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SUGGESTION OF A PROBABILISTIC METHOD FOR DIFFERENT PERFORMANCE LEVELS EVALUATING OF STRUCTURES UNDER PROGRESSIVE COLLAPSE CAUSED BY EARTHQUAKE

PREDLOŽENA METODA VEROVATNOĆE ZA OCENU RAZLIČITIH NIVOA PERFORMANSI KONSTRUKCIJA IZLOŽENIH PROGRESIVNOM KOLAPSU USLED ZEMLJOTRESA

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- progressive collapse
- uncertainty parameters
- yield limit states
- structure capacity

Abstract

After events such as explosion, formidable earthquake, etc., main columns and bearing elements of a building may be destroyed in some parts and if remaining structure does not have adequate resistance for bearing vertical loads, then a progressive collapse initiates and even fully destroys the building in the worst circumstances. In this study, structures are investigated in two states. First state includes 4 floor structures with plans for 4 spans of 6 m, 6 spans of 4 m, and 3 spans of 8 m that are 3D evaluated and analysed. In the second state, each investigated model is evaluated and analysed in 5 states with various compressive strength. Comparison of frame behaviour indicates that structural capacity reduces by increasing structure plan dimensions. By contemplation in produced results, it is demonstrated that four floor frames with 6 spans of 4 m almost have similar capacities. Structural exceedance probability will also enhance from each performance limit state CP, IO, and LS (collapse prevention, immediate occupancy, and life-safety) by increasing the length of span in a determined and constant spectral acceleration.

Ključne reči
- progresivni kolaps
- parametri nesigurnosti
- stanja na granici tečenja
- konstrukciona nosivost

Izvod

Nakon događaja kao što su eksplozija, snažan zemljotres, itd., glavni noseći stubovi i elementi oslanjanja zgrade mogu biti oštećeni na nekim mestima, a ako ostatak zgrade nema zadovoljavajuću otpornost za vertikalna opterećenja, nastaje progresivni kolaps i u potpunosti se razara zgrada, u najgorim okolnostima. U ovom radu, ovakve konstrukcije su proučene u dva slučaja. U prvom slučaju su konstrukcije od 4 sprata sa projektima za 4 grede od 6 m, 6 greda od 4 m i 3 grede od 8 m, koje su obrađene 3D proračunom i analizom. U drugom slučaju, svaki obrađeni model je proračunat i analiziran za 5 stanja promenljive pritisne čvrstoće. Poređenje ponašanja ramova pokazuje da nosivost konstrukcije opada sa porastom dimenzija plana-tlocrta. Razmatranjem dobijenih rezultata, pokazuje su sličnih kapaciteta nosivosti četiri spratna rama sa 6 greda po 4 m. Verovatnoća očekivanja konstrukcije se proširuje sa odgovarajućim graničnim stanjem CP, IO, i LS (prevencija kolapsa, neposredni boravak i životna bezbednost) povećanjem dužine grede u determinisanom i konstantnom spektru ubrzanja.

INTRODUCTION

Kapil Khandelwal in 2008 /1/ by using numerical calculations and finite element method, with utilization of ‘alternate path’ method showed that the use of beams with smaller spans is so useful instead of girders in steel construction with moment frame against progressive collapse. Also, the joint strength in this phenomenon will enhance by increasing the joint shear strength. This researcher has used a multi scale model for these investigations suggesting a new technique called ‘Pushdown’. Kandil et al. in 2013 /2/ with the use of finite element method and nonlinear analysis investigated the behaviour of steel structures. They indicated that structural displacement reduces by increasing the damping and addition of floors. They also expressed that three dimensional analysis and including nonlinear dynamic analysis would achieve more realistic results. Ferraioli et al. in 2014 /3/ conducted research widely in the Pushdown method field in order to investigate the progressive collapse on steel moment frames. By changing the numbers of spans, height, column spacing, and various scenarios of column removal in two-dimensional state, they demonstrated that static analysis factors of codes are so conservative. Yi et al. in 2008 /4/ performed another test on 1/3 concrete scale frame and in the form of column loss and constant vertical loading. Thus, from the first the vertical loading has been constantly proceeded and column loss has been conducted. Here also the results showed that the breaking of the beam’s tension bar was high after bearing a displacement and rotation.

Tsitos in 2009 /5/ tested two samples of two-dimensional steel structures with scale of 1/3 in two states which are particular moment frame and Post Tensioned Energy Dissipating frame (PTED). He applied 2 increasing quasi-static loads to the top of the lost column via an actuator to break joints and showed that final capacity in PTED frame is dependant on tension strength of prestressed cable. These two frames have been designed and made based on the earthquake design criteria. He indicated that this design is suitable for resisting against progressive collapse and matches with the column removal scenario by alternate path method. Tiecheng Wang et al. in 2016 /6/ tested and discussed a 1/3 concrete scale building. They indicated that the tension strength mechanism of the beam would aid the structure after column removal in order to re-perform the force distribution. Loading in this test was quasi-static. They emphasised on the beam role on damage deduction. Tasnimi and Mohebkhah in 2011 /7/ experimentally and numerically studied seismic behaviour of a steel frame and an unequipped masonry infill with and without opening. They demonstrated that including interaction of frame and infilled frames causes an improvement in seismic response of the structure. Infilled frames also increase the ability of energy dissipating of the building.

Investigating the mentioned studies in the field of progressive collapse, it is demonstrated that in most of these works, all evaluated parameters are considered certain and non probabilistic. Therefore, it is essential to conduct research based on probabilistic parameters. In this research, some geometric and strength parameters of structure such as effect of span length of frames on structures with the same plan dimension, and also concrete compressive strength of a structure will be considered uncertain and probabilistic for estimating the progressive collapse in concrete structure, and the effect of each of these parameters will be reported.

