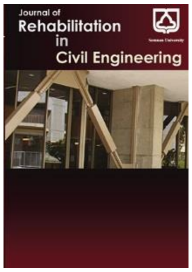
Journal of Rehabilitation in Civil Engineering 7-4 (2019) 180-193



journal homepage: http://civiljournal.semnan.ac.ir/

Performance Based Seismic Rehabilitation of Steel Structures with Different Types of Shear Walls

A. Khodadadi[1], A. Sivandi-Pour[1*] and S.H. Madani[1]

1- Faculty of Civil and Surveying Engineering, Graduate University of Advanced Technology, Kerman, Iran

Corresponding author: *a.sivandi@kgut.ac.ir*

ARTICLE INFO

Article history:
Received: 28 July 2018
Accepted: 15 February 2019

Keywords:
Seismic Rehabilitation,
MRF,
Concrete Shear Wall,
Adaptive Modal Pushover Analysis,
Ida,
Fragility Curves.

ABSTRACT

Seismic rehabilitation provides existing buildings with more resistance to seismic activity, ground motion, or geotechnical failure as a result of earthquakes. Performance-based rehabilitation is a general concept through which the retrofitting criteria are defined regarding to performance objectives when the structural and nonstructural members are subject to different levels of earthquake hazards. In this study, several moment resistant steel frames with different numbers of stories were initially designed as vulnerable models. These models were retrofitted in consonance with the current seismic rehabilitation standards and codes criteria. Three models of shear walls were applied to retrofitting the vulnerable structures. In the first model, the wall surrounds column perimeter as boundary elements. In the second model, wall is connected to the column and in the 3rd model, wall is placed with a small gap from the column, and there is no contact between them. The nonlinear behavior of buildings is evaluated applying adaptive modal pushover and incremental dynamic analysis before and after rehabilitation.

1. Introduction

Moment resistant steel frames (MRFs) are prevalent in many major buildings around the world,.Therefore, extending their service life and capacity reduction are among the criteria that make the vulnerability of these structures a significant topic in structural engineering. High costs of demolishing and re-building on the one hand, and the time spent to build a new structure on the other hand, cause the retrofit option of these buildings to be highly prioritized. Each retrofitting plan is accurately investigated and planned pursuant to the current situation of the building. Consequently, it could be assured all acceptance criteria of credible codes for structural and non-structural members are met. The criteria considered in a retrofit plan are confirming to general rules of seismic

DOI: 10.22075/JRCE.2019.15507.1289

resistant buildings, but it differs as in this case there are limits of current structural, architectural, and usage status which requires more creative methods and solutions [1-4]. After vulnerability assessment of a building at seismic levels and determining the weak points, designers can proceed to craft a retrofit plan. The retrofitting goal is to ameliorate the structure behavior against seismic forces to limit the damages and effects on building elements to expected levels at each performance level once the retrofitting is applied to the structure [5-7]. There have been many studies on seismic retrofitting of structures. Gerami and Sivandi-Pour retrofitted residential steel buildings with dual system of moment resisting frames and eccentrically braced frames (MRF–EBF). The cover plates were applied to perform the rehabilitation of models [8]. Di Sarno and Elnashai presented three methods to retrofit a mid-rise building. Special concentrically braced frames, buckling-restrained braced frames, and mega-braced frames were used in their research. They noted that applying the mega-braced frames was the most economical method. The value of steel in structural members and connections in the model of mega-braced frames is 20% lower than those with special concentrically braced frames [9]. Kurata et al. presented a retrofitting method for low-rise steel buildings. In their proposed model, a thin steel plate as a cover shear wall mechanism was installed in the middle of the bay, separate from columns. [10]. Shakib et al. retrofitted a 19-storey steel building with semi-rigid connections employing shear wall (steel and concrete), and steel bracing. They concluded that using concrete shear wall was the most optimized system [11]. Jiang et al. investigated the seismic behavior of panel wall (steel and composite) strengthened steel frames by experimental study. An experimental model of 1/3-scaled sample with single-bay and one-story was built. They inspected the effects of the length-to-height ratio, stiffeners, and the type of walls on the performance of models. They concluded that the models tolerate 4% to 5% story drift, the panel wall (steel and composite) increased the seismic behavior of the system [12]. Mirza et al. evaluated the effect of fatigue on the behavior of 120 years old and new equivalent steel buildings. They assessed the fatigue performance of the structures (old and new) by analyzing the stresses at critical zones within the buildings. They argued rehabilitation methods to the both structures to increase the fatigue performance and upgrade the design life of steel buildings [13]. TahamouliRoudsari et al. experimentally investigated a comparison between different steel brace models on the performance of reinforced concrete moment resisting frames. Vulnerable models were rehabilitated with the chevron, X, the knee, the eccentric brace and the chevron brace with a vertical link. They deduced that the eccentric brace had a better behavior compared to the other methods in the ductility. However, from the strength, stiffness and cracking control sight, the performance of the X brace was better [14].

In this study, the vulnerable moment resistant frames are rehabilitated through applying three different execution methods. In the first method, the shear wall is with boundary elements around the column. In the second method, the shear wall is executed connected to the columns on either side, and in the third method, the shear wall is executed with a small gap from surrounding columns. The life safety (LS) performance level for the BSE-1 and collapse prevention (CP) performance level- for the BSE-2 earthquake hazard level were considered as rehabilitation goals. Vulnerable structures were retrofitted based on methods of the current seismic rehabilitation codes. The nonlinear behavior of buildings is assessed based on adaptive modal pushover analysis and incremental dynamic analysis (IDA). SeismoStruct [15] software was applied for nonlinear seismic analyses of structures.

2. Designing and Modeling the Structures

Four MRF frames with 5, 10, 15 and 20 stories were modeled and designed for rehabilitation the residential buildings. LRFD specification and AISC seismic provisions [16] were used to design of buildings in ETABS software. The models were loaded confirming to ASCE 7-10 [17]. The vulnerable steel buildings were designed as moment resistant frames under the assumption that the relative hazard is underestimated than real amount. The stories' height was 3.4 m. For designing 15 and 20 story buildings, spectral analysis was employed. The sections were chosen: plate girder was used for the beams; box section was used for the columns. The specifications of the frames are presented in table 1.

Table 1. Specifications of the designed frames.

