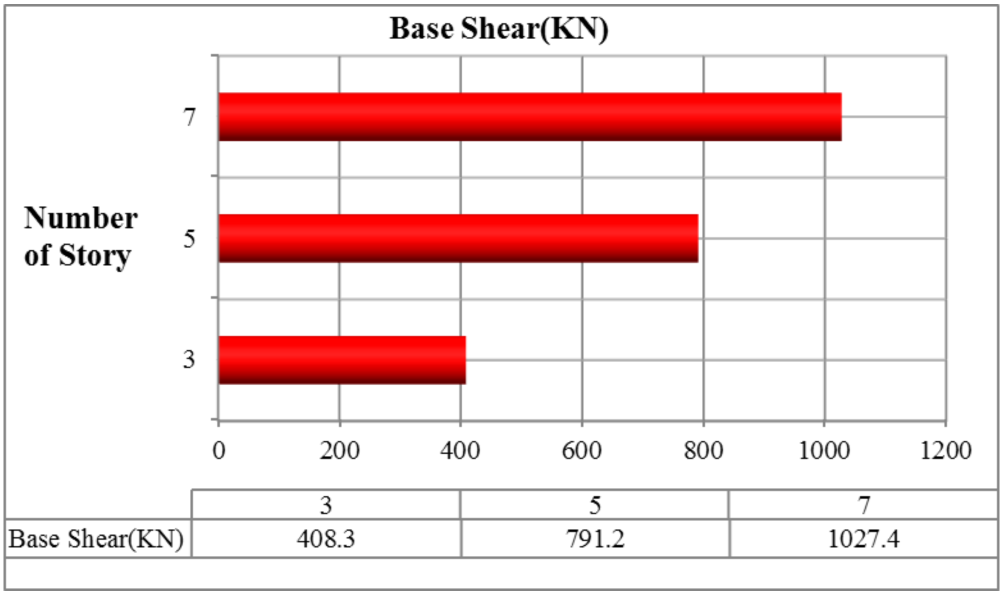


Figure 2- The amount of base shear that creates the plastic joint for 3, 5 and 7 floor structures

The capacity of the various structures under study is shown in Figure 3 with the basic shear diagram. As shown, the maximum amount of roof displacement was 0.2 m for a 3-storey structure, 0.4 m for a 5-storey structure, and 0.45 m for a 7-storey structure.

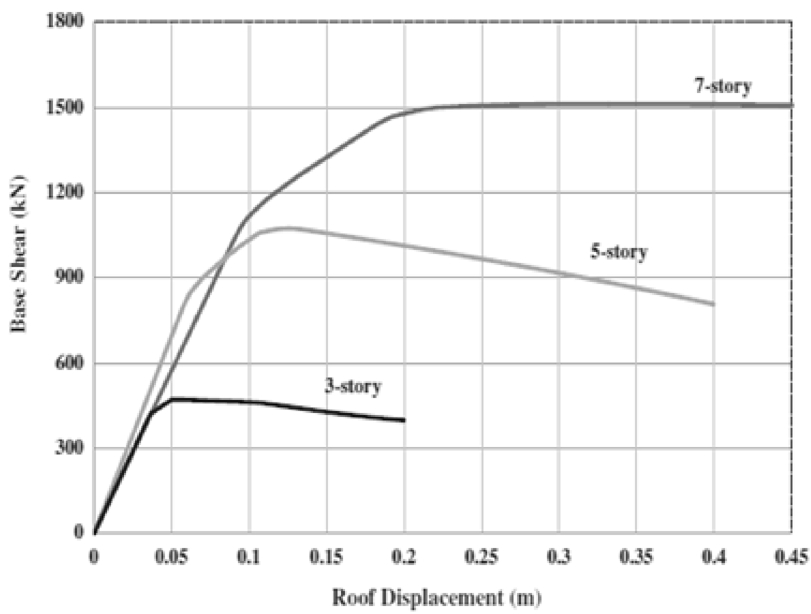


Figure **3-** Pushover curves of studied frames

3-1- IDA curves

IDA curves of the studied frames have been presented in Fig.6 in terms of maximum inter-story drift ratio-spectral acceleration. All behavior stages of the structure, subjected to earthquake, from elastic limit to collapse and global instability are shown in the curves. As it is seen in the curves, the structures enter into the nonlinear zone sooner with increasing the heights. Moreover, the IM values are reduced in the curves for a constant value of DM. In other words, the Sa capacity of structures corresponding to a certain damage criterion is reduced with increasing the height of structure. Figure 4 it shows Incremental dynamic analysis results for 5 story and Figure 5. Incremental dynamic analysis results for 7 story.

