Investigation of Seismic Vulnerability of Reinforced Concrete Frames Equipped with Steel Plate Shear Walls

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Hamze Rouhi*, Majid Gholhaki

Department of Civil Engineering, Central Administration of Semnan University, Semnan University, Semnan, 3513119111, Islamic Republic of Iran.

*Email: h.rouhi@semnan.ac.ir; Corresponding Author.

ABSTRACT

There are various methods in order to assessing the seismic vulnerability of buildings that vary in cost and accuracy. In this regard, fragility curves, which consider the probability of structural damage as a function of ground motion characteristics and design parameters, are more common. These curves express the probability of the structural response exceeding the functional limit at different seismic intensities. The FEMA-P695 analytical method evaluates the existing uncertainties in assessing the seismic behavior of structures using collapse fragility curves. In this research, the seismic vulnerability of a relatively new system of reinforced concrete frames (RCF) equipped with steel plate shear wall (SPSW) is evaluated using this method. Relationships and criteria proposed by FEMA-P695 are investigated employing four models with 4, 8, 12, and 24 stories subjected to the records proposed by the FEMA-P695. According to the results of this study and evaluation of effective parameters in the assessment of collapse safety margin, the RCF equipped with SPSW shows acceptable performance against earthquake and maximum earthquake probability.

Keywords: Reinforced concrete frame; Steel plate shear wall; Incremental dynamic analysis; Fragility curves; FEMA-P695.

INTRODUCTION

Fragility curves indicate the probability of structural failure for different levels of earthquake intensities and are used to prioritize the structures for retrofit by determining the vulnerability of the structures. In 1980, the fragility curves were first drawn for nuclear power plants (Kennedy, R.P et al., 1980). These curves were plotted using the factors of water pressure fragility, concrete strength, displacement, and stress in the shells of reservoirs based on different levels of peak ground acceleration (PGA). After the Northridge earthquake (1994), more attention was paid to estimating the amount of damage to structures. The focus was placed on predicting the amount of financial damage to structures in the event of more severe earthquakes. In 1994, during a study considering California structures, the ATC-13 criteria were used to plot fragility curves (Anagnos, T et al., 1994). Shin et al. (2014), investigated the change in structural performance due to the occurrence of seismic sequencing phenomenon by analyzing the fragility of RCF.

SPSW is a type of system suitable for earthquake and wind lateral loads consisting of a series of separate panels, each panel is enclosed inside two beams and columns and is considered connected to the surrounding elements (Astaneh-Asl, A,2001). As an effective lateral system in seismic rehabilitation, it increases the lateral strength and stiffness of buildings against earthquake (technical rehabilitation strategies) in steel structures. Recently, according to the analysis and design of SPSW, which requires boundary structural elements (beams and columns) with high rigidity, to increase the lateral strength and stiffness of concrete buildings with a moment-resisting frame that inherently has such elements are used.

The method of incremental dynamic analysis (IDA) (Vamvatsikos, D et al., 2002) was first proposed in 2000 by Professor Cornell at Stanford University, and then was examined for a 20-story building during Vamvatsikos ' project by Professor Cornell in 2002. Incremental dynamic analysis (IDA) is a nonlinear dynamic analysis that can be used to determine the amount of damage to the structure in terms of the intensity of the earthquake.

Recently, the RCF with SPSW has been proposed as a new system against lateral loads (Tarkan, G et al., 2012).

The paper entitled "Use of knowledge to reduce vulnerability to seismic hazards" reports the ongoing activities carried out jointly by the federal government and academia on the development of inventory and seismic and flood risk analysis tools (M. Nastev et al., 2015).

Quantification of building system performance and response parameters such as response modification coefficient (RR), system over strength factor, and deflection amplification factor is of great importance to reliably assess the performance of RC walls. To assess the validity of these parameters, 20 special and 20 ordinary RC walls, with varying seismic design conditions and physical parameters, were designed and modeled in a software framework (Gogus, A et al., 2015). Static pushover and incremental dynamic analyses using 44 ground motions were conducted on each archetype using standard methodology, and the obtained adjusted collapse margin ratios are compared to established limits. Results indicated that the parameters of the archetypes designed based on current code provisions were within reasonable limits except for archetypes having height-to-length aspect ratios of 3 or greater, where the use of a larger RR value was suggested.

The documentation of 111 buildings built between 1962 and 1987 from various parts of the city of Osijek were gathered (Hadzima - Nyarko, M et al., 2016). This research aims to provide the first steps in assessing seismic risk in Osijek by applying a method based on vulnerability index. According to this method, defining five damage states, the action is expressed in terms of the macro seismic intensity and the seismic quality of the buildings through a vulnerability index. The value of the vulnerability index can be changed depending on the structural systems, quality of construction, etc., by introducing behavior and regional modifiers based on expert judgments. Depending on the proposed modifiers, the seismic vulnerability of existing buildings in the city of Osijek was assessed. The resulting vulnerability of the considered residential buildings provided necessary insight for emergency planning and the identification of critical objects vulnerable to seismic loading. The possibility of creating an "ID card" for each new and existing building in urban areas, based on the transfer of necessary data into electronic format, is analyzed (Işık, M. F et al., 2018).

To study the seismic performance of school buildings, which have been built by template unreinforced masonry school projects in Albania, the most widely used two template designs which were damaged during the 2019 Durrës (Albania) Earthquakes, have been selected (Hysenlliu, M et al., 2020). Analytical models of each school were prepared following the experimental data on the quality of the masonry constitutive components of the selected school buildings. Geotechnical investigations were deployed to obtain the soil characteristics of the area where the schools' foundations are located. Nonlinear static analyses have been performed to obtain the seismic capacity, the performance point, and the damage level states. The performance-based method has been used for that purpose. The detailed examination of capacity curves and performance evaluation identified deficiencies and weak parts of the school building blocks. Results have shown that existing school buildings constructed pre-modern codes are far from satisfying the required performance criteria, suggesting that urgent response and necessary measures should be put into action.

A 6.8-magnitude earthquake that occurred on January 24, 2020, with PGA 0.292g, has been effective in Turkey's eastern regions, causing the destruction or heavy damage of buildings, especially in the city center of Elazığ province (Dogan, G et al., 2021). In this research, the damages observed in especially RC buildings as well as in masonry and rural buildings were summarized, the lessons learned were evaluated, and the results were interpreted concerning Turkish earthquake codes.

To assess of seismic vulnerability of existing buildings, various approaches and methodologies were developed in different countries to overcome the disastrous effects of earthquakes on the structural parameters of the building and the human losses (Harirchian, E G et al., 2021). There are structures still in service with a high seismic vulnerability, which proposes an urgent need for a screening system's damageability grading system. Rapid urbanization and the proliferation of slums give rise to improper construction practices that make the building stock's reliability ambiguous, including old structures that were constructed either when the seismic codes were not advanced or not enforced by law.

As a new system, reinforced concrete frames equipped with steel plate shear wall system needs more numerical and experimental studies. This research is also part of the researches to better understand this system. Numerous studies have been done on a type of structural system, including resisting moment frames, braced systems, and concrete shear walls. Although, the RCF with SPSW has been studied from various aspects recently, the failure behavior on this system has not been studied yet.