SPECIFICATIONS OF STUDIED MODELS

In this study, structures are investigated in two states. First state includes 4 floor structures with plans for 4 spans of 6 m, 6 spans of 4 m, and 3 spans of 8 m that are designed in 3D and in a constant basic state on soil type II, and then a 3D structure of them is evaluated and analysed. In the second state, each investigated model is evaluated and analysed in 5 states with various compressive strengths. Comparison of frames behaviour indicates that structure capacity reduces by increasing the structure plan dimensions.

Regarding the considered height for floors, the four floor structures have 16 m height of foundation level. As it is also shown in Fig. 1, the considered plan in this study is in square shape for 3D structures that the length of each edge of it is 24 m. Frame spacing from each other is assumed 4, 6, and 8 m. Also it should be mentioned that all beam-column joints are fixed in moment frame direction. According to the code ASCE /8/ Standard 7-05, Tehran is assumed for structures positioned such that, considering that Tehran is placed in a relatively high risk zone, the basic accelerate of the plan is $A = 0.3g$ and with attention to the residential application of the building, the building's importance coefficient is considered I = 1. Structures behaviour factor is assumed $R = 7$. Figure 2 shows the view of 3D structures of various span lengths. All investigated structural modelling is conducted by OpenSees software, /9/.

Table 1. Samples of applied sections in regular structures with four floors.

Floor	Column	Beam
1	45*45 cm^2, 16T16	45*45 cm^2
2	40*40 cm^2, 16T16	40*40 cm^2
3	40*40 cm^2, 12T16	40*40 cm^2
4	35*35 cm^2, 10T16	35*35 cm^2

In Table 1, applied sections in two dimensional frames of models are presented.

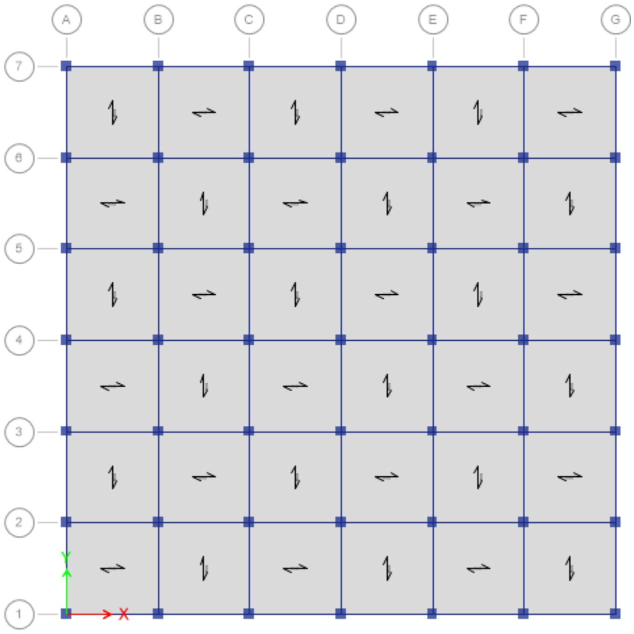
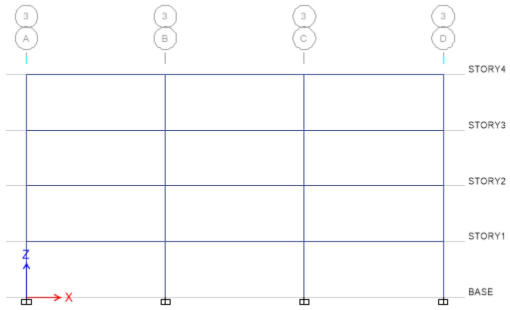
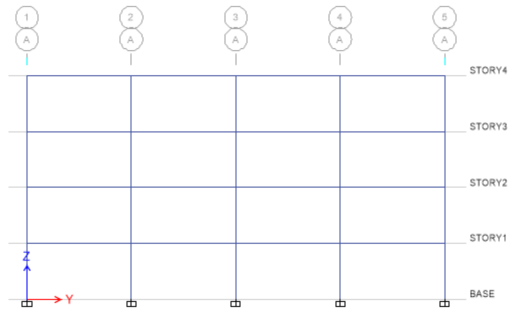


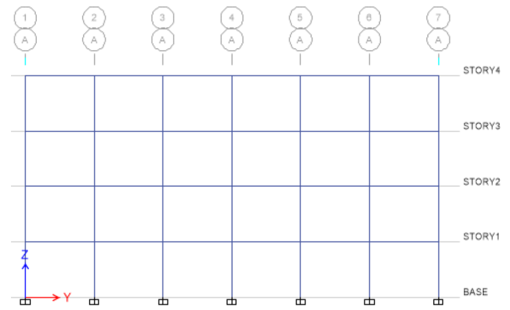
Figure 1. The plan of 3D structures (6 spans of 4 m).



(a) 3 spans of 8 m



(b) 4 spans of 6 m



(c) 6 spans of 4 m

Figure 2. View of 3D structures with various span lengths.

THE USED ACCELEROGRAMS

In order to use incremental dynamic analysis for each of the studied models, totally 10 earthquake records are chosen among earthquakes of the zone near to a fault. Details of the used earthquake records in this study are presented in Table 2.

Table 2. Details of used records in incremental dynamic analysis.

Record No.	Record	Recorder Station	Soil Type	Date of occurrence	PGA(g)
1	Chi-Chi, Taiwan	CHY080	II	9/20/1999	0.902
2	Coyote Lake	Gilroy Array 3	II	8/6/1979	0.434
3	Kobe	KJMA	II	1/16/1995	0.821
4	Landers	Coolwater	II	6/28/1992	0.417
5	Loma Prieta	Corralitos	II	10/18/1989	0.644
6	Morgan Hill	Anderson Dam	II	4/24/1984	0.423
7	N. Palm Springs	N. Palm Springs	II	7/8/1986	0.694
8	Northridge	Santa Monica	II	1/17/1994	0.883
9	Bam	Bam	II	26/12/2003	0.767
10	Tabas	9101 Tabas	II	9/16/1978	0.917

INCREMENTAL DYNAMIC ANALYSIS (IDA)

For conducting IDA analysis, at first Intensity Measure (IM) parameter (e.g. maximum ground acceleration, PGA, or spectral acceleration corresponding to the first mode of the structure, $S_a(T_1)$) scales from a very small value for elastic behaviour in structural model under dynamic analysis to a determined level of intensity measure for achieving to the intended collapse limit with an appropriate algorithm, and every time this scale factor is applied to a earthquake record and structure affected by that record undergoes time history dynamic analysis, /10-13/.