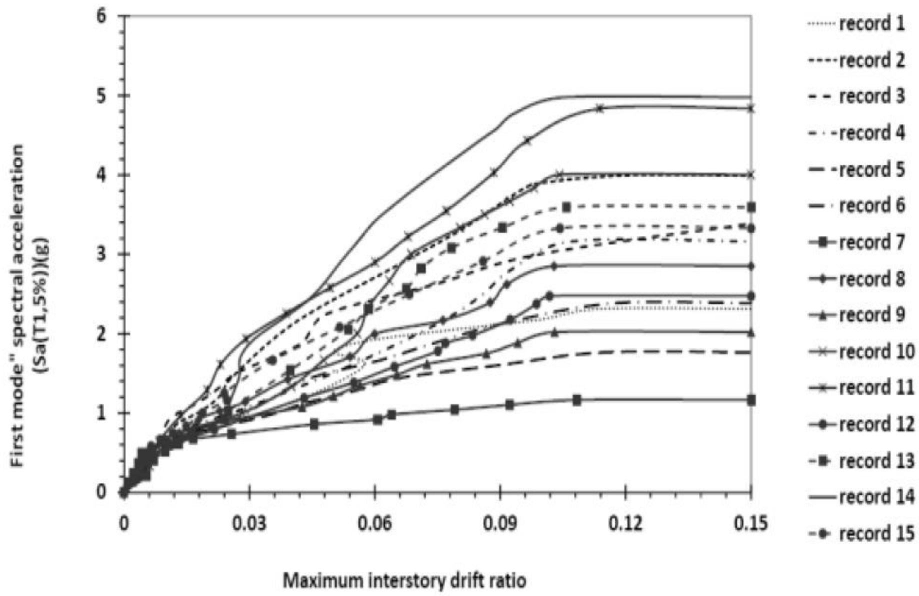


Figure 4. Incremental dynamic analysis results for 5 story

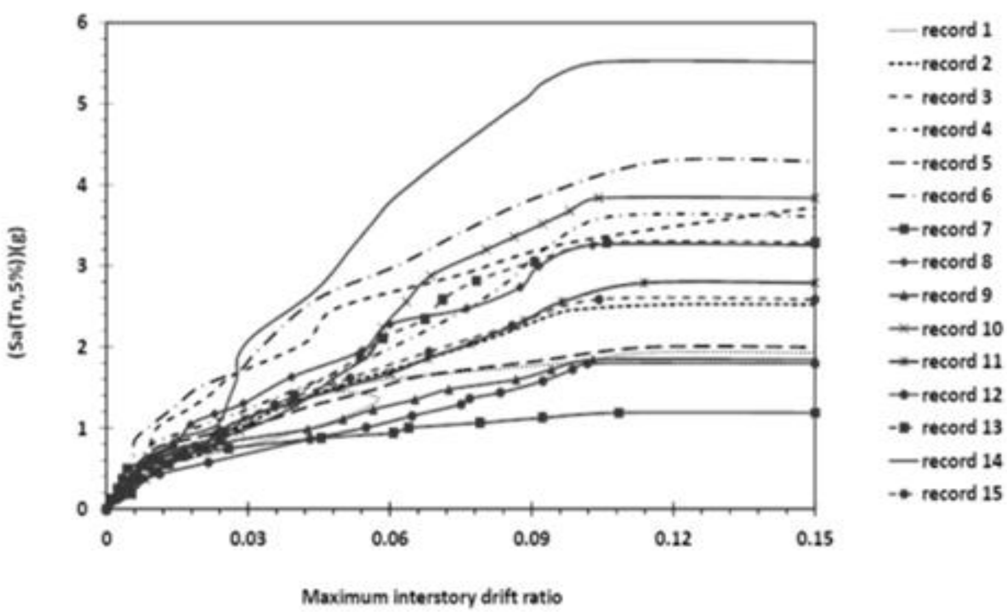


Figure 5. Incremental dynamic analysis results for 7 story

Figure 6 It shows Median IDA curves for 5 story Braced and Figure **7** Median IDA curves for 5 story Braced.