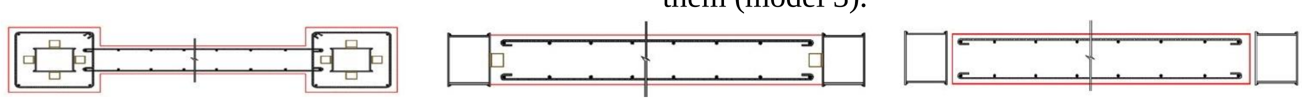
Building	Story	Column		Beam (Web-Flange)	
		Min Section	Max Section	Min Section	Max Section
5	5	BOX150x10	BOX180x12	W150x6-F120x8	W200x8-F150x10
	3-4	BOX150x10	BOX200x15	W250x8- F200x12	W300x8- F200x15
	1-2	BOX250x15	BOX300x20	W250x8- F200x12	W350x8- F250x20
10	9-10	BOX180x10	BOX240x15	W250x10-F200x20	W250x10-F200x20
	7-8	BOX200x15	BOX300x15	W250x10-F200x20	W300x10- F250x20
	4-6	BOX200x15	BOX400x20	W250x10-F200x20	W300x10- F250x20
	1-3	BOX400x20	BOX500x20	W300x10- F250x20	W400x10- F300x30
15	13-15	BOX240x12	BOX400x20	W250x8- F200x15	W450x20- F350x25
	9-12	BOX240x12	BOX500x20	W250x8- F200x15	W450x20- F350x25
	5-8	BOX300x15	BOX650x30	W250x8- F200x15	W450x20- F350x25
	1-4	BOX500x20	BOX800x40	W450x20- F350x25	W700x25- F500x30
20	16-20	BOX180x12	BOX450x20	W250x8-F200x12	W400x10-F300x30
	11-15	BOX400x20	BOX550x25	W400x10-F300x30	W500x25-F400x30
	6-10	BOX550x25	BOX650x30	W400x10-F300x30	W700x20-F550x30
	1-5	BOX650x30	BOX1000x55	W650x20-F450x30	W900x30-F650x35

In this research concrete shear walls in 3 models were deliberated as highlighted in figure 1:

(a) The wall surrounds the column perimeter as boundary elements (model 1).

(b) Wall is connected to the column (model 2).

(c) The wall is placed with a small gap from the column, and there is no contact between them (model 3).



(a) (b) (c)

Fig. 1. The shear wall models: (a) model1, (b) model2, (c) model3.

The allocation of walls in plans is portrayed in figure 2.

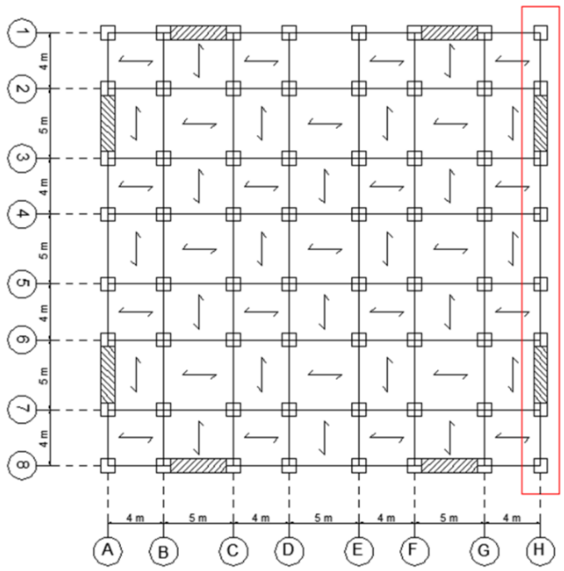


Fig. 2. Plan of buildings.

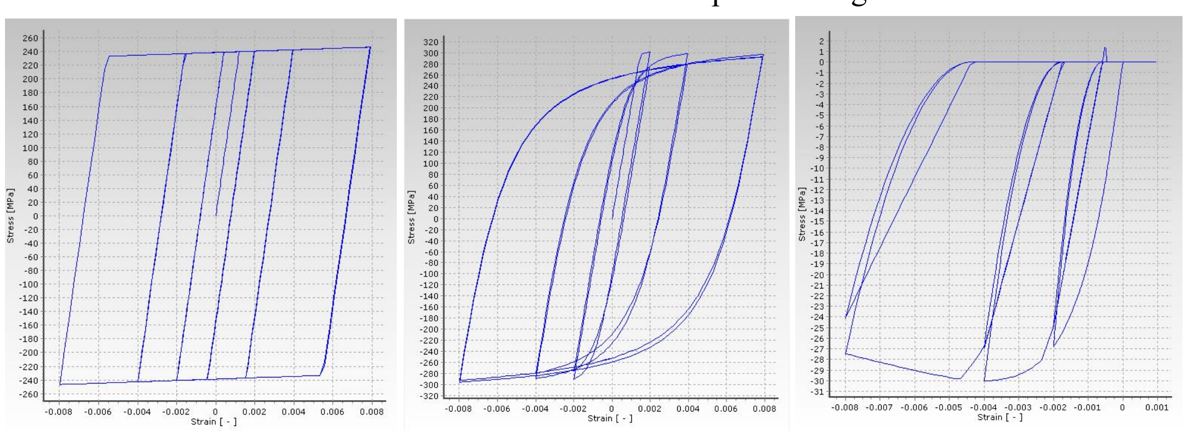
Deformation capacities of elements in models should not be less than maximum deformation demands at the target displacement. Element demands were within the acceptance criteria at the selected performance level of structure.

The characteristics of the walls applied to retrofit the frames are listed in table 2.

Table 2. Properties of shear walls.

Building	Longitudinal Bar ($\frac{mm^2}{m}$)		Transversal Bar ($\frac{mm^2}{m}$)		Thickness (cm)
	min	max	min	max	
5	2500	8648	500	500	20
10	3126	13712	625	1730	25
15	6125	17811	750	3980	30
20	7011	18724	826	6860	35

The columns and beams were modeled by force base control element. The walls were modeled by displacement base control element. The behavior of steel and concrete are depicted in figure 3.