In the present study, three groups of seismic records, are used including group I, containing 44 records of far-field proposed by FEMA-P695 (FEAMA-P695, 2009**)**, group II, consist of 28 records of near-field with pulse proposed by FEMA-P695 and group III, including 28 records of near-field without pulse proposed by FEMA-P695. Landers accelerometer seismic record was presented In Figure 1.

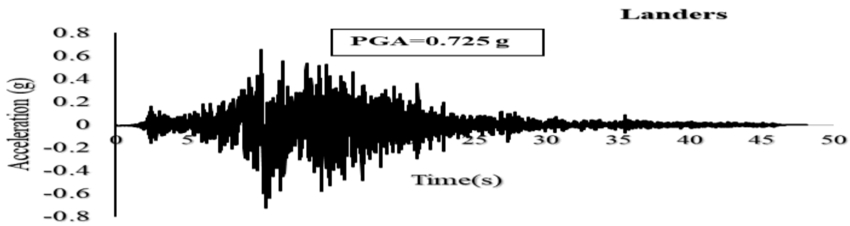
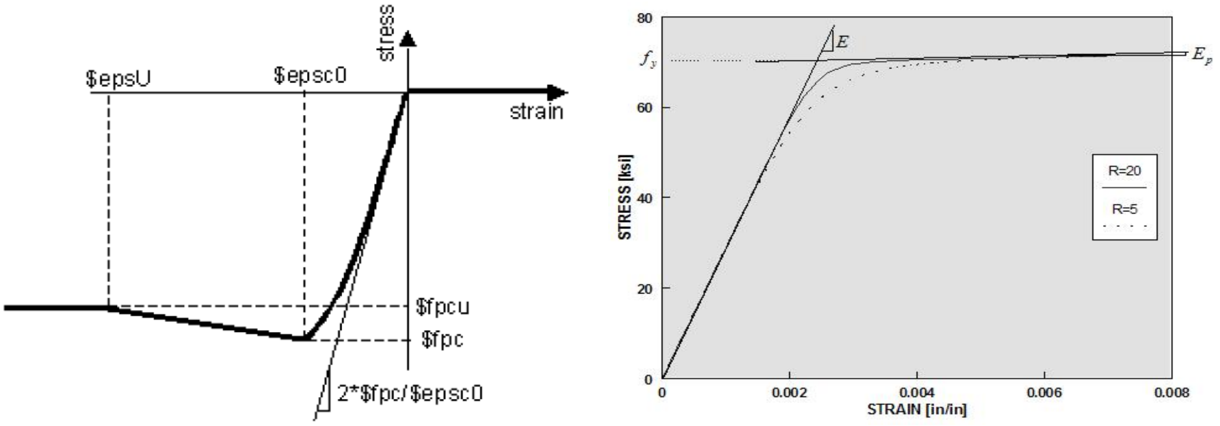


Figure 1 Seismic record Landers accelerometer.

Validation of RCF equipped with SPSW

In this research, to validate the numerical process, the experimental study of Choi and Park (2011) shown in Figure 3a has been used. Choi conducted an experimental study to investigate the cyclic behaviour of walls consisting of boundary elements of RCF and thin steel sheets. To ensure the accuracy of the modelling, the numerical model of the laboratory sample was modelled and analyzed in OpenSees finite element software (Mazzoni, S el al., 2006**)** and (McKenna, F el al., 2006). For modelling, the Nonlinear-Beam-Column element has been used for the beam and column elements with deformation control, which can take into account the P-Δ effects and large deformations. The strip method has been used to model the steel plate (Sabouri S., 2000). In this method, a truss element is used to model the tensile strips. To model the wide plasticity of the elements in the program, the cross-sections of the beam and column elements are divided into a number of fibers. Concrete01 and Steel02 materials (see Figure 2) have been used for modelling concrete and steel materials of reinforcements, respectively. To model the actual behaviour of the strips that should not react when pressed, Hysteretic materials are used, which with the three-line behaviour in tension and pressure gives the strips the property that does not show resistance when under pressure and allows that the diagonal tensile field of a steel shear wall is well modelled. Also, the discussion of concrete confinement of columns is accounted for in the model. Numerical results from cyclic loading are compared with experimental results (see Figure 3b). The values of load-bearing capacity, initial stiffness, and energy absorption determined from the experiment and the corresponding simulated model are presented in Table 1. The comparison between the two diagrams in Figure 3b shows the acceptable accuracy in the numerical modelling process of this research.

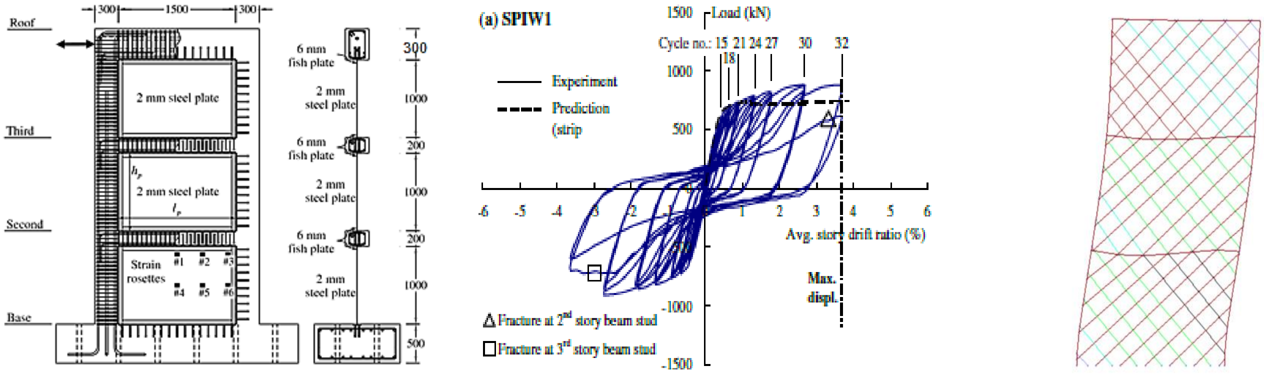


(a) (b)

Figure 2 Stress-Strain relationships (a) Concrete01 (b) Steel02

Table 1 Comparison of Finite Element Analysis Results and Choi and Park Model Test (Choi, I et al., 2011).

Lateral Load (KN)			Elastic Stiffness (KN/mm)			Energy Dissipation (KN.M)		
Test	Finite Element	Ratio Finite Element To Test	Test	Finite Element	Ratio Finite Element To Tes	Test	Finite Element	Ratio Finite Element To Test
886	903	1.02	53	48	0.91	323.98	349.23	1.08



(a) (b) (c)

Figure 3 (a) Geometric and reinforcing details of the RCF equipped with SPSW (Choi, I et al., 2011). (b) Hysteresis curve of numerical model and Choi and Park laboratory sample (Mazzoni, S el al., 2006**)**. (c) Deformed shape of numerical model (Mazzoni, S el al., 2006**)**.