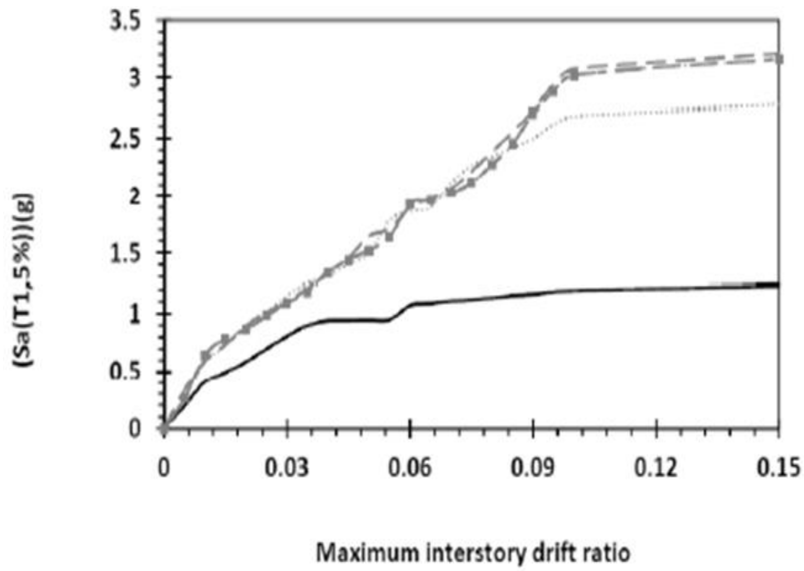


Figure 6- Median IDA curves for 5 story Braced

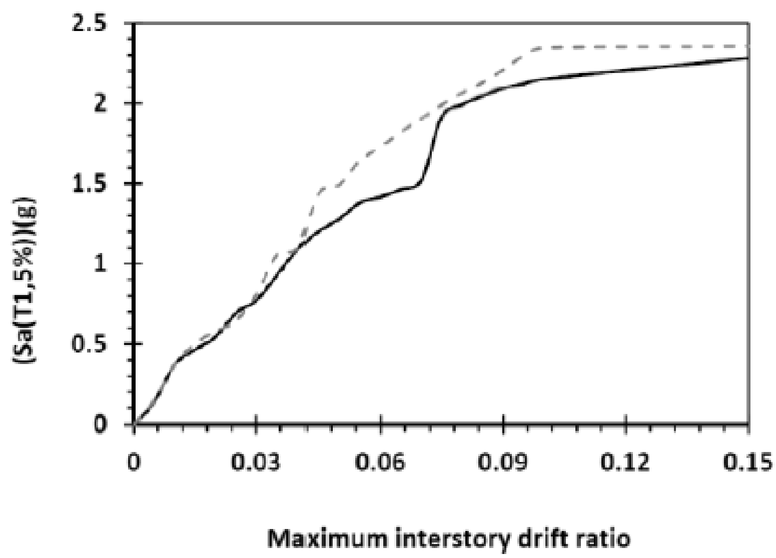
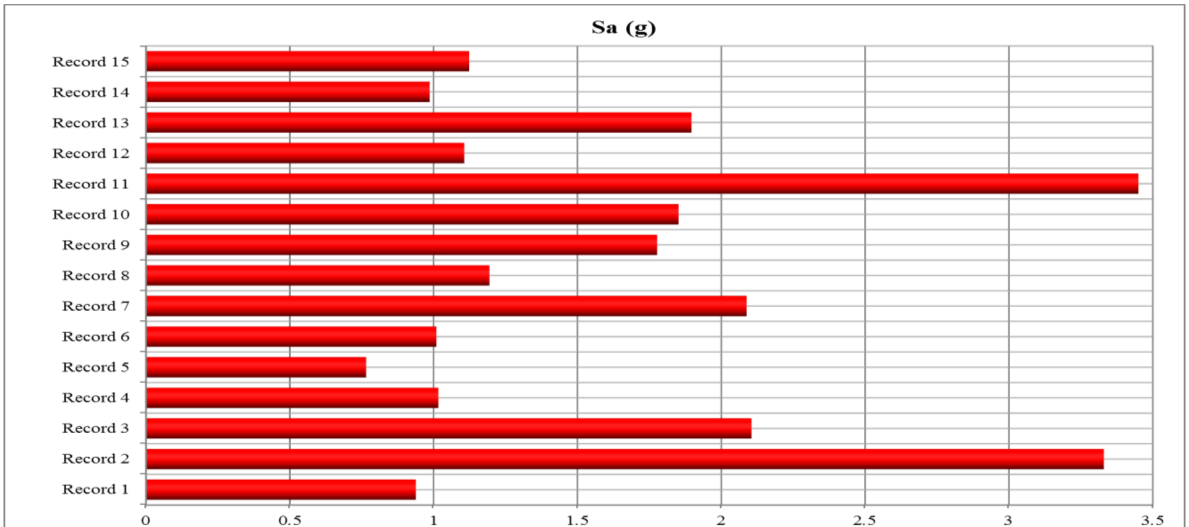