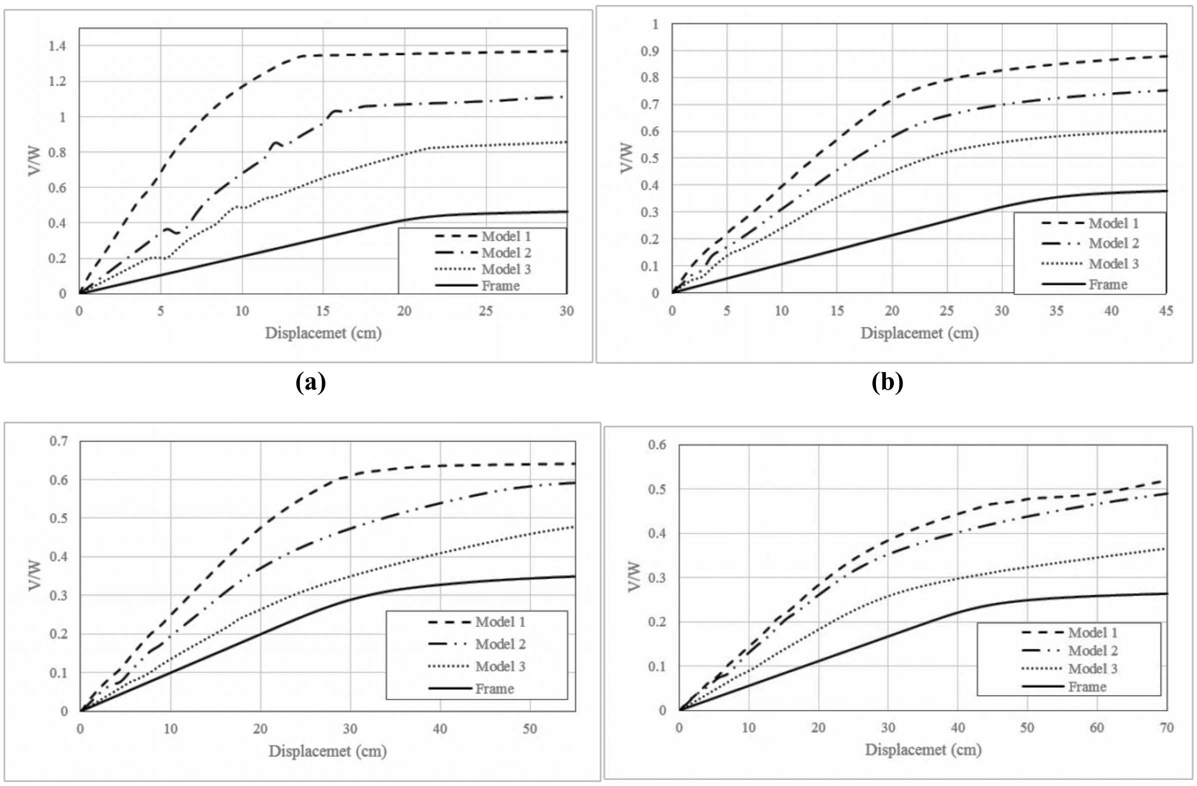
(a) (b) (c)

Fig. 3. The stress-strain curve of (a) steel in members, (b) concrete in walls, (c) steel in bars.

3. Adaptive Pushover analysis

Fundamental periods of each structure in this study did not fall in the constant-velocity portion of the response spectrum and did not exceed twice the site period. Moreover, pursuant to FEMA 440 [18], the behavior of buildings can be evaluated with nonlinear static pushover analysis (NSP).

The lateral load in NSP is applied statically and in increasing increments to the building to the point that the displacement in a certain location (control point) reaches a certain amount (target displacement) under the lateral load or the structure collapses. In the adaptive increasing pushover method, the load pattern at each stage of the analysis is considered in consonance with momentary modal characteristics of the adaptive model at dynamic characteristics of structures due to hinge formation and plastic deformations. Figure 4 portrays the adaptive pushover curves before and after rehabilitation.



(c) (d)

Fig. 4. Pushover curve of frames before and after retrofitting: (a) 5 story, (b) 10 story, (c) 15 story,(d) 20 story.

The increments of capacity and decrement of displacement at failure point of each model after rehabilitation is illustrates in table 3.

Table 3. Changes in models after rehabilitation.

Building	Displacement decrement (%)			Capacity increment (%)		
	Model1	Model2	Model3	Model1	Model2	Model3
5	14.6	20.4	38.6	187	249	300
10	27.6	32.5	37.5	158	202	231
15	14.5	18.4	23.6	137	169	189
20	15.6	17.8	22.2	129	182	193

The average increase in structure capacity after retrofitting for models with boundary elements, gapless wall, and gapped wall are 2.28, 2, and 1.54, respectively.

The displacement at failure point of the structures after retrofitting for wall models with boundary element, gapless, and gapped is 30.5, 22.3, and 18.1, respectively. The results demonstrate the best nonlinear static behavior of walls belongs to the model1.

4. Incremental Dynamic Analysis

In order to evaluate the nonlinear dynamic behavior of the models increasing dynamic analysis was used.

4.1. Earthquake Ground Motion Data Selection

PEER database records are chosen for this study [19]. Properties of the selected records were exhibited in table 4. Figure 5 illustrates the pseudo acceleration response spectra of the earthquake records.

Table 4. Properties of selection records.

No.	Event	Station	PGA (g)	R^d (km)	M^c	$^a\phi$
1	Northridge, 1994	Castaic Old Ridge Route	0.568	22.6	6.7	90
2		LA_116th St School	0.208	41.9		
3		Malibu Point Dume Sch	0.13	35.2		
4		LA Obregon Park	0.355	37.9		
5	San Fernando, 1971	Palmdale Fire Station	0.151	25.4	6.6	210
6		Pasadena_CIT Athenaeum	0.088	31.7		0
7		Upland San Antonio Dam	0.058	58.1		15
8		Wrightwood_6074 Park Dr	0.061	60.3		25

4.2. IDA Curves

Dynamic behavior diagram of 5 story buildings is portrayed in figure 5. Dynamic behavior diagram of buildings is presented in figures 6-9.

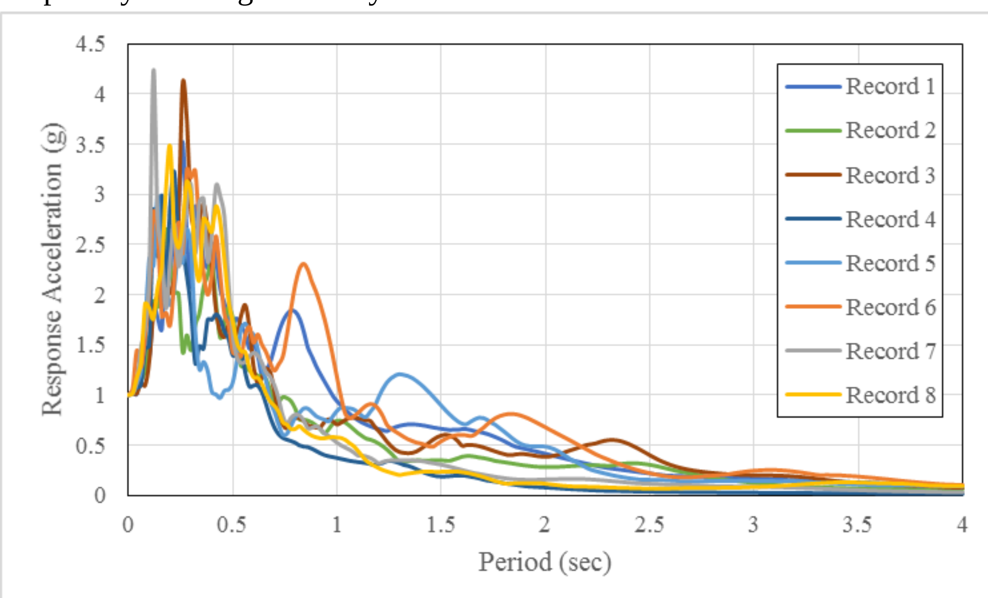


Fig. 5. Spectral acceleration of earthquakes.