Modelling

Finite Element models in ETABS software

Regarding the classification of structural systems, some have considered the ratio of the height to the smallest horizontal dimension of the structure as a criterion for classifying buildings, and height to the smallest dimension ratios greater than 1.5 π, between π and 1.5 π, between π and 0.5 π and less than 0.5 π are known as super high, high, mid and low-rise buildings, respectively (Stafford Smith et al., 1991) Accordingly. In this research, four models of 4, 8, 12, and 24 stories with height to the smallest dimension ratios of 0.54, 1.09, 1.63, and 3.26 are simulated. They are classified as low, low, mid, and high-rise structures with a rectangular plan according to Figure 4 a, including lateral resistant system of RCF with SPSW. The stories of the models are 3.4 meters height and the roof is considered as a block joist. The construction site of the structures was considered to be a high earthquake risky region and soil type was assumed III. Concrete used in simulating structural elements, has a characteristic strength of 220 kg/cm^2 (C22) and are reinforced with rebars of type A3 with a yield stress of 4000 kg/cm^2. The steel used for equivalent braces was St37 with a yield stress of 2400 kg/cm^2. In the analysis and design of the studied structures, the sixth (Iranian National Building Code. Applied Loads on Buildings, 2013**)** and ninth (Iranian National Building Code. Design and Implement of Concrete Buildings, 2013**)** national building regulations and the IS 2800 earthquake standard, fourth edition (Code IS. Iranian Code of Practice for Seismic Resistant Design of Buildings 2800, 2014**)** 4th ed. Tehran, Iran: Ministry of Roads & Urban Development.) have been used. The dead load of stories and roof was considered 640 kg/m^2, the live load of stories and roof was applied 200 kg/m^2 and the load of walls of stories was assumed 600 kg/m, according to the sixth chapter of the National Building Regulations. For the design of structures, according to the results of experimental study performed on a RCF with steel plate shear wall and the criteria set by the American loading code ASCE07-2010 (American Society of Civil Engineers (ASCE), 2010) for steel plate shear wall, the behaviour factor 7 for the reinforced concrete equipped with SPSW lateral system was used. To design thin SPSW, according to Canadian and American steel regulations, an equivalent brace is considered instead of each steel plate, and after calculating the cross-sectional area of each brace, the thickness of the steel plate is calculated from equation (1).

$$t = \frac{2A_b \sin\theta \cos 2\theta}{\mathrm{L} \sin^2 2\alpha} \quad (1)$$

Where θ is the angle between the brace and the column, L is the span of the frame, A_b is the equivalent cross-sectional area of the brace, α is the angle of formation of the diagonal tensile field in the steel plate

After determining the thickness, each plate is converted into a number of diagonal strips, the cross-section of each strip A_s is obtained from equation 2:

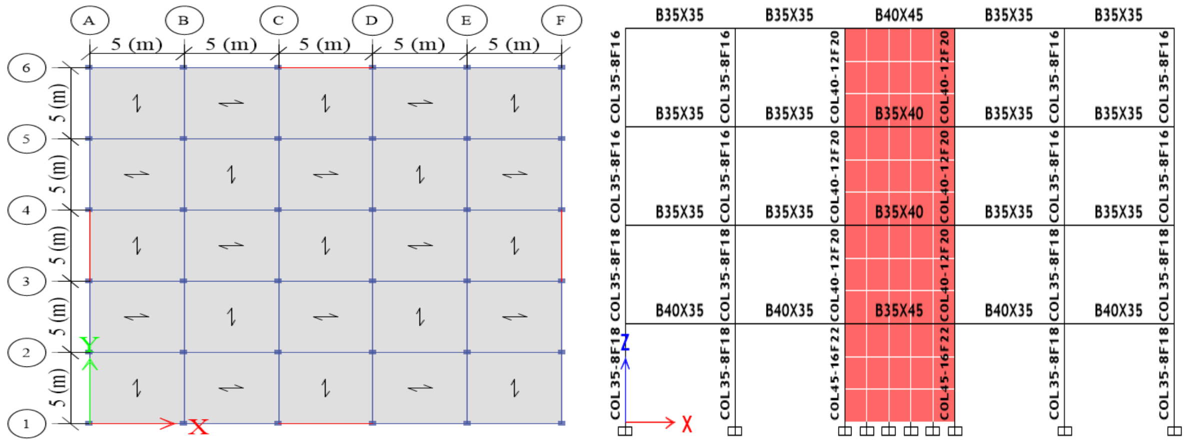
$$A_s = \frac{L\,cos\alpha + L\,sin\alpha}{n}.t \qquad (2)$$

Where n is the number of bars. Numerous studies have been performed on the number of required strips, the results of which indicate the adequacy of 10 diagonal strips for the analysis of a thin steel shear wall. Given that the columns may buckle under the influence of the diagonal tensile field, the stiffness of the columns should be controlled by equation 3.

$$I_c \geq \frac{0.00307\,t\,h_s^4}{L} \qquad (3)$$

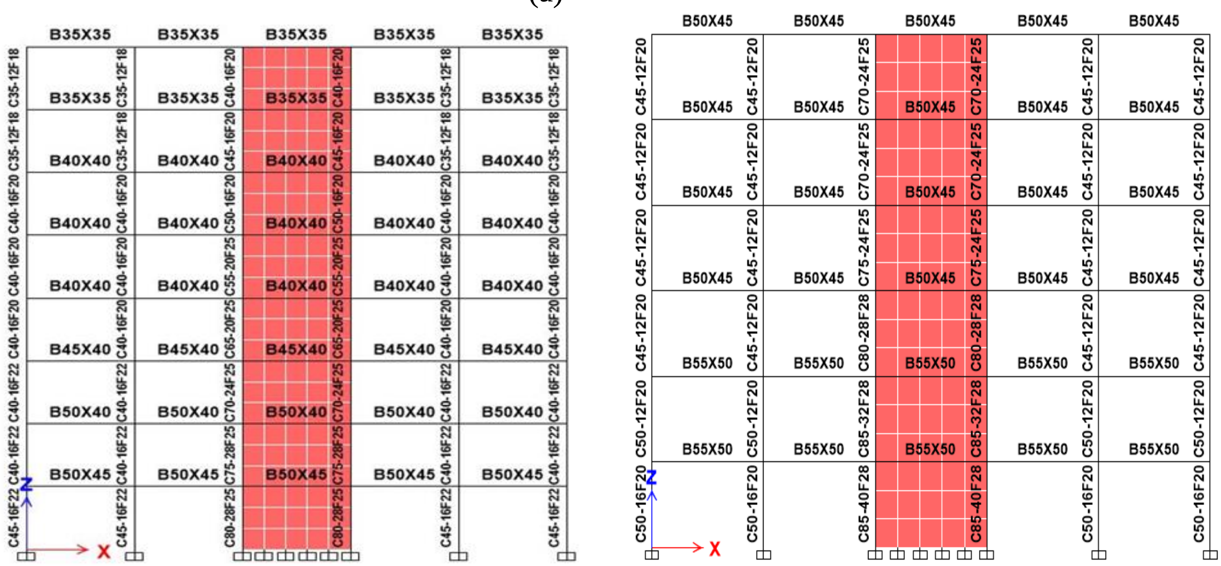
Where I_c is the moment of inertia of the columns and h_s *is* story height. Also, in order to prevent the bending of the upper beam of the SPSW due to the effects of the asymmetric tensile field, equation 4 must be controlled:

$$M_{fpb} \geq \frac{\sigma_{ty}\,t\,L^2}{8}\sin^2\alpha \qquad (4)$$



(b)

(a)



(d)

(c)

Figure 4 a) plan of structural models (b) 4-story model (c) 8-story model (d) 6 first stories of 12-story model.