Figure 7- Median IDA curves for 5 story Braced

3-2- Response modification factor

Ultimate base shear ($V_{b(Dyn,y)}$) and limit state are obtained from nonlinear dynamic analysis. They are tabulated in Table 4 under the earthquake records selected for designed braced frames. Table 5 shows the maximum elastic base shear ($V_{b\ (Dyn,\ e)}$) resulting from linear dynamic analysis under the selected earthquake records. Concerning the above results and the descriptions of limit state and allowable stress designing methods in Section 4, ductility, over strength and response modification factors have been calculated for the studied frames and presented in Table 6. The values obtained for over strength of the frames are 1.2–1.5. According to Table 6, the values of over strength, ductility and response modification factors decrease as the height of building increases. In the shorter frames, the slope of structural behavior curve tends to increase the pre-yield stiffness of the system sharply and thereby reduce the value of Δy. The specified global drift limit (Δ_{max}) however remains constant at 2% or 2.5% of the height of the system. This in turn greatly increases the ductility and consequently R value of the braced system. In the taller frames, the increase in ductility and R value are of lower magnitudes. Figure 8- it shows Nonlinear maximum base shear and limit state point for models Earthquake motions 3 story and Figure 9- Nonlinear maximum base shear and limit state point for models Earthquake motions 3 story.

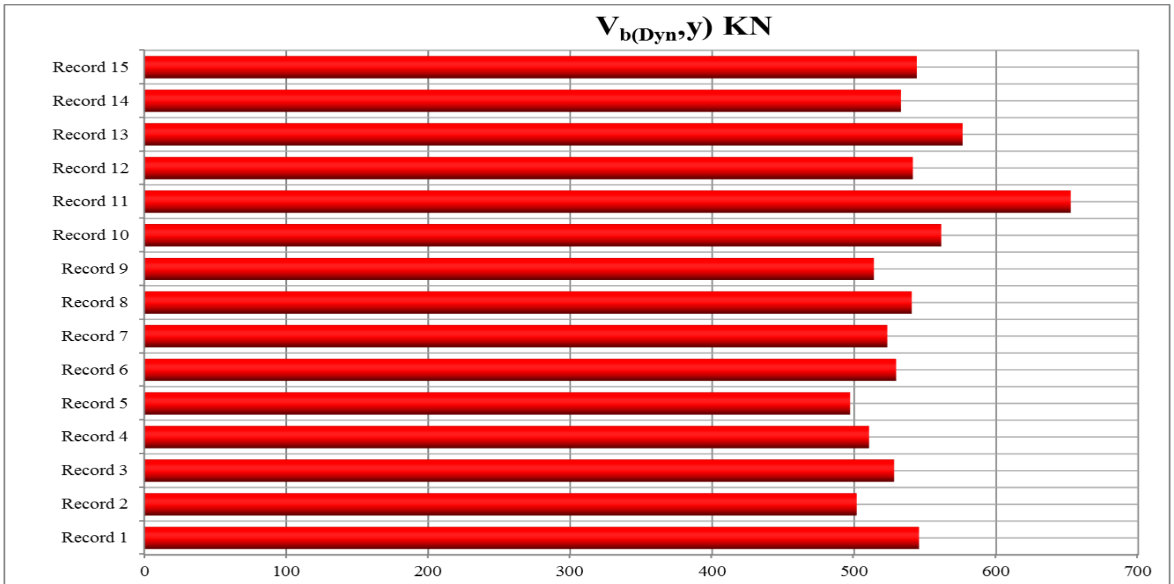
As shown in the figure 8, the maximum parameter of Sa (g) is 3.45 and the structure under earthquake 11 has the highest value of Sa (g). The lowest value of the Sa (g) parametrs, as shown in the figure 8, is related to earthquake 5 and equal to 0.765.



	Record 1	Record 2	Record 3	Record 4	Record 5	Record 6	Record 7	Record 8	Record 9	Record 10	Record 11	Record 12	Record 13	Record 14	Record 15
Sa (g)	0.938	3.33	2.105	1.016	0.765	1.009	2.088	1.194	1.777	1.851	3.45	1.106	1.896	0.986	1.123

Figure 8- Nonlinear maximum base shear and limit state point for models Earthquake motions (3 story)