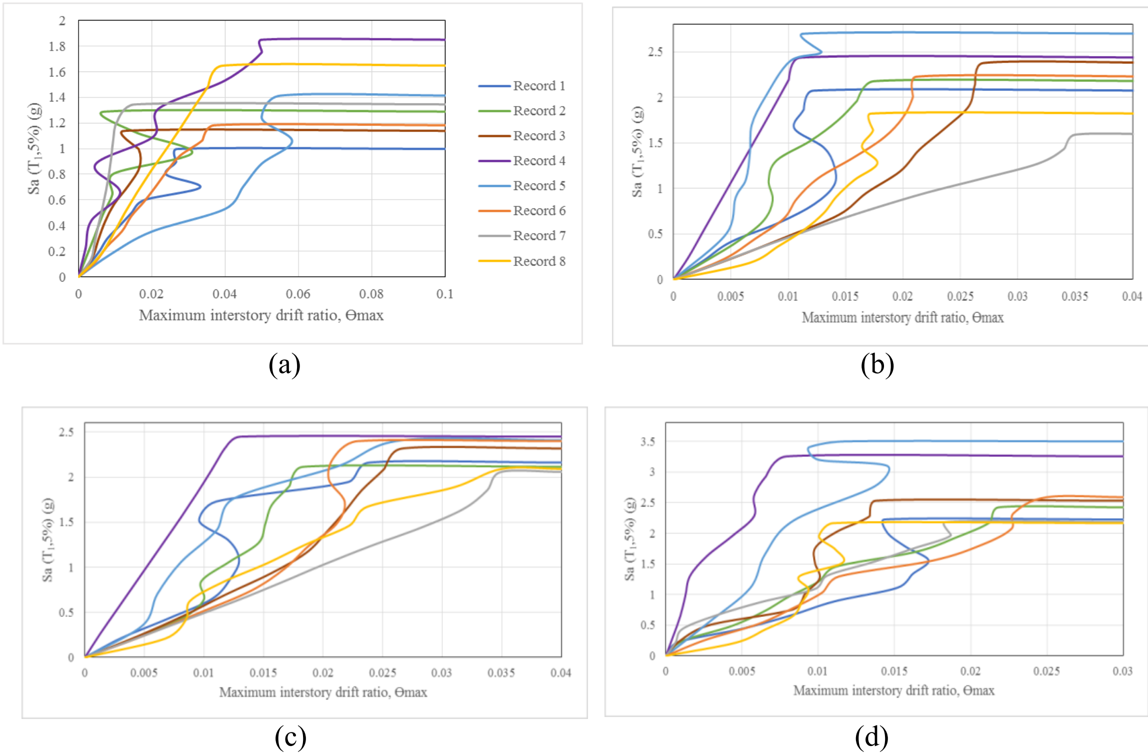


Fig. 6. IDA curve of 5 story building (a) before rehabilitation, (b) model1 (c) model2 (d) model3.

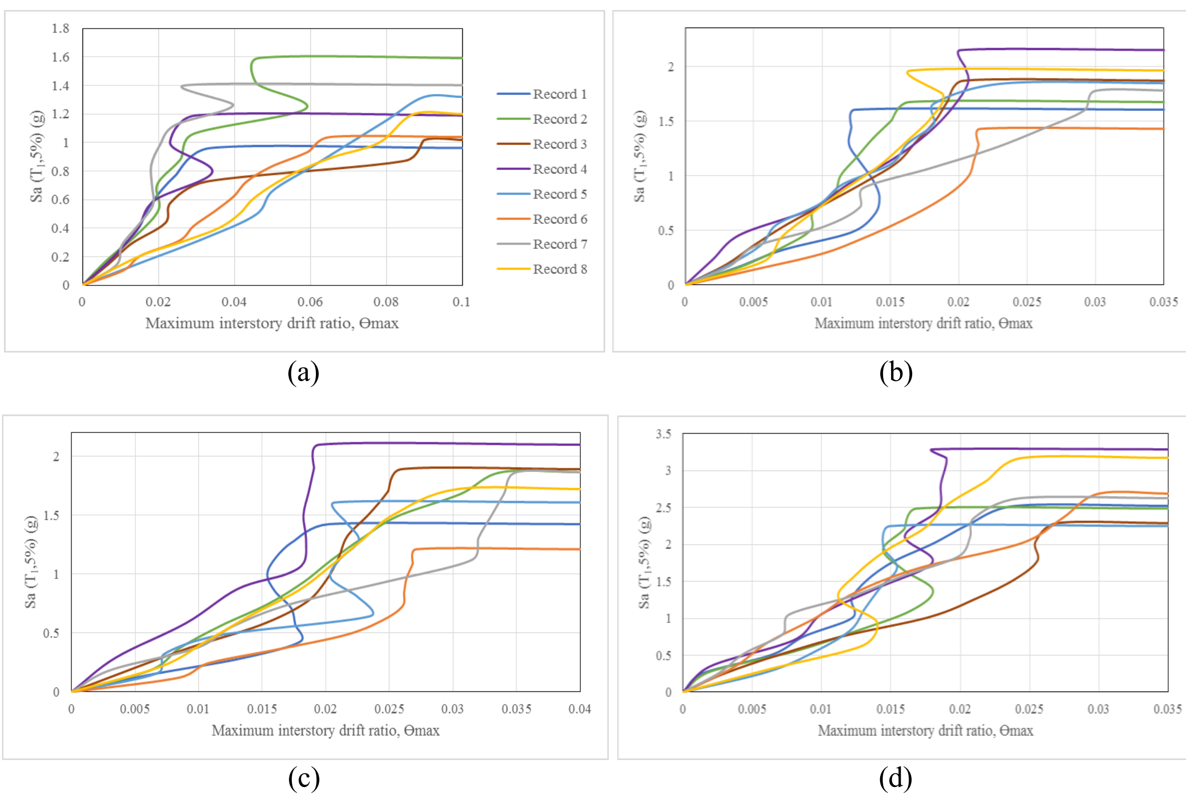


Fig. 7. IDA curve of 10 story building (a) before rehabilitation, (b) model1 (c) model2 (d) model3.