Category of IDA curves

Derived IDA curves of incremental dynamic analysis of studied frames under 10 mentioned records in Table 2 are presented in Figs. 3-11. Structural behaviour in these frames are depicted from elastic limit to collapse threshold and total instability in the diagrams. Then, nonlinear behaviour in the curves is determined by passing of this area, and nonlinear behaviour of the structure is displayed by increase or decrease of stiffness. Comparison of frames with different heights shows that by increasing the span length, structures have less capacity and enter the nonlinear area faster.

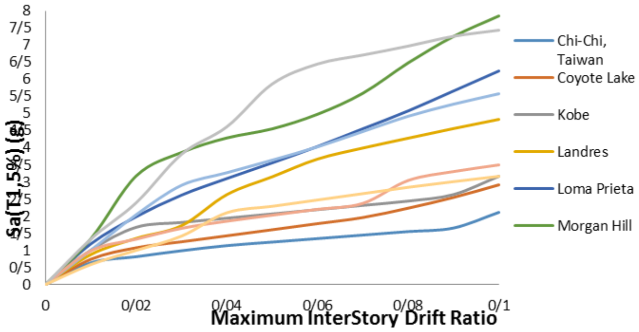


Figure 3. Category of IDA curves for four floor frame with 6 spans of 4 m and compressive strength of 18.58 MPa.

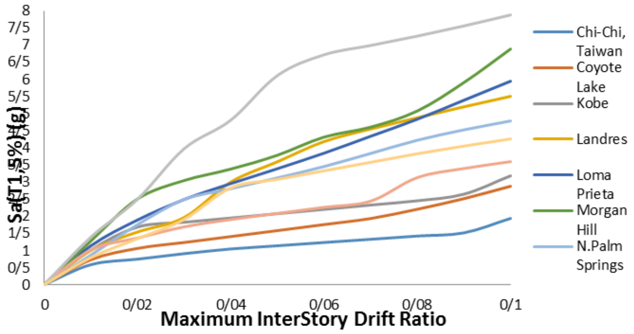


Figure 4. Category of IDA curves for four floor frame with 6 spans of 4 m and compressive strength of 21 MPa.

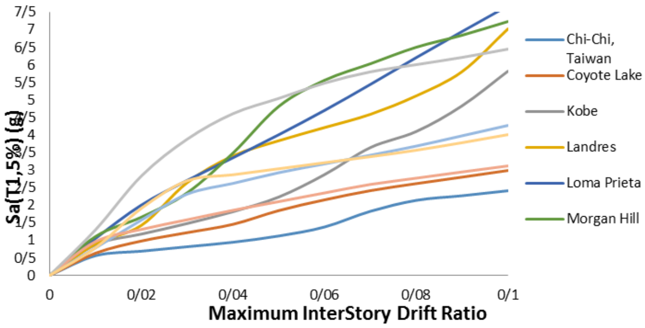


Figure 5. Category of IDA curves for four floor frame with 6 spans of 4 m and compressive strength of 23.42 MPa.

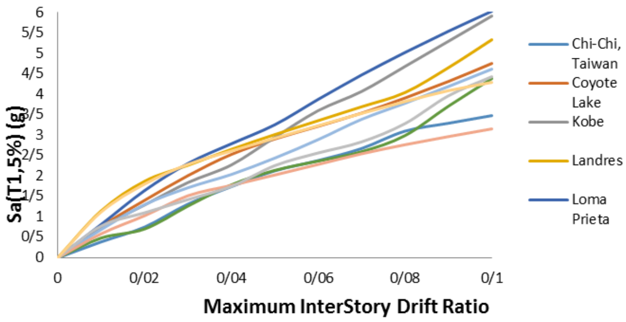


Figure 6. Category of IDA curves for four floor frame with 4 spans of 6 m and compressive strength of 18.58 MPa.

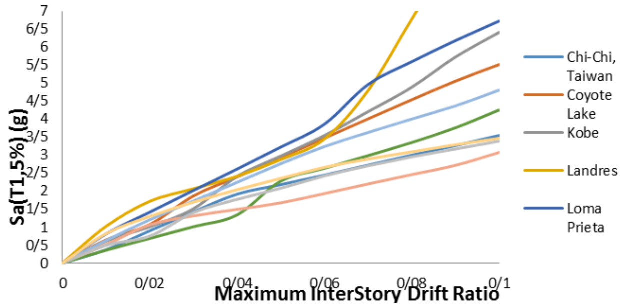


Figure 7. Category of IDA curves for four floor frame with 4 spans of 6 m and compressive strength of 21 MPa.

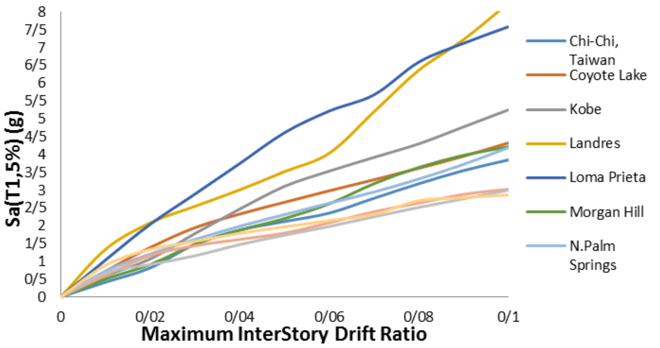


Figure 8. Category of IDA curves for four floor frame with 4 spans of 6 m and compressive strength of 23.42 MPa.