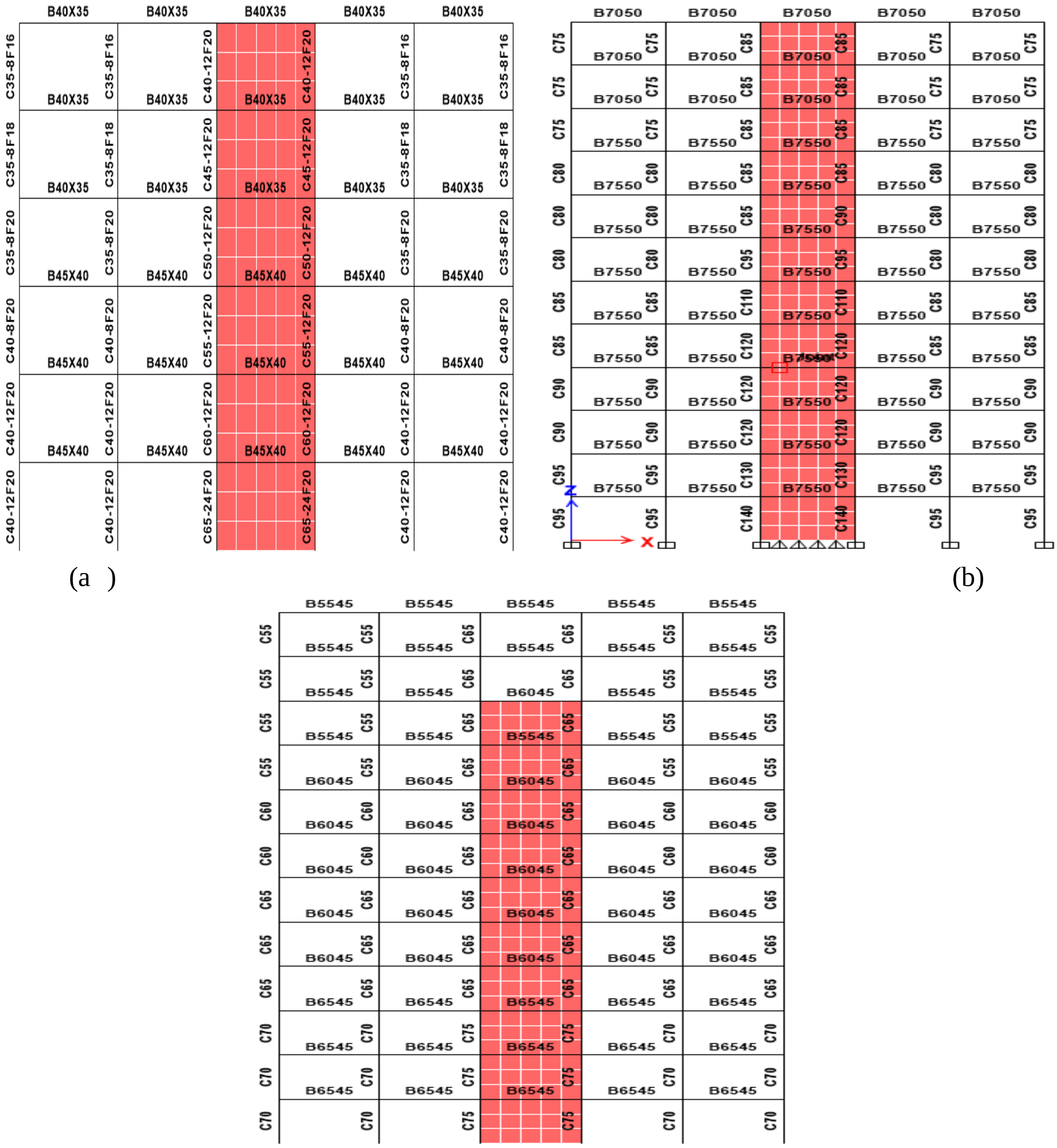
Where M_{fpb} is the plastic anchor of the cross-section of the beam and $\sigma_{ty}$is the final stress of the diagonal tensile field, which for thin plates is equal to their yield stress. Due to the small difference in the intensity of the diagonal tensile field between two adjacent stories, control of this relationship is required only for the end beam, but if the difference between the diagonal tensile fields between two adjacent stories is large, the relationship should be controlled for the middle beams. To ensure that the perimeter columns can withstand the stresses due to the environmental loads along with the stresses due to the tensile field effect, it is necessary to check equation 5 for the plastic anchor of the columns:

$$M_{fpc} \geq \frac{\sigma_{ty}\, t\, h^2}{4} \cos^2\alpha \qquad (5)$$

Equivalent tensile bracing was used to model the SPSW in the design stages due to the impossibility of modelling the steel plate in the model geometry and its analysis by ETABS software. In the last two stories of the 24 story model, due to the negative shear in the braces, the use of SPSWs were avoided and only the special RCF system was used to deal with the lateral force. Designed structural sections of 4, 8, 12, and 24 story models are presented in Figures 4 and 5.

Modelling in OpenSees

In order to modeling of the beam and column elements type (Nonlinear Beam-Column) the ability to take into account the P-Δ effect and large deformation have been used. Strip method is used to modeling of SPSW. For the concrete and steel reinforcement materials respectively Concrete01 and Steel02 materials have been used. For providing the actual behavior of strips the materials hysteretic is used which allows the use of diagonal tension steel plate shear. For example, the bracing of the 4- story is presented in table 2. In the 8-story model, the thickness of the plate is 2 mm for stories one to six, 1.6 mm for the seventh story and 1.1 mm for the eighth story. In the 12-story model, the thickness of the plate is 1.1 mm for the first to fourth stories, 1.8 mm for the fifth to eighth stories, 1.6 mm for the ninth and tenth stories, 1.4 mm for the eleventh story. And for the twelfth story is 1.1 mm. In the 24- story model, the thickness of the plate in the first story is 4.4 mm, in the second to eighth stories, 3.2 mm, in the ninth to fifteenth stories, 2.9 mm, in the sixteenth story, 2.6 mm, in the seventeenth to eighteenth stories, 2.1 mm, in the nineteenth to twentieth stories, 1.8 mm, in the twenty-first and twenty-second stories, 1.6 mm, in the simplest and third stories, 1.4 mm and on the twenty-fourth story, 1.1 mm was calculated and used in the analysis. Also, concrete confinement of columns is considered in the model. OpenSees uses the distributed plasticity by the fiber element. Regarding geometric nonlinearity, it should be said that the effects of geometric nonlinearity are defined by the transfer matrixes that are a feature of OpenSees. In the mentioned program, after defining the geometry of the model,



(a) (b)

(c)

Figure 5 Structural sections of models (a) 6-second stories of 12-story (b) 12 first stories of 24 story (c) 12-second stories of 24story.

the gravitational analyzes are performed (non-linear static) and by setting the time in the amplitude of the problem to zero before performing the nonlinear dynamic analysis, the gravitational load values remain constant in subsequent dynamic analyzes. The effect of P-Δ is considered in the analyses. 5% damping was applied for all models.

Table 2 Summary of calculations of bracing equivalent for 4 story model.

Story	The equivalent cross-sectional area of the brace(cm^2)	Plate Thickness(mm)	Cross-section of each strip (As)
1	52	1.6	9.51
2	52	1.6	9.51
3	52	1.6	9.51
4	36	1.1	6.5

Selected seismic records in nonlinear dynamic analysis

In this study, two groups of seismic records, including 1- recommended seismic records of FEMA-P695 in far-field, 2- recommended seismic records of FEMA-P695 in near-field have been used.