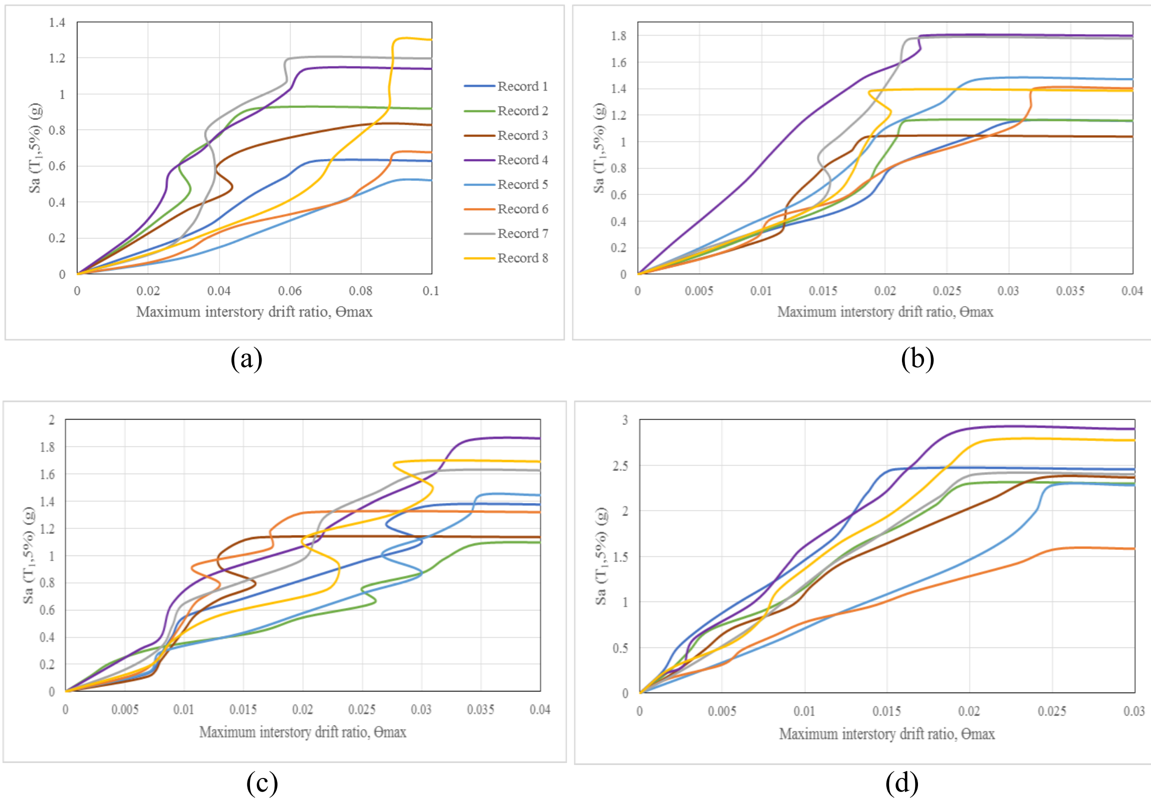


Fig. 8. IDA curve of 15 story building (a) before rehabilitation, (b) model1 (c) model2 (d) model3.

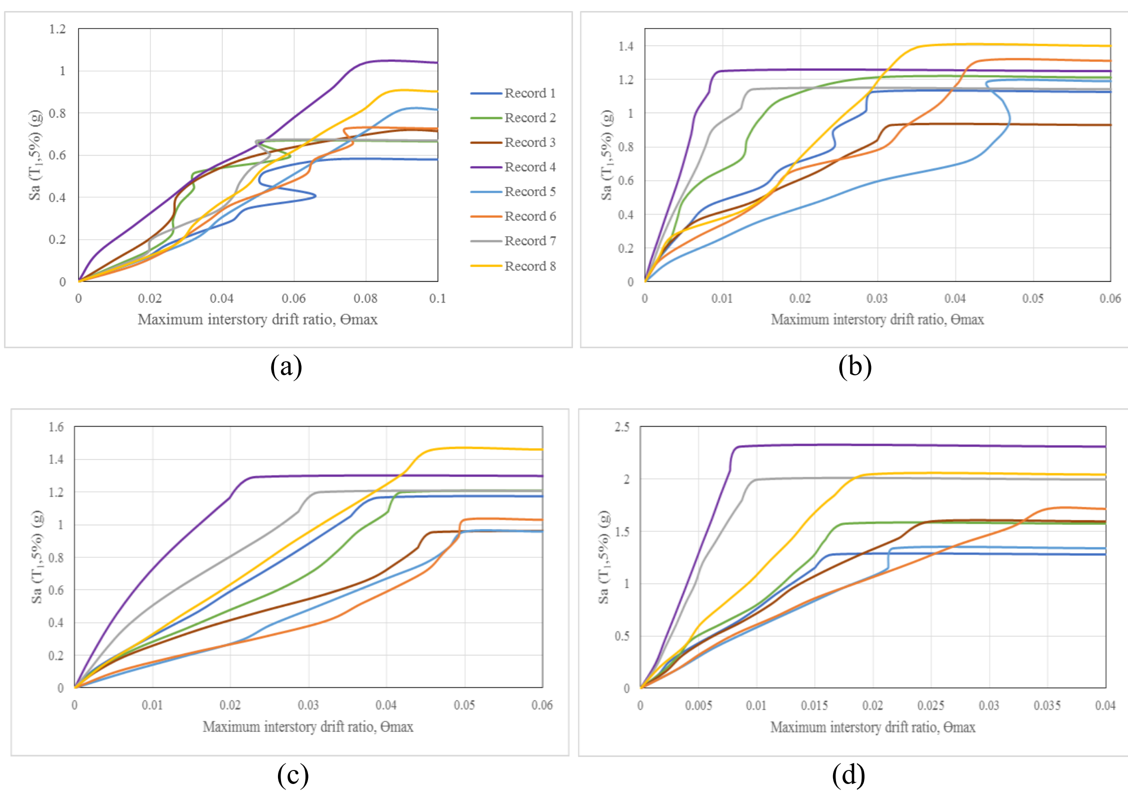
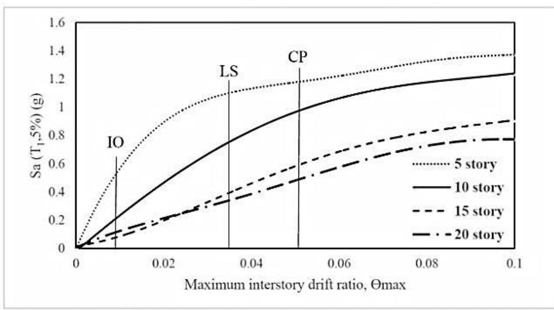


Fig. 9. IDA curve of 20 story building (a) before rehabilitation, (b) model1 (c) model2 (d) model3.