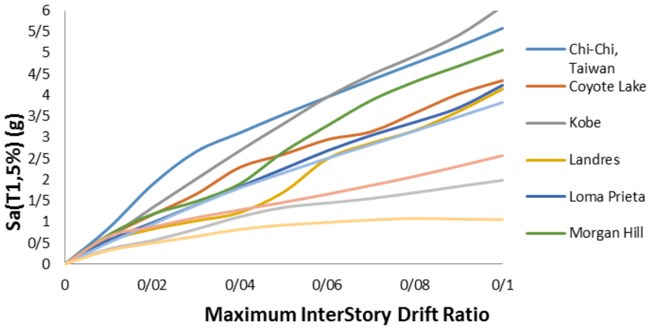


Figure 9. Category of IDA curves for four floor frame with 3 spans of 8 m and compressive strength of 18.58 MPa.

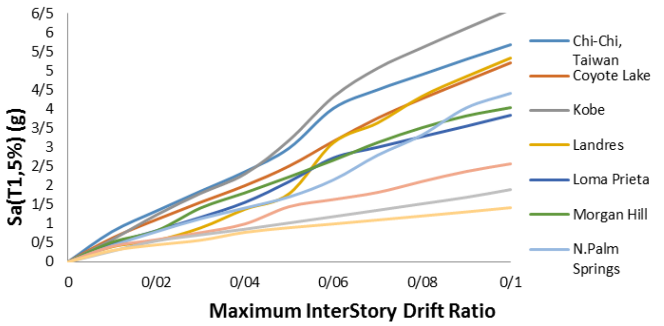


Figure 10. Category of IDA curves for four floor frame with 3 spans of 8 m and compressive strength of 21 MPa.

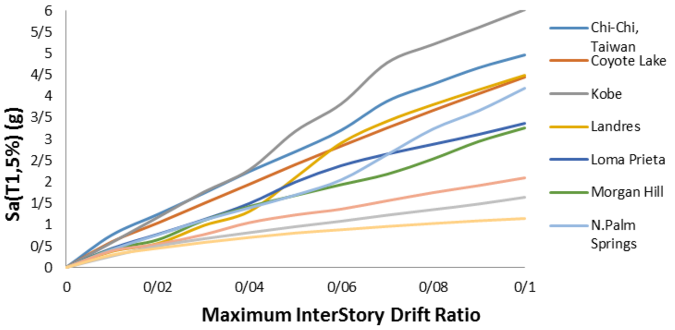


Figure 11. Category of IDA curves for four floor frame with 3 spans of 8 m and compressive strength of 23.42 MPa.

A category of IDA curves is obtained after conducting an IDA analysis under several various earthquake records, that considering the large quantity of curves in a curve's category which each of them represents a special behaviour of the structure under earthquake records and does not express total performance of the structure about types of earthquakes. The mean values of every 3 categories of IDA curves including four floor frame with 6 spans of 4 m, 4 spans of 6 m, and 3 spans of 8 m are also shown together in Figs. 12 to 14 for comparison.

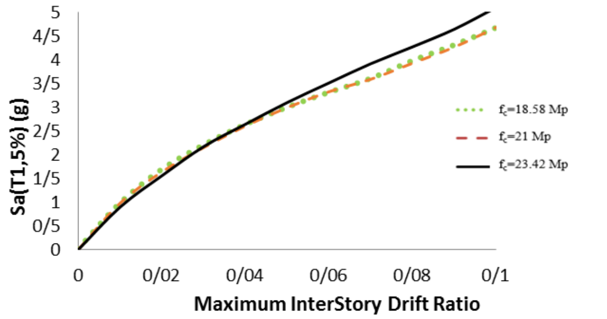


Figure 12. Mean values of IDA curves of four floor frame with various compressive strengths for 6 spans of 4 m.

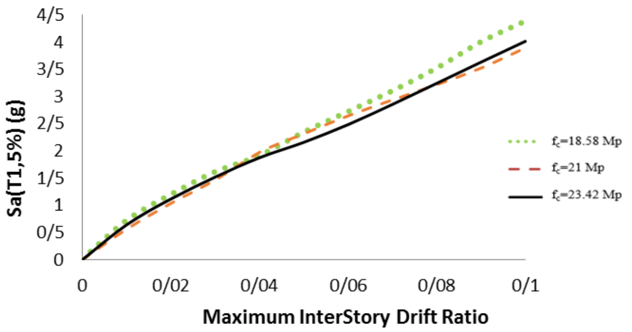


Figure 13. Mean values of IDA curves of four floor frame with various compressive strengths for 4 spans of 6 m.

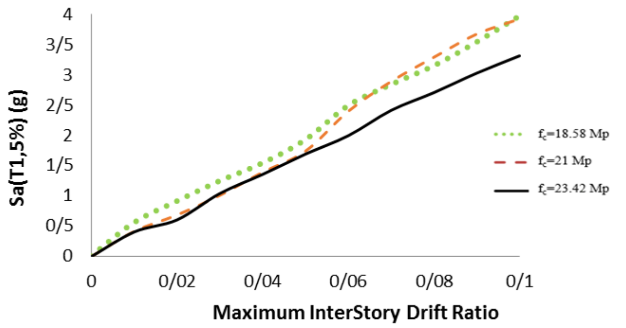


Figure 14. Mean values of IDA curves of four floor frame with various compressive strengths for 3 spans of 8 m.

Also the mean values of every 3 categories of IDA curves including four floor frame related to states of 6 spans of 4 m, 4 spans of 6 m, and 3 spans of 8 m are shown together in Figs. 15 to 17 for comparison.