Seismic records recommended by FEMA-P695

FEMA-P695 has introduced a set of earthquake records for IDA. This set of seismic records in group 1 includes 22 far-field seismic records (records with a distance of more than 10 km from the fault) with two components horizontal and 28 near-field seismic records (14 near-field records with pulse and 14 near-fields without pulse) with two horizontal components from 1971 to 1999. The models are subjected to incremental nonlinear dynamic analysis under the proposed FEMA-P695 seismic records.

Investigating the behaviour of structural models by IDA

In this study, the ground motion intensity measure is characterized by S_a(T1, 5%), which is the spectral acceleration corresponding to the first-mode elastic vibration period and the 5% damping ratio. θ_{max} is maximum inter-story drift ratio.

Results of IDA for a. far-field records

The results of the IDA of structural models under FEMA-P695 far-field records in figure 6 and their fragility curves in figure 7 are presented.

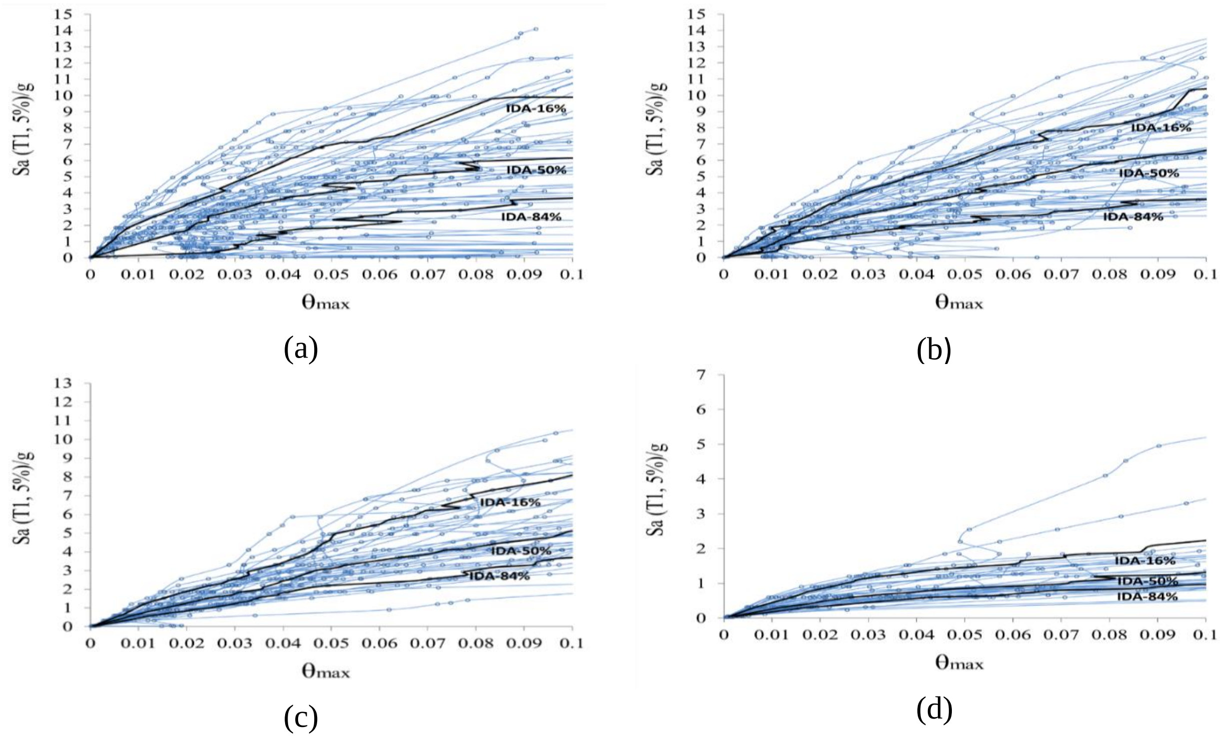


Figure 6 IDA curves of models for FEMA-P695 far-field: a) 4 story b) 8 story c) 12 story d) 24 story.

Results of IDA for b. near-field records with pulse

The results of the IDA of structural models under FEMA-P695 near-field with pulse records in figure 8

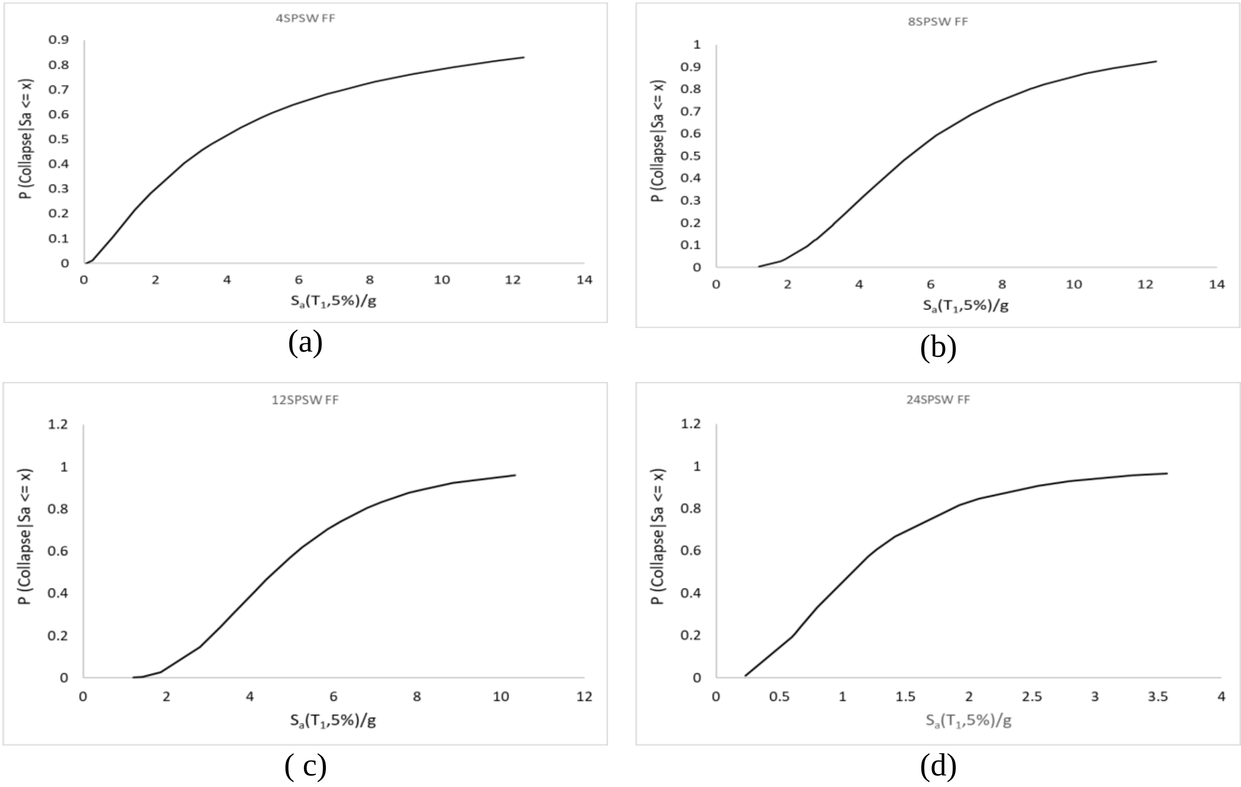


Figure 7 Fragility curves of models for FEMA-P695 far-field records: a) 4 story model (b) 8 story model (c) 12 story model (d) 24 story model.

and their fragility curves in figure 9 are presented.