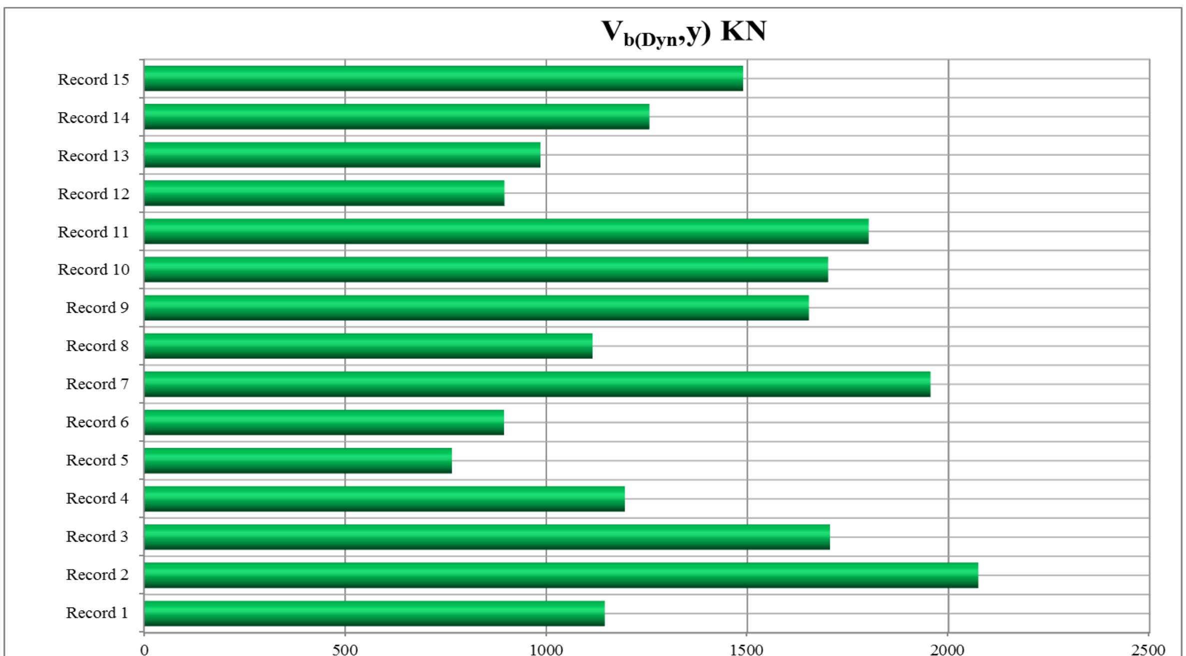
As shown in Figure 9. The maximum parameter of Vb is 561.2 and the structure under earthquake 11 has the highest value of Vb. The minimum value of Vb, as shown in the figure 9, is related to earthquake 2 and is equal to 501.8.



	Record 1	Record 2	Record 3	Record 4	Record 5	Record 6	Record 7	Record 8	Record 9	Record 10	Record 11	Record 12	Record 13	Record 14	Record 15
Vb(Dyn,y) KN	545.6	501.82	528.17	510.4	497.04	529.48	523.31	540.55	513.85	561.23	652.4	541.3	576.3	532.9	544.1

Figure 9- Nonlinear maximum base shear and limit state point for models Earthquake motions (3 story)

Figure 10-it shown linear maximum base shear of models Earthquake motions 3 story. The highest value of V_b was obtained in earthquake 15 and equal to 2073.97 KN. Figure11- Linear maximum base shears of models Earthquake motions 5 story.



	Record 1	Record 2	Record 3	Record 4	Record 5	Record 6	Record 7	Record 8	Record 9	Record 10	Record 11	Record 12	Record 13	Record 14	Record 15
Vb(Dyn,y) KN	1145.21	2073.97	1705.28	1195.32	765.71	895.31	1955.02	1115.26	1652.78	1700.88	1801.32	896.32	985.65	1256.32	1489.65