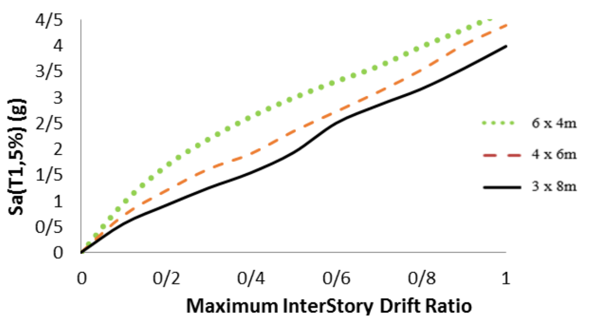


Figure 15. Mean values of IDA curves of four floor frame for spans with various dimension and comp. strength of 18.58 MPa.

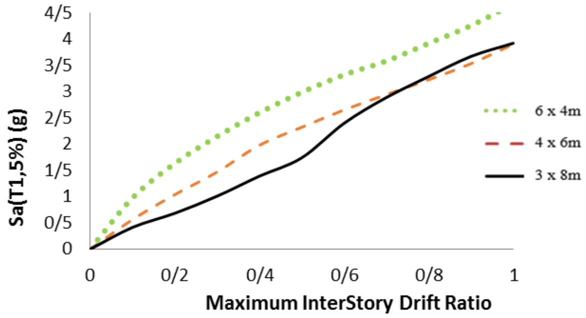


Figure 16. Mean values of IDA curves of four floor frame for spans with various dimension and comp. strength of 21 MPa.

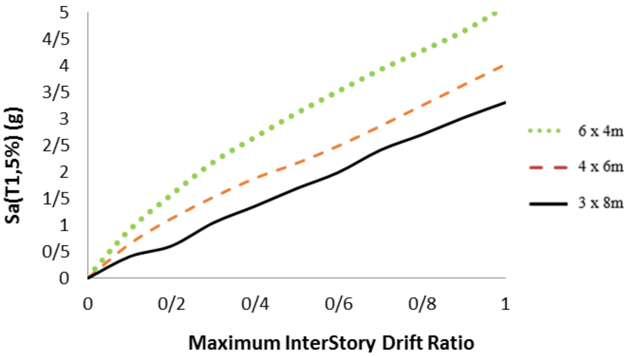


Figure 17. Mean values of IDA curves of four floor frame for spans with various dimension and comp. strength of 23.42 MPa.

As shown in Figs. 15 to 17, IM values decrease in curves for a constant amount of DM by increasing the length of the building's span. Regarding the curves in above figures, it can be said that capacity (*Sa*) of structures reduces with their height enhancement.

COLLAPSE ANALYSIS AND FRAGILITY CURVES

In order to reach the occurrence probability of limit states from outcomes of IDA analysis, some curves are used termed 'Fragility Curve'. The fragility curves express the relation between structural damages and selective intensity scale of earthquake. These curves generally are modelled with normal log distribution. The fragility curves are usually modelled by normal log cumulative function and indicate the occurrence or exceedance probability from a collapse state in a determined value of selective intensity scale of earthquake.

In this research, the fragility curves are drawn based on spectral acceleration in the period of the structures, and are modelled by normal log two-parameter cumulative function. The occurrence probability of a determined collapse state (*Dsi*) in determined spectral acceleration is gained from

$$P(DS \geq Dsi \mid S_a(T1)) = \Phi\left(\frac{\ln(X)-\lambda}{\beta}\right), \quad (1)$$

where: Φ(.) is the standard normal cumulative distribution; X is spectral acceleration that has normal log distribution; and λ and β are average and standard deviation of ln(X), respectively.

The spectral acceleration values in all three performance levels IO, LS, and CP for studied frames are presented in Tables 3 to 5. In Table 6, also the parameters for normal log distribution are presented for these levels of studied frames.

Table 3. Spectral acceleration values in terms of *g* in main period of structure for four floor frame with plan of 6 spans of 4 m and various compressive strength in three performance levels.

Record	f_c = 18.58 MPa			f_c = 21 MPa			f_c = 23.42 MPa		
	IO	LS	CP	IO	LS	CP	IO	LS	CP
1	0.642	0.82	1.22	0.586	0.747	1.045	0.56	0.69	0.94
2	0.751	1.101	1.431	0.728	1.058	1.403	0.64	0.98	1.46
3	1.01	1.701	1.94	1.009	1.680	1.946	0.88	1.2	1.8
4	0.891	1.402	2.64	1.001	1.537	2.994	0.9	1.5	3.4
5	1.201	1.99	3.1	1.138	1.895	2.960	1.1	2	3.4
6	1.402	3.21	4.3	1.276	2.503	3.377	1.11	1.7	3.5
7	1.0	2.1	3.3	0.863	1.752	2.803	0.784	1.6	2.6
8	0.991	1.32	1.86	1.019	1.369	1.914	0.97	1.31	1.86
9	1.41	2.43	4.6	1.387	2.491	4.803	1.32	2.84	4.61
10	0.61	1.03	2.11	0.791	1.345	2.829	0.79	1.91	2.87

Table 4. Spectral acceleration (*g*) in fundamental period of structure for four floor frame with plan of 4 spans of 6 m and various compressive strength in three performance levels.

Record	f_c = 18.58 MPa			f_c = 21 MPa			f_c = 23.42 MPa		
	IO	LS	CP	IO	LS	CP	IO	LS	CP
1	0.4	0.76	1.7	0.377	0.9	1.92	0.4	0.8	1.88
2	0.77	1.4	2.5	0.65	1.1	2.41	0.73	1.4	2.33
3	0.75	1.3	2.3	0.6	1.1	2.42	0.59	1.07	2.5
4	1.15	1.89	2.65	11	1.72	2.43	1.34	2.1	3
5	0.82	1.68	2.8	0.82	1.42	2.62	1.04	2.05	3.75
6	0.49	0.71	1.79	0.37	0.69	1.3	0.5	0.92	1.89
7	0.7	1.3	2	0.6	1.21	2.2	0.73	1.2	1.9
8	0.59	1.1	1.77	0.52	1.1	1.5	0.6	1.2	1.6
9	0.78	1	1.75	0.5	0.76	1.78	0.68	0.9	1.5
10	1.2	1.8	2.61	0.82	1.3	2.1	0.9	1.34	1.79

Table 5. Spectral acceleration (g) in fundamental period of structure for four floor frame with plan of 3 spans of 8 m and compressive strength of 18.58 MPa in three performance levels.