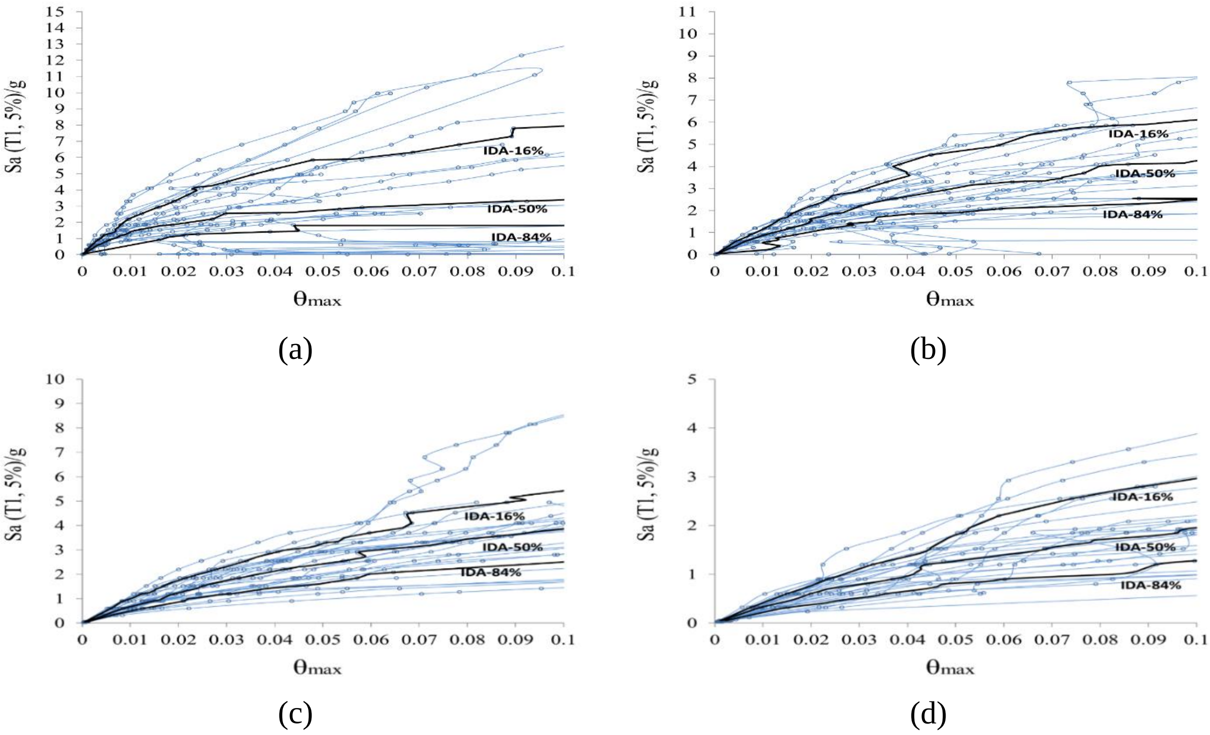


Figure 8 IDA curves of models for FEMA-P695 near-field with pulse: (a) 4 story model (b) 8 story model (c) 12 story model (d) 24 story model.

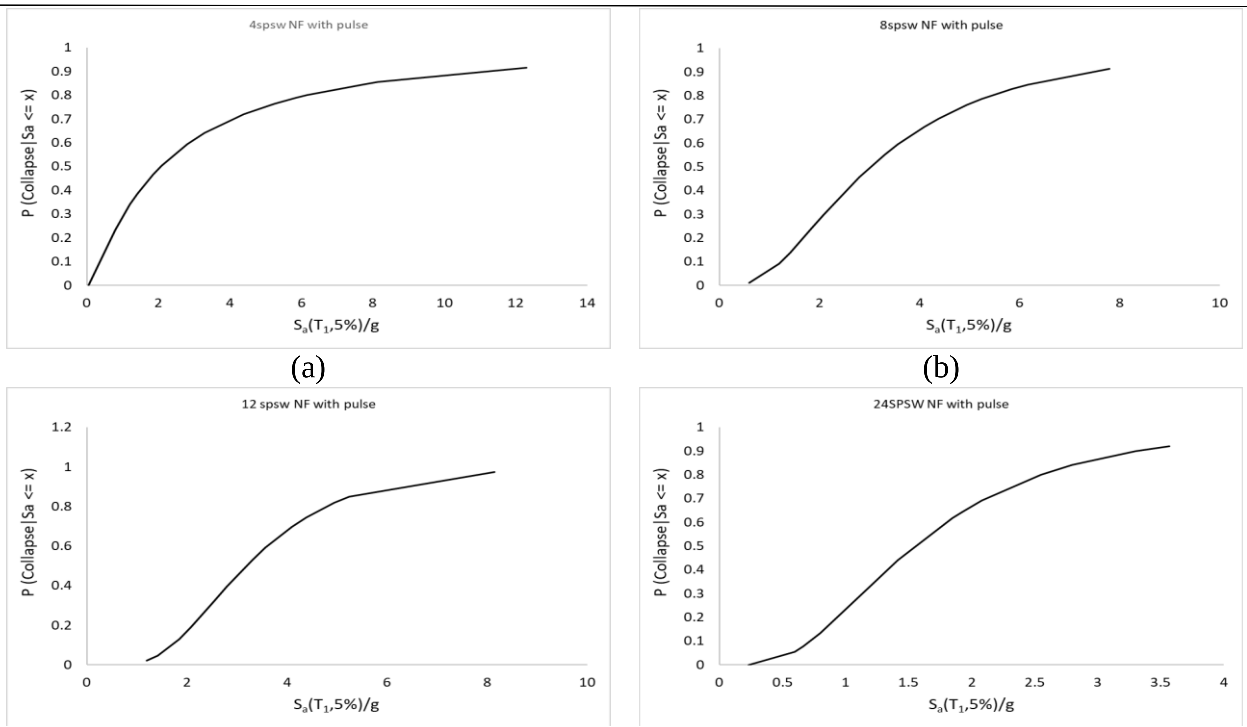


Figure 9 Fragility curves of models for FEMA-P695 near-field with pulse records: (a) 4 story model b) 8 story model (c) 12 story model (d) 24 story model.

Results of IDA for c. near-field records without pulse

Figure 10 shows the fragility curves of the models under near-field without pulse seismic records FEMA-P695.