Figure 10- Linear maximum base shear of models Earthquake motions 3 story

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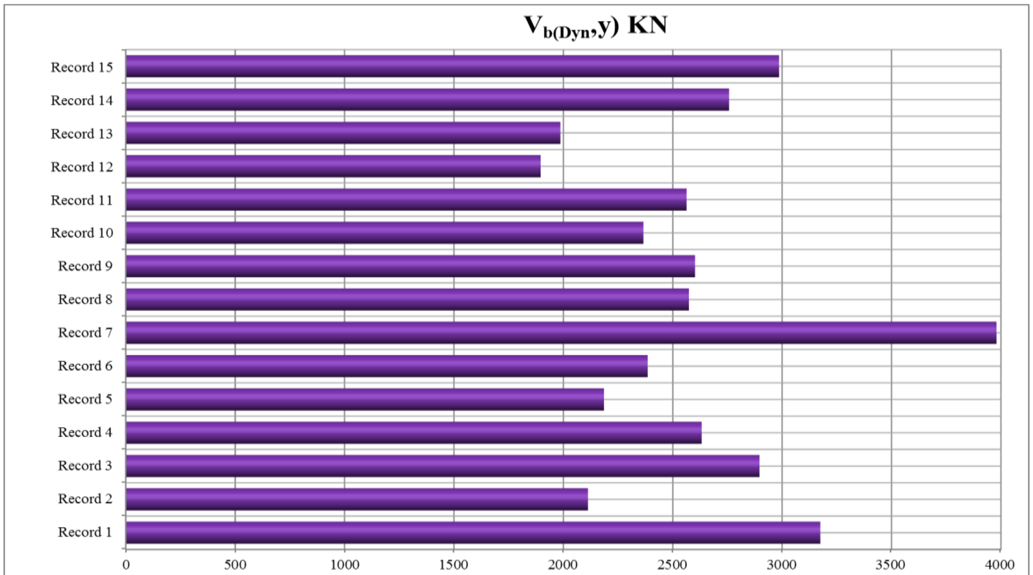
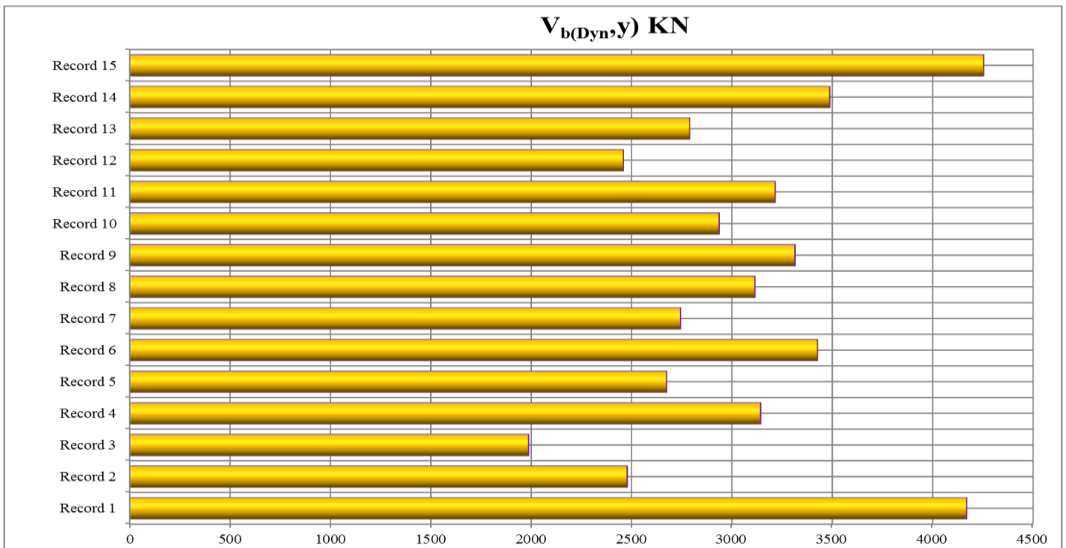
As shown in Figure 11. Maximum parameter value Vb under earthquake 7 has the highest value and is equal to 3980.69. The lowest value Vb as shown in Figure 11 was related to earthquake 2112.33.



	Record 1	Record 2	Record 3	Record 4	Record 5	Record 6	Record 7	Record 8	Record 9	Record 10	Record 11	Record 12	Record 13	Record 14	Record 15
Vb(Dyn,y) KN	3174.23	2112.33	2896.54	2632.14	2185.69	2385.14	3980.01	2574.32	2601.77	2365.61	2563.23	1896.32	1986.36	2756.98	2986.32

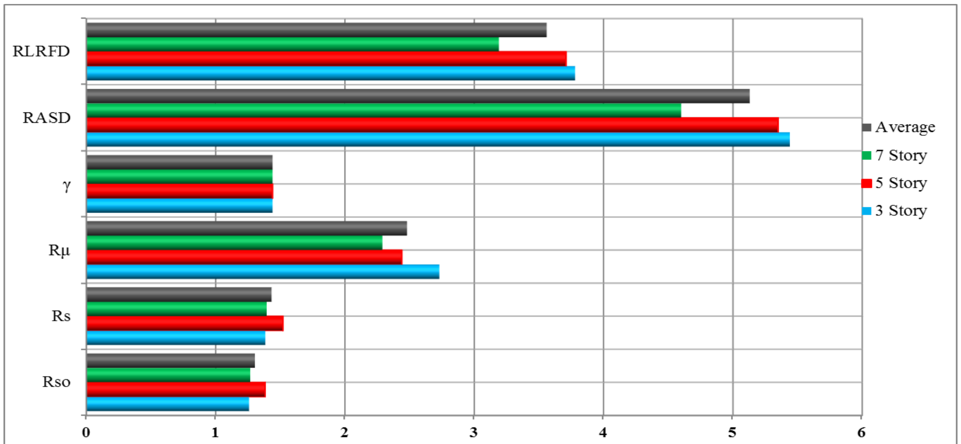
Figure11- Linear maximum base shear of models Earthquake motions 5 story

Figure12- it shows Linear maximum base shear of models Earthquake motions 7 story .As shown in Figure 11. The highest value of the parameter Vb was obtained under earthquakes 1 and 15 and was equal to 4253.65 and 4169.5 .The lowest value Vb as shown in the figure is related to the earthquake and the value of 1987.36 was obtained.