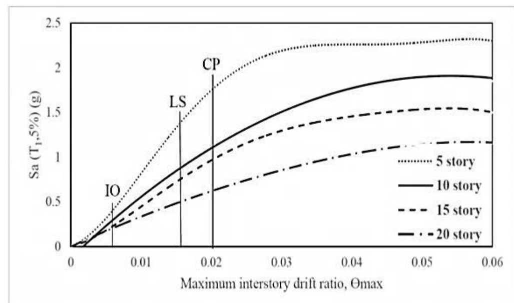
Drift average at failure point for 5 story building in model 1, 2, and 3 reduced 54.8%, 50%, and 31.2%, respectively. This value for 10 story building in model 1, 2, and 3 reduced 62.9%, 50%, and 56.5%, respectively. Drift average at failure point for 15 story building in model 1, 2, and 3 reduced 71.6%, 67.3%, and 61.3%, respectively. In 20 story building the drift average at failure point for model 1, 2, and 3 reduced 75.4%, 59.9%, and 46.5%, respectively.

The limit states on average IDA diagrams in different models are illustrated in figure 10. The value of Sa is highlighted for different performance levels in each model on 50% IDA at Table 5.

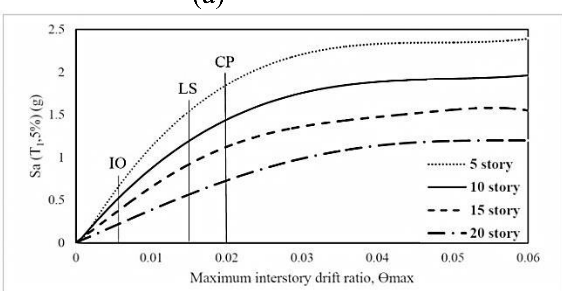
In order to depict various performance levels on this diagram at the vulnerable frame average relative drift for IO, LS, and CP is contemplated 0.7%, 3.5%, and 5%, respectively.



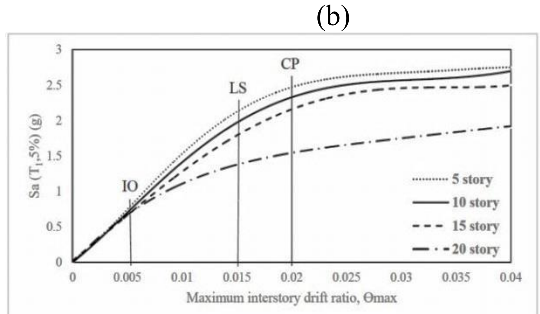
(a)



(b)



(c)



(d)

Fig. 10. The limit states on 50% IDA curves in (a) frame, (b) model1 (c) model2 (d) model3.

Table 5. The value of Sa on 50% IDA.

Building	Performance level	Frame	Model1	Model2	Model3
5 story	IO	0.44	0.48	0.46	0.74
	LS	1.09	1.53	1.32	2.06
	CP	1.17	1.95	1.74	2.48
10 story	IO	0.17	0.24	0.23	0.3
	LS	0.71	1.1	0.75	1.44
	CP	0.94	1.42	1.13	2.17
15 story	IO	0.08	0.19	0.18	0.69
	LS	0.38	0.81	0.69	1.76
	CP	0.56	1.08	0.99	2.12
20 story	IO	0.07	0.2	0.15	0.62
	LS	0.32	0.67	0.52	1.38
	CP	0.47	0.84	0.59	1.53

5. Determination of Building Fragility

Fragility curves are applied for evaluating the seismic behavior of rehabilitated models when the actual building damage data or any engineering notion is not available. In this technique, buildings are modeled and earthquake records with different intensity levels are taken into account for simulating the seismic failure in buildings by executing many analyses. Seismic fragility curves are conditional probability functions which give the probability of a model attaining or exceeding a certain damage level for an earthquake with a specific intensity level [20,21].

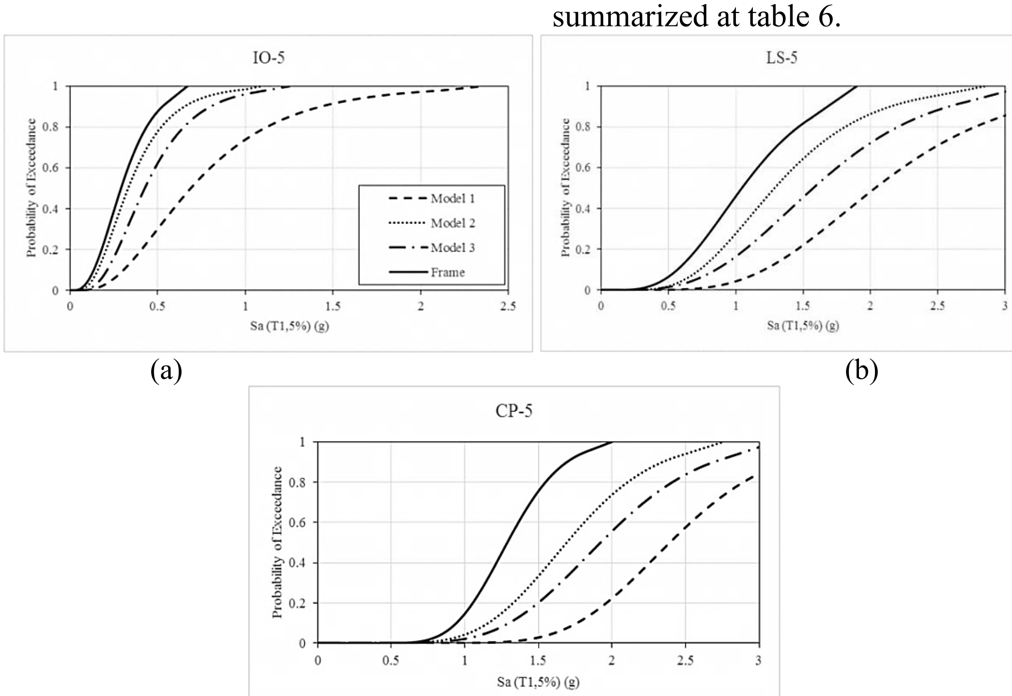
A specific technique that relies on component-based evaluation of the building models. The following steps outline the methodology;

- Create the analytical model of each building and obtain the response quantities under each seismic record.
- Define the damage limit states and corresponding demand parameters.
- Determine the performance levels
- Assess the global performance level of models for the selected seismic record.
- Determine the exceedance probabilities of each specified damage level for earthquake records.
- Determine the fragility curves for damage levels and models by curve fitting.

The fragility curves are grouped separately in Figures 10-13 for three performance level (IO, LS, and CP) to compare the effect of different retrofitting models on the fragility curves.