Record	f_c = 18.58 MPa			f_c = 21 MPa			f_c = 23.42 MPa		
	IO	LS	CP	IO	LS	CP	IO	LS	CP
1	0.86	1.89	3.1	0.797	1.3	2.36	0.759	1.239	2.241
2	0.66	1.2	2.3	0.63	1.1	2	0.604	1.041	1.949
3	0.6	1.3	2.69	0.59	1.2	2.3	0.585	1.172	2.284
4	0.55	0.83	1.2	0.398	0.57	1.38	0.376	0.562	1.315
5	0.57	0.99	1.8	0.48	0.8	1.56	0.447	0.775	1.496
6	0.7	1.18	1.9	0.52	0.8	1.81	0.425	0.650	1.428
7	0.5	0.95	1.8	0.43	0.793	1.4	0.422	0.766	1.399
8	0.66	0.88	1.3	0.41	0.59	0.995	0.390	0.548	1.058
9	0.3	0.56	1.13	0.29	0.5	0.86	0.267	0.504	0.815
10	0.3	0.5	0.8	0.3	0.46	0.78	0.293	0.450	0.703

Table 6. Normal log distribution parameters (λ, β) for all three performance levels of studied structures.

Span size	Compressive strength (MPa)	Performance Level					
		IO		LS		CP	
6×4	18.58	-0.064	0.288	0.440	0.425	0.878	0.458
	21	-0.050	0.263	0.435	0.370	0.874	0.449
	23.42	-0.139	0.255	0.373	0.398	0.875	0.484
4×6	18.58	-0.333	0.335	0.206	0.338	0.768	0.195
	21	-0.515	0.338	0.071	0.281	0.704	0.224
	23.42	-0.342	0.346	0.209	0.321	0.757	0.287
8×3	18.58	-0.587	0.301	-0.043	0.394	0.517	0.417
	21	-0.779	0.321	-0.256	0.371	0.367	0.397
	23.42	-0.831	0.322	-0.320	0.361	0.317	0.397

In Figs. 18 to 26, the fragility curves in limit states of continuous usability, life safety, and collapse threshold are compared for frames with 6 spans of 4 m, 4 spans of 6 m, and 3 spans of 8 m.

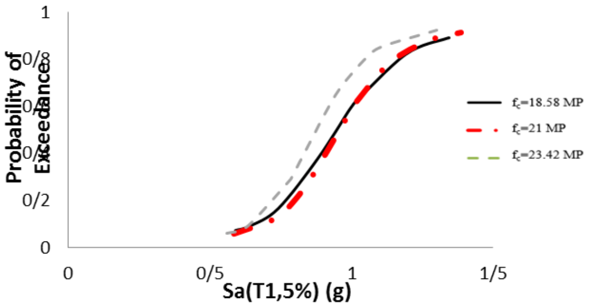


Figure 18. Fragility curves for four floor frame with 6 spans of 4 m in IO performance level.

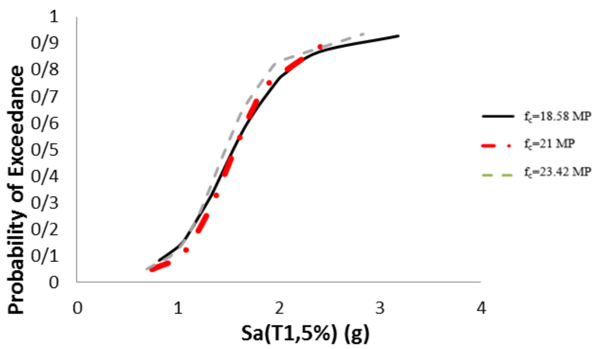


Figure 19. Fragility curves for four floor frame with 6 spans of 4 m in LS performance level.

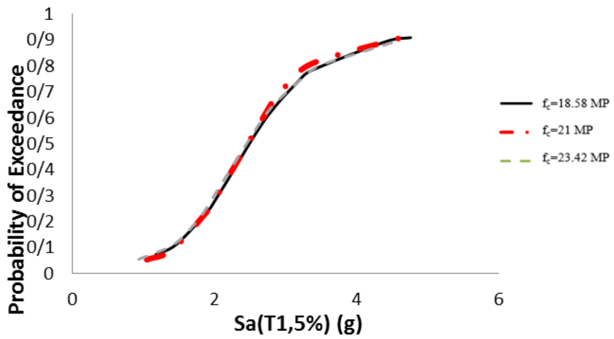


Figure 20. Fragility curves for four floor frame with 6 spans of 4 m in CP performance level.

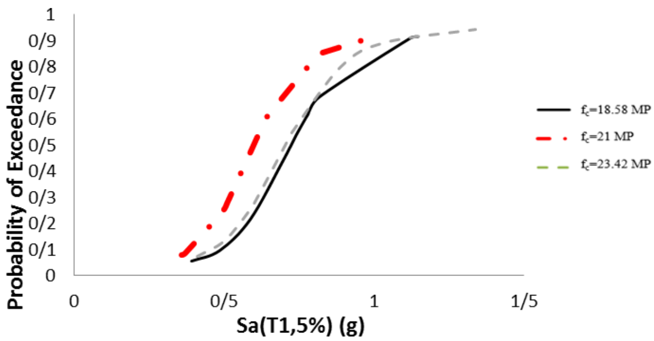


Figure 21. Fragility curves for four floor frame with 4 spans of 6 m in IO performance level.