	Record 1	Record 2	Record 3	Record 4	Record 5	Record 6	Record 7	Record 8	Record 9	Record 10	Record 11	Record 12	Record 13	Record 14	Record 15
Vb(Dyn,y) KN	4169.5	2478.36	1987.36	3142.65	2674.83	3426.32	2744.16	3114.68	3314.12	2936.63	3215.36	2459.36	2789.63	3486.98	4253.65

Figure12- Linear maximum base shear of models Earthquake motions 7 story



	Rso	Rs	Rμ	γ	RASD	RLRFD
Average	1.303	1.433	2.48	1.44	5.13	3.56
7 Story	1.267	1.394	2.29	1.44	4.6	3.19
5 Story	1.383	1.521	2.44	1.44	5.35	3.71
3 Story	1.258	1.384	2.73	1.44	5.44	3.78

Figure13- Over strength, ductility factor and response modification factor of model

Figure 13 shows more than resistance, ductility coefficient and response correction factor. Different behavioral parameters were obtained for 3rd, 5th and 7th floor structures.

3-3- Fragility curves

In this research, the fragility curves are used to derive the occurrence probability of the limit state from IDA results. The processes involved in plotting these curves are: first, the IM values which correspond to the considered limit state occurrence are sorted in descending order for all records; second, the occurrence probability of the limit state is calculated for values lower than or equal to the considered IM value. These curves show the occurrence probability of the limit state for each IM value at any performance level of the structure regardless of the seismic hazard. The only condition is that the density value is limited to the considered level. The fragility curves of the limit states of IO have been plotted for all three studied structures and presented in Fig. 14. The values given in this table can be used for designing earthquakes with the probability of certain level of collapse and evaluating the design codes of structures against such earthquakes.

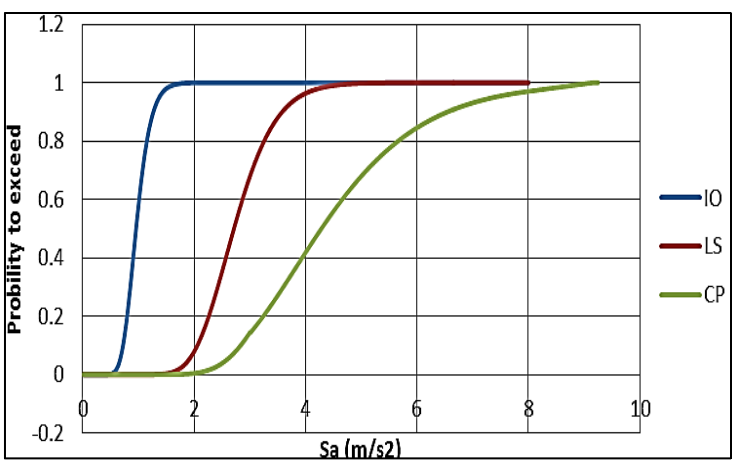


Figure 14- Fragility curves for 5 story for performance level of Immediate Occupancy (IO).

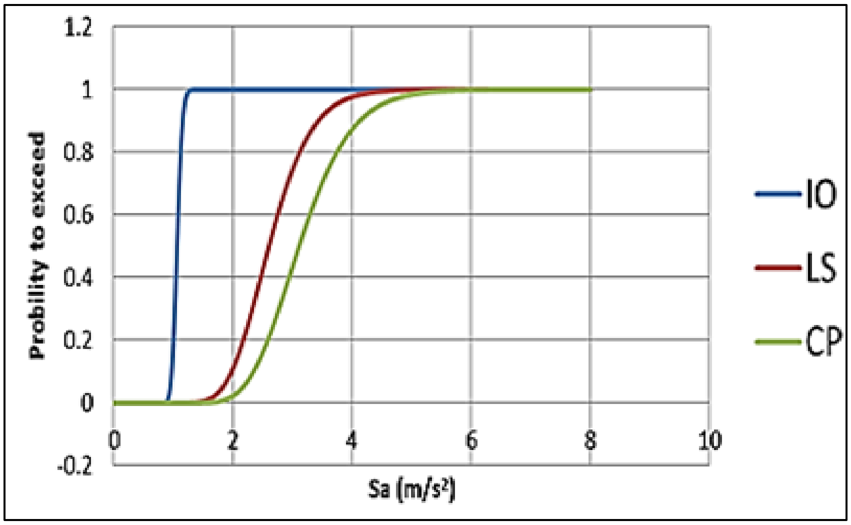
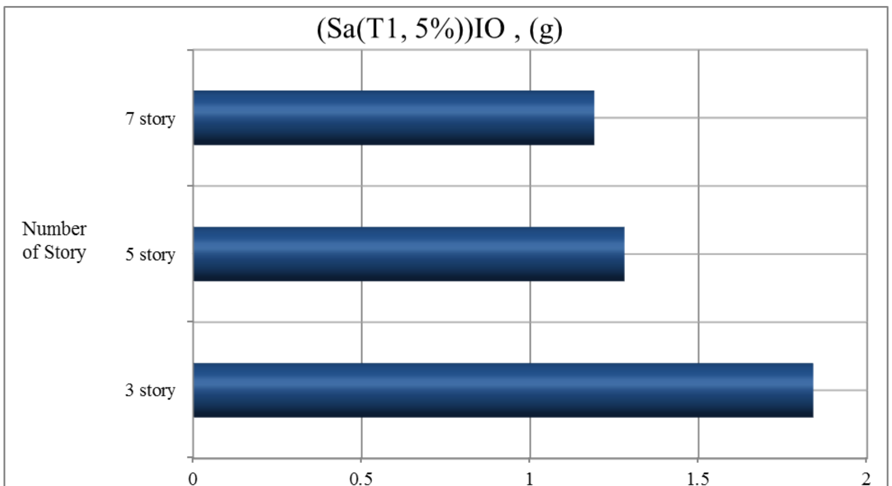