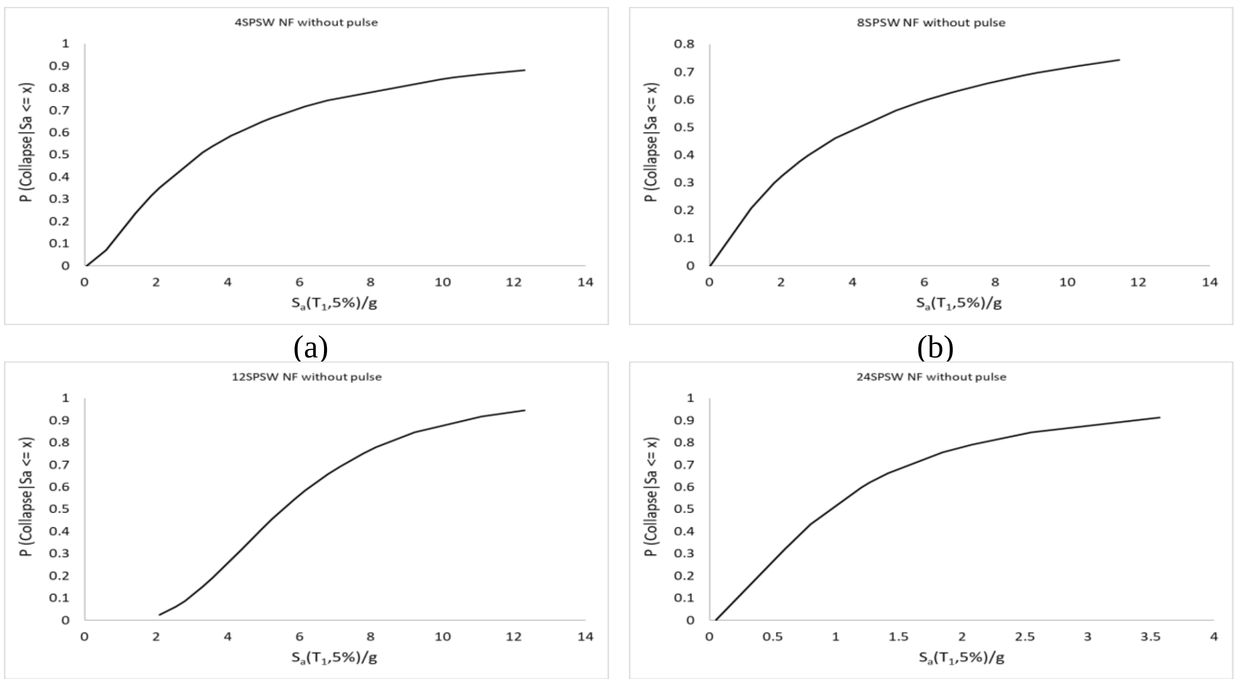


Figure 10 IDA curves of models for FEMA-P695 near-field without pulse records:

(a) 4 story model (b) 8 story model (c) 12 story model (d) 24 story model.

Fragility curves and total uncertainty of the structure

Since earthquakes are random in nature, so this issue should be viewed with a probabilistic approach, and taking into account all the uncertainties, the risk levels in the design of existing buildings should be calculated. One of the advantages of the probabilistic approach is that it can directly investigate a large

number of uncertainties in earthquake engineering. These uncertainties include the demand (magnitude, mechanism for wave propagation, frequency content, duration, duration, aftershocks, and aftershocks, etc.) and capacity (resistance, hardness, inherent damping, cyclic behavior, etc).
The FEMA P695 guideline considers the overall uncertainty of structural collapse to be dependent on the uncertainty of selected records, design, experimental information, and modelling. The total uncertainty of the structure is calculated according to equation (6):

$$\beta_{TOT} = \sqrt{{\beta_{RTR}}^2 + {\beta_{DR}}^2 + {\beta_{TD}}^2 + {\beta_{MDL}}^2} \qquad (6)$$

Where β_{RTR}, β_{DR} ، β_{TD} ، β_{MDL} ، β_{TOT} represent the uncertainties of the selected records, design, experimental information, modelling, and general uncertainty of the structure, respectively. Differences in the response of the structures under different earthquake records can be attributed to differences in the frequency content and different dynamic properties of earthquake records. According to research conducted by Hasleton (2006), Ibarra and Kravinkler (2005), and Zareian et al. (2006), it can be stated that the amount of uncertainty related to selected records (β_{RTR}) for different building systems is about
0.35 to 0.45. Equation (7) relates the amount of uncertainty in association with the set of earthquake records to the ductility coefficient of the structure.

$$\beta_{RTR}=0.1+0.1\mu_T\leq 0.4 \qquad (7)$$

Uncertainty related to the design of the structure depends on the completeness of the design assumptions to avoid unexpected failure modes. The uncertainty of experimental information is the completeness of this information to introduce and determine the technical characteristics of dependent instrument systems. Uncertainty in structural modelling depends on the degree of completeness of the nonlinear model of the structure. The more complete and comprehensive the modelling is and the more it can observe the failure modes and also the better it can be in line with the actual behaviour of the structure, the less the modelling uncertainty decreases. These curves show the probable level of damage for different levels of earthquake magnitude. With the help of these curves, by changing the vulnerability of structures, they can be prioritized for retrofitting. Two important factors play a role in creating fragility curves: 1- Damage to the structure, which is expressed by a probability function; 2. How the earth moves, where the fragility curve can be plotted based on one of the earth movement indicators such as maximum ground acceleration PGA, maximum ground speed PGV, or maximum ground displacement PGD.

Assessing the collapse of the structure

To evaluate the collapse of the structure, first, the ACMR parameter is calculated according to the spectral shape correction factor (SSF) and also the CMR parameter and is compared with similar values in the FEMA-P695 instruction. The S_{CT} indicates the acceleration at which the probability of rupture of the structure is 50% and its value is determined from the fragility curve. The S_{MT} , spectral acceleration in the first mode of the structure for an earthquake with a return period of 2475 years is the same as the Maximum Credible Earthquake (MCE) surface earthquake (see Figure 11). SSF is the spectral shape correction factor. In this study, the collapse of structures has been evaluated according to the method presented in FEMA-P695. According to which, in order to considering a structure as a safe one, the probability of collapse of the structure must be limited to 20%. According to the calculations, the performance of 4, 12, 8 stories structures (Tables 3 to 6) in far-field earthquakes, near-field, both with and without a pulse, is acceptable. However,

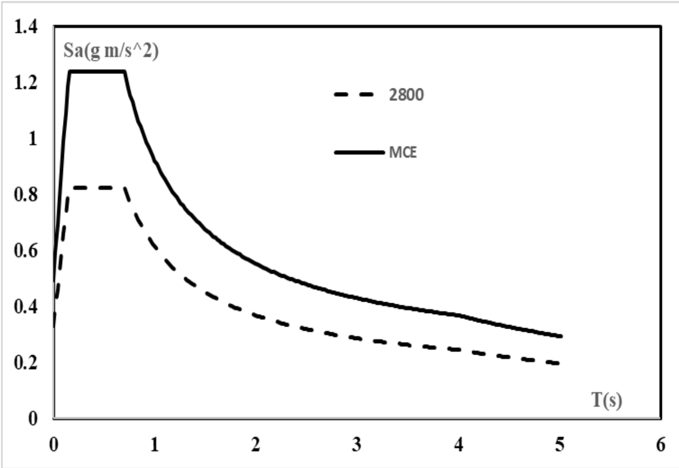
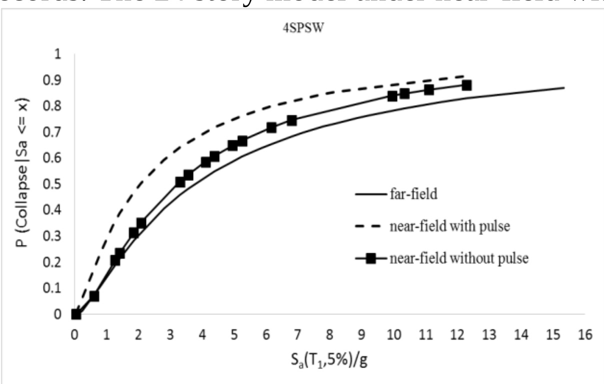
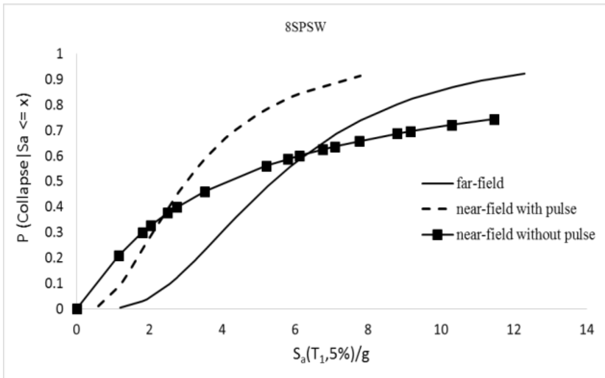