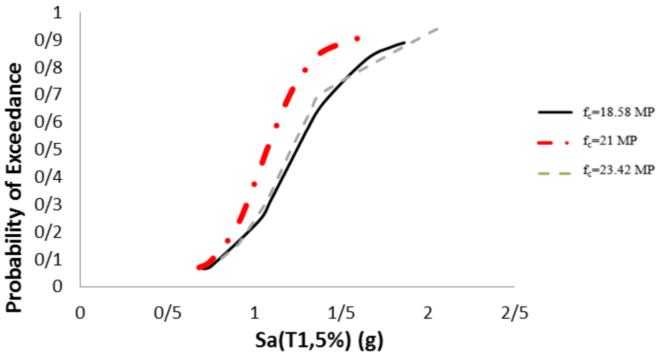


Figure 22. Fragility curves for four floor frame with 4 spans of 6 m in LS performance level.

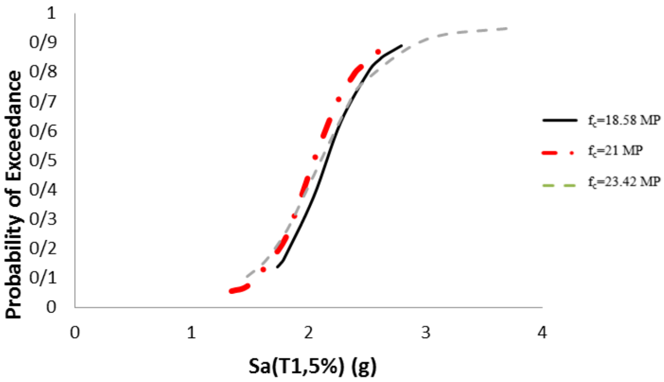


Figure 23. Fragility curves for four floor frame with 4 spans of 6 m in CP performance level.

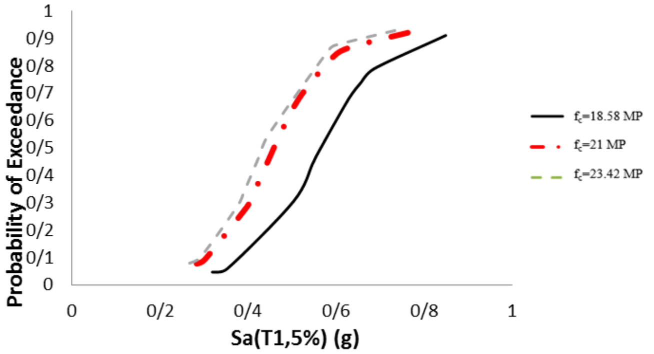


Figure 24. Fragility curves for four floor frame with 3 spans of 8 m in IO performance level.

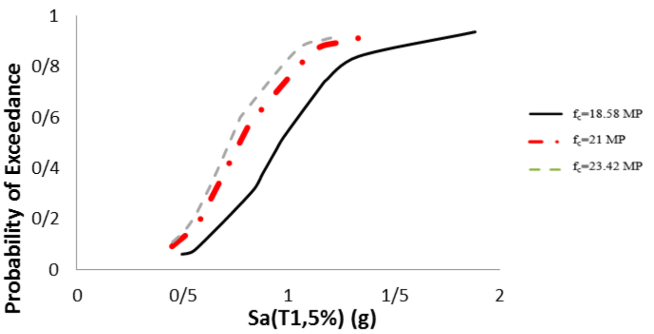


Figure 25. Fragility curves for four floor frame with 3 spans of 8 m in LS performance level.

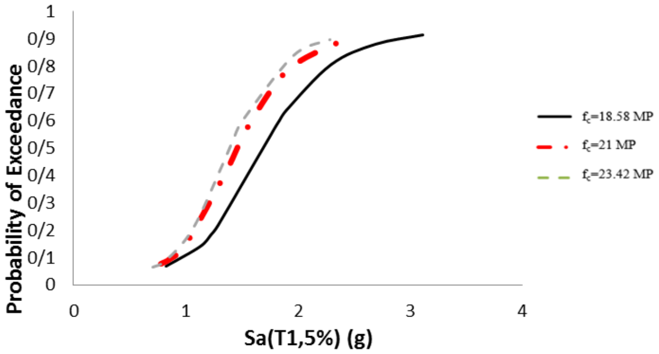


Figure 26. Fragility curves for four floor frame with 3 spans of 8 m in CP performance level.

Comparison of diagrams (Figs. 18-26) shows that four floor frames with 6 spans of 4 m in IO, LS, and CP performance levels have higher failure probability in f_c = 23.42 MPa than other two states. The difference in values of failure probability reduce in life safety performance level, and are approximately matched to each other in fragility curves on CP level. About four floors frames with 3 spans of 8 m, the failure probability in frames with compressive strength f_c = 23.42 MPa is higher than compressive strength f_c = 21 MPa and f_c = 18.58 MPa in all performance levels. In this state, frames with compressive strength f_c = 21 MPa and f_c = 23.42 MPa have close fragility curves to each other in all performance levels.

Fragility curves in IO, LS, and CP limit states are compared in Figs. 27-35 for frames with 6 spans of 4 m, 4 spans of 6 m, and 3 spans of 8 m, related to various compressive strengths.

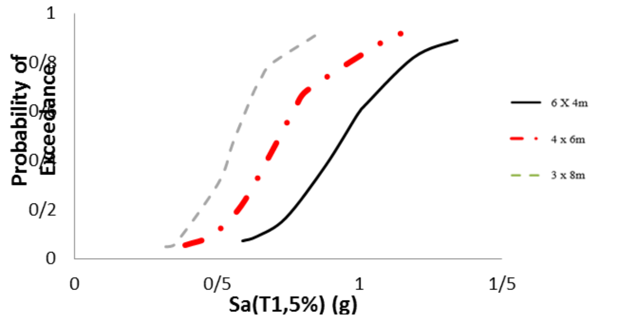


Figure 27. Fragility curves for frames with 4 floor structure with compressive strength of 18.58 MPa in IO performance level.