Figure 15- Fragility curves for 7 story for performance level of Immediate Occupancy (IO).



	3 story	5 story	7 story
(Sa(T1, 5%))IO , (g)	1.84	1.28	1.19

Figure 16. The values of Sa, corresponded to 50% of structural failure.

4- Conclusion

In this paper, Analysis of different performance levels by comparing fragility curves and behavioral parameters in zipper bracing structures under increasing dynamic analysis. Seismic analysis of 3rd, 5th and 7th floor structures was performed with Zipper bracing structural system . Fragility curves were obtained for functional levels IO, LS and CP. The fragility curves have been plotted for the zipper bracings for the first time. These curves can be used as the bases for estimating seismic demands and performance based design of the structures for such bracings. From the study of 6 seismic parameters to evaluate the behavior of structures with different number of floors with the zipper bearing system, it was concluded that:

- The value of Rso parameter for the average of three structures of 3, 5 and 7 story with zipper bracing was 1.303.
- The value of Rs parameter was obtained for the average of three structures of 3, 5 and 7 floors with zipper bracing equal to 1.443.
- The value of Rμ parameter for the average of three structures of 3, 5 and 7 floors with zipper bracing was 2.48.
- The value of RASD parameter for the average of three structures of 3, 5 and 7 floors with zipper bracing was 5.13.
- The value of RLRFD parameter for the average of three structures of 3, 5 and 7 floors with zipper bracing was 3.56.

The results of seismic-behavioral parameters of 3-story structures were found to be:

- The value of Rso parameter for a 3-story structure with zipper bracing was 1.258.
- The value of Rs parameter for a 3-story structure with zipper bracing was 1.384.
- The value of Rμ parameter for a 3-story structure with zipper bracing was 2.73.
- The value of RASD parameter for 3-story structure with zipper bracing was 5.44.
- The value of RLRFD parameter for 3-story structure with zipper bracing was 3.78.

From the results of seismic-behavioral parameters of the 5-story structure, it was concluded that:

- The value of Rso parameter for a 5-story structure with zipper bracing was 1.383.
- The value of Rs parameter for a 5-story structure with zipper bracing was 1.521.
- The value of Rμ parameter for a 5-story structure with zipper bracing was 2.44.
- The value of RASD parameter was obtained for 5-story structure with zipper bracing equal to 5.35.
- The value of RLRFD parameter for 5-story structure with zipper bracing was 3.71.

From the results of seismic-behavioral parameters of a 7-story structure, it was concluded that:

- The value of Rso parameter for a 7-story structure with zipper bracing was 1.267.
- The value of Rs parameter for a 7-story structure with zipper bracing was 1.394.
- The value of the parameter Rμ for a 7-story structure with a zipper brace was 2.29.
- The value of RASD parameter for 7-story structure with zipper bracing was 5.13.
- The RLRFD parameter value was 3.56 for a 7-story structure with a zipper brace.

As a general conclusion, the results showed that the RLRFD parameter increases with increasing altitude. The RASD parameter also increases with increasing altitude. As the height increases, the parameter Rμ increases. But for the parameters Rso and Rs the results showed that the maximum value is obtained in a 5-story structure. As the height and floors of the structure increase and decrease, it decreases to 5 and 7 floors of two parameters Rso and Rs.

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