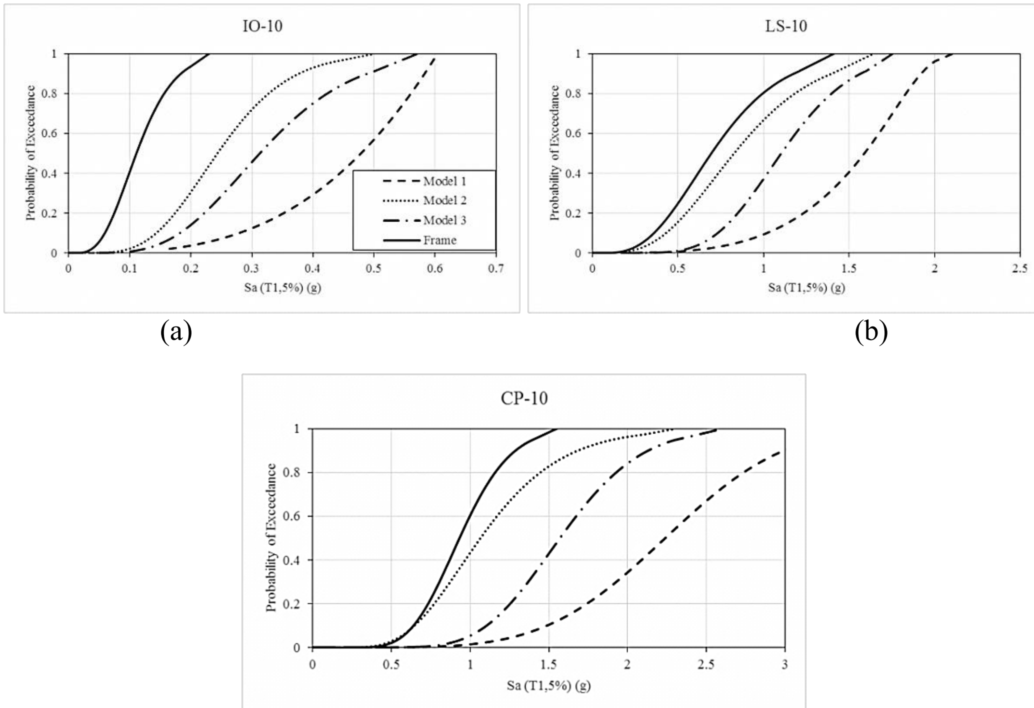
The fragility curves method, is the perfect way to gain determination over the probabilistic damage of the structures. From the fragility curves in figures 11-14, all the data in the entire input motion zone witch the minimum error ignoring the details can be read.

The damage exceedance probability under various performance levels at 0.35g is summarized at table 6.



(c)

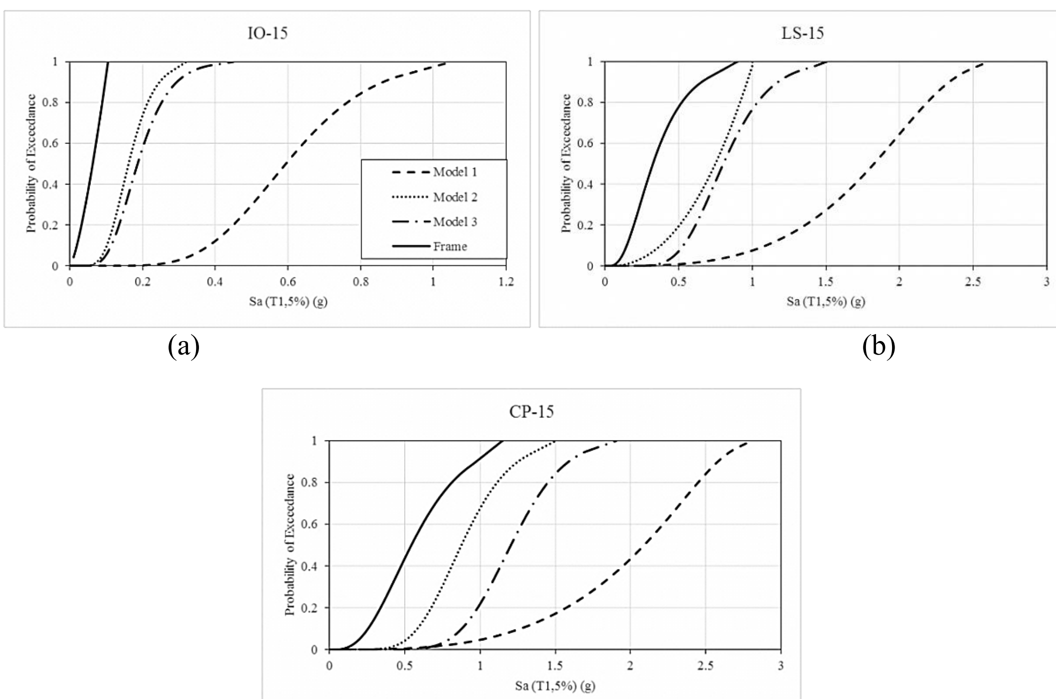
Fig. 11. Fragility curves of 5 story building in the performance level: (a) IO (b) LS (c) CP.



(a) (b)

(c)

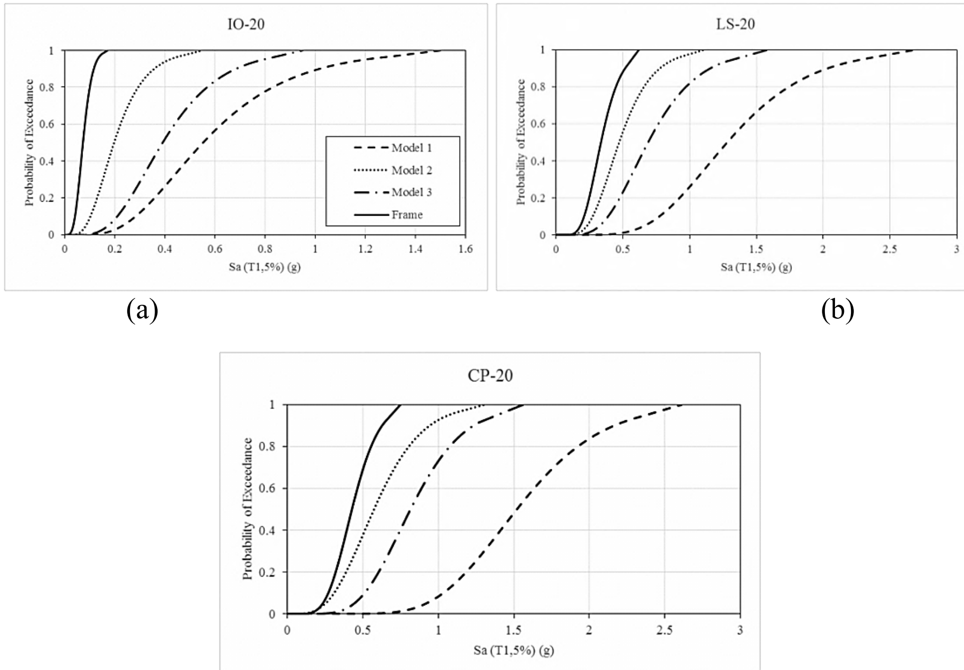
Fig. 12. Fragility curves of 10 story building in the performance level: (a) IO (b) LS (c) CP.