Figure 11 2800 spectrum and maximum possible MCE earthquake

the 24-story structural model, which is considered a high-rise structure, does not have acceptable performance in the near-field without pulse and has an almost acceptable performance in the remote near-field, and this shows that the design principles of tall structures brackets with shear walls should be reconsidered. The fragility curves and the effect of changes in the studied models on the final capacity and fragility are shown in Figure 12. The ACMR parameter or collapse capacity of 4, 8, 12 and 24-story RCF with SPSW have improved under far-field and near-field with pulse records of FEMA-P695 so the increase of the ACMR parameter up to169% and 228% in 12-story model in far-field and near-field without pulse records. The 24 story model under near-field without pulse records of FEMA-P695 has failed (Table 6).



a)



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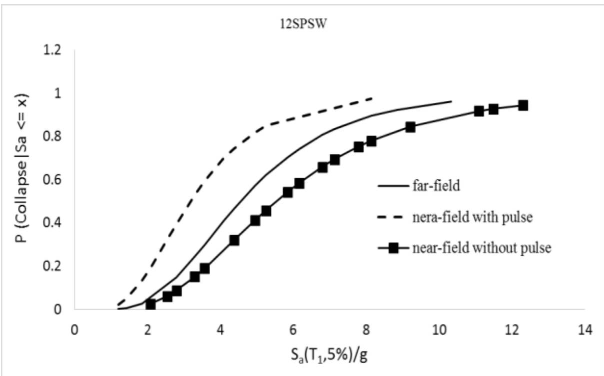

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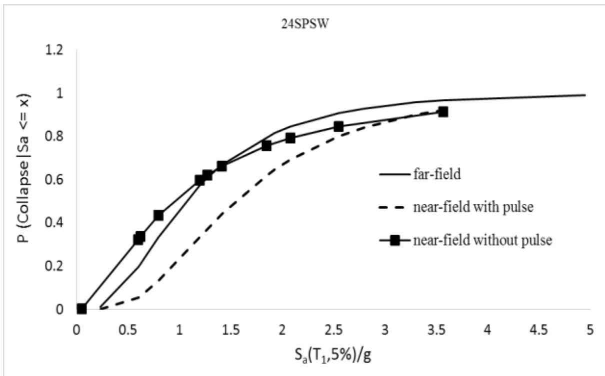

(d)

Figure 12 Fragility curves of models for FEMA-P695 far-field and near-field records:
(a) 4- story model b) 8- story model (c) 12- story model (d) 24- story model.

Table 3 Effective parameters in assessing the safety margin of the modified collapse and evaluating the performance of the 4-story model.

Seismic Records	S_{CT}	S_{MT}	CMR	SSF	ACMR	Accepted ACMR	Percent increase of ACMR	Performance
Far-field	4.4	1.24	3.56	1.0	3.56	1.66	114%	accepted
Near-field without pulse	3.2	1.24	2.58	1.0	2.58	1.66	55%	Accepted
Near-field with pulse	2.06	1.24	1.66	1.0	1.66	1.66	0%	Accepted

Table 4 Effective parameters in assessing the safety margin of the modified collapse and evaluating the performance of the 8-story model.

Records	S_{CT}	S_{MT}	CMR	SSF	ACMR	Accepted ACMR	Percent increase of ACMR	Performance
Far-field	5.40	1.24	4.35	1.0	4.35	1.66	162%	Accepted
Near-field without pulse	4.18	1.24	3.37	1.0	3.37	1.66	103%	Accepted
Near-field with pulse	3.03	1.24	2.44	1.0	2.44	1.66	47%	Accepted

Table 5 Effective parameters in assessing the safety margin of the modified collapse and evaluating the performance of the 12-story model.

Records	S_{CT}	S_{MT}	CMR	SSF	ACMR	Accepted ACMR	Percent increase of ACMR	Performance
Far-field	4.56	1.02	4.47	1.0	4.47	1.66	169%	Accepted
Near-field without pulse	5.55	1.02	5.44	1.0	5.44	1.66	228%	Accepted
Near- field with pulse	3.20	1.02	3.14	1.0	3.14	1.66	89%	Accepted

Table 6 Effective parameters in assessing the safety margin of the modified collapse and evaluating the performance of the 24-story model.

Records	S_{CT}	S_{MT}	CMR	SSF	ACMR	Accept ACMR	Percent increase of ACMR	Performance
Far-field	1.08	0.66	1.64	1.0	1.64	1.66	0%	Accepted
Near-field without pulse	0.96	0.66	1.45	1.0	1.45	1.66	0%	Not accepted
Near-field with pulse	1.56	0.66	2.36	1.0	2.36	1.66	42%	Accepted

CONCLUSION

In this research, buildings with lateral resisting system of RCF equipped with SPSW are modeled and the recommended far-field and near-field seismic records of FEMA-P695 are selected in order to perform IDA. On the FEMA P695 guideline the overall uncertainty of structural collapse was considered that to be dependent on the uncertainty of selected records, design, experimental information, and modelling. The total uncertainty of the structure and effective parameters in assessing the safety margin of the models were calculated according to the equations of FEMA-P695 and the probability of collapse of the structure was limited to 20%.

- By looking at the category of IDA curves, we can get an overview of the behaviour of the structure, from full elastic to complete failure. Comparing the behavior of the structure with different heights, it can be said that with increasing height, the structure enters the nonlinear area sooner and the capacity of the structure decreases.
- This study has been carried out to evaluate the effects of near-field and far-field ground motions using FEMA P695 methodology on the buildings designed with the method presented in FEMA-P695. Effects of different uncertainties on the collapse capacity and fragility curves of structures including record-to record (β_{RTR}) uncertainty, design requirement (β_{DR}) uncertainty, test data (β_{TD}) uncertainty, and modelling (β_{MDL}) uncertainty were investigated thoroughly. Analysis results are strictly valid for the range of parameters considered in this study.
- The results showed that the ACMR parameter or collapse capacity of 4, 8, 12 and 24-story reinforced concrete frames with steel plate shear wall have improved by an average 56%, 104%, 162 and 14% respectively, corresponding to the ACMR parameter proposed by FEMA- P-695. The 24 story model under near-field without pulse records of FEMA-P695 has failed.
- In the discussion of fragility curves and the probability of failure, the performance of RCF equipped with SPSW with a suitable confidence margin seems to be acceptable.

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