(a) (b)

(c)

Fig. 13. Fragility curves of 15 story building in the performance level: (a) IO (b) LS (c) CP.



(a) (b)

(c)

Fig. 14. Fragility curves of 20 story building in the performance level: (a) IO (b) LS (c) CP.

Table 6. Damage exceedance probability.

Building	Performance level	frame	Model1	Model2	Model3
5 story	IO	64	13	53	35
	LS	3	0.0	0.0	0.0
	CP	0.0	0.0	0.0	0.0
10 story	IO	100	20	86	63
	LS	11	0.0	8	0.0
	CP	0.0	0.0	0.0	0.0
15 story	IO	100	70	100	95
	LS	65	0.0	9	0.0
	CP	22	0.0	0.0	0.0
20 story	IO	100	17	91	40
	LS	54	0.0	22	7
	CP	25	1.53	16	2
Mean	IO	91	14.2	82.5	58.2
	LS	33.2	0.0	9.7	33.2
	CP	11.7	0.0	4	11.7

According to table 6, the difference between the fragility curves of all the retrofitting models for the IO limit state is more than CP and LS limit states. Using the shear wall with boundary elements has the best dynamic behavior at all performance levels, and also the gapped shear wall has a better performance compared to the gapless wall.

6. Conclusion

In this study, vulnerable buildings with moment resistant steel frames are rehabilitated using the concrete shear walls in three models (wall gapped from the columns, connected to the columns, and with boundary elements). The nonlinear behavior of these buildings is evaluated employing nonlinear static adaptive pushover analysis. The nonlinear static analysis results depict that capacity of structure is increasing by adding the shear wall which it's a significant difference in many occasions. This increase is the highest for the building retrofitted with the wall with boundary elements compared to the other models. In the gapless wall model, compared to the gapped wall there is more capacity increase and the displacement corresponding to failure has no significant difference.

In consonance with the nonlinear behavior of models, it can be deduced that the shear wall with boundary elements has the best dynamic and static performance at all performance and hazard levels. Although employing the shear wall connected to the columns has increased the structure capacity in static analysis compared to the gapped wall, yet the nonlinear dynamic behavior of the wall with gap is better than the gapless wall and at all hazard levels it performs better.

The first option to retrofit steel structures is applying wall with boundary element and then wall with gap is advisable, which the gap in that case can be filled with a flexible material. For future studies, the usage of dampers to connect wall and column is suggested.

REFERENCES

[1] Judd, J. P., Charney, F. A. (2016). Seismic collapse prevention system for steel-frame buildings. Journal of Constructional Steel Research, 118, 60-75.

[2] Kim, J., Lee, J., Kang, H. (2016). Seismic retrofit of special truss moment frames using viscous dampers, Journal of Constructional Steel Research, 123, 53-67.

[3] Qu, B., Sanchez-Zamora, F., Pollino, M., & Hou, H. (2017). Rehabilitation of steel concentrically braced frames with rocking cores for improved performance under near-fault ground motions. Advances in Structural Engineering, 20(6), 940-952.

[4] Morelli, F., Piscini, A., & Salvatore, W. (2017). Seismic behavior of an industrial steel structure retrofitted with self-centering hysteretic dampers. Journal of Constructional Steel Research, 139, 157-175.

[5] Sivandi-Pour, A., Gerami, M., Kheyroddin, A. (2015). Determination of modal damping ratios for non-classically damped rehabilitated steel structures, Iranian Journal of Science and Technology, Transactions of Civil Engineering, 39(C1), 81.

[6] FEMA 356, (2000). Federal Emergency Management Agency, Pre-standard and Commentary for the Rehabilitation of Buildings.

[7] Sivandi-Pour, A., Gerami, M., Khodayarnezhad, D., (2014). Equivalent modal damping ratios for non-classically damped hybrid steel concrete buildings with transitional storey. Structural Engineering and Mechanics, 50(3), pp.383-401.

[8] Gerami, M., Sivandi-Pour, A. (2014). Performance based seismic rehabilitation of existing steel eccentric braced buildings in near fault ground motions. The Structural Design of Tall and Special Buildings, 23(12), 881-896.

[9] Di Sarno, L., Elnashai, A. S. (2009). Bracing systems for seismic retrofitting of steel frames. Journal of Constructional Steel Research, 65(2), 452-465.

[10] Kurata, M., Leon, R. T., DesRoches, R., Nakashima, M. (2012). Steel plate shear wall with tension-bracing for seismic rehabilitation of steel frames. Journal of Constructional Steel Research, 71, 92-103.

[11] Shakib, H., Joghan, S. D., Pirizadeh, M., Musavi, A. M. (2011). Seismic rehabilitation of semi-rigid steel framed buildings—A case study. Journal of Constructional Steel Research, 67(6), 1042-1049.

[12] Jiang, L., Zheng, H., Hu, Y. (2017). Experimental seismic performance of steel-and composite steel-panel wall strengthened steel frames. Archives of Civil and Mechanical Engineering, 17(3), 520-534.

[13] Mirza, O., Shill, S. K., Mashiri, F., Schroot, D. (2017). Behaviour of retrofitted steel structures using cost effective retrofitting techniques. Journal of Constructional Steel Research, 131, 38-50.

[14] TahamouliRoudsari, M., Entezari, A., Hadidi, M., & Gandomian, O. (2017). Experimental assessment of retrofitted RC frames with different steel braces, Structures 11, 206-217

[15] SeismoSoft, Inc., SeismoStruct – Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures, Downloadable software with registration from URL: http://www.seismosoft.com, 2012.

[16] AISC, (2010). American Institute of Steel Construction. Steel construction manual, 14th ed. Third Printing. Chicago, IL.

[17] ASCE 7-10, (2010). Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers, Reston, Virginia.

[18] FEMA440. (2005). Improvement of nonlinear static Seismic analysis procedures. Federal Emergency Management Agency. Washington, DC, USA.

[19] The Pacific Earthquake Engineering Research Center (PEER), https://ngawest2.berkeley.edu

[20] FEMA P695, (2009). Quantification of building seismic performance factors. Washington DC, USA: Federal Emergency Management Agency.

[21] Ji, X., Wang, Y., Zhang, J., & Okazaki, T. (2017). Seismic behavior and fragility curves of replaceable steel coupling beams with slabs. Engineering Structures, 150, 622-635.