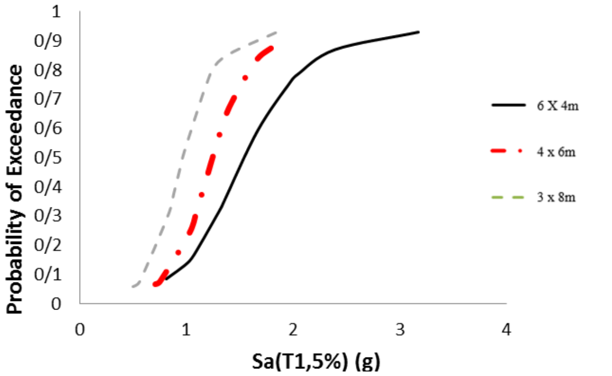


Figure 28. Fragility curves for frames with 4 floor structure with compressive strength of 18.58 MPa in LS performance level.

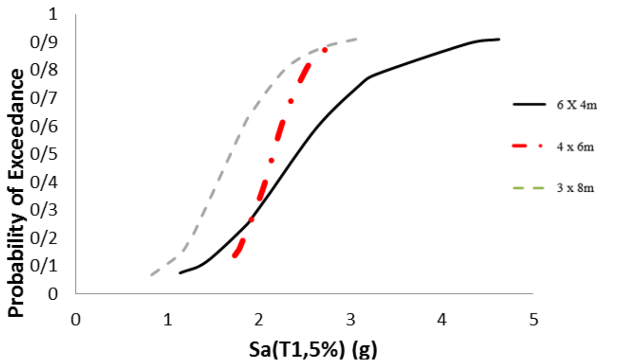


Figure 29. Fragility curves for frames with 4 floor structure with compressive strength of 18.58 MPa in CP performance level.

Considering Figs. 27-35 in three (IO, LS, and CP) performance levels, the failure probability increases by span dimension enhancement in different states with different compressive strength. The S_a values corresponding to 16%, 50%, and 84% failures are presented in Tables 7, 8, and 9, respectively, for three performance levels, uninterrupted operability (IO), life safety (LS), and collapse threshold (CP). *Sa* values corresponding to 50% collapse probability that is named *Sa* structure capacity, are presented in Table 10.

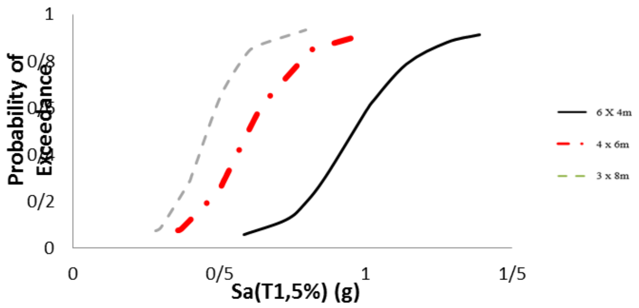


Figure 30. Fragility curves for frames with 4 floor structure with compressive strength of 21 MPa in IO performance level.

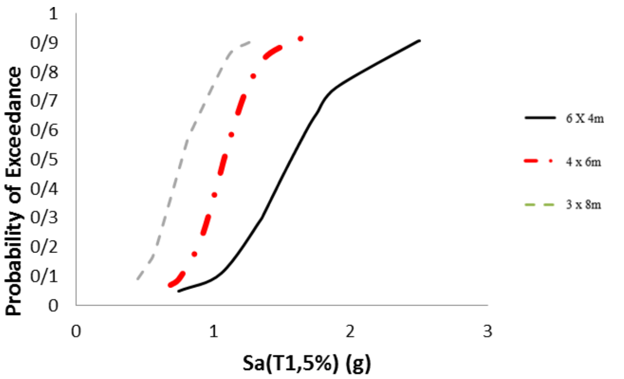


Figure 31. Fragility curves for frames with 4 floor structure with compressive strength of 21 MPa in LS performance level.

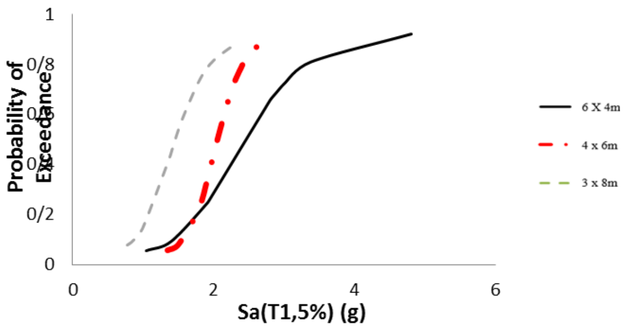


Figure 32. Fragility curves for frames with 4 floor structure with compressive strength of 21 MPa in CP performance level.

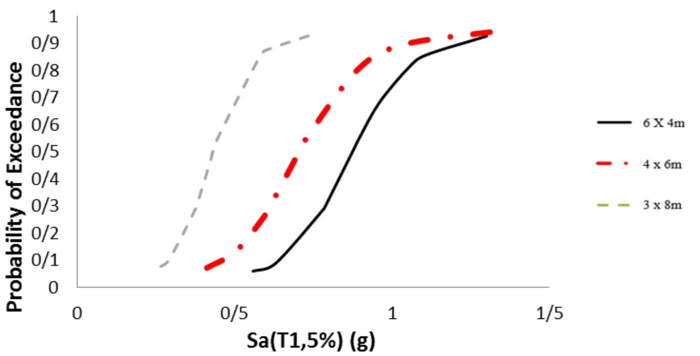


Figure 33. Fragility curves for frames with 4 floor structure with compressive strength of 23.42 MPa in IO performance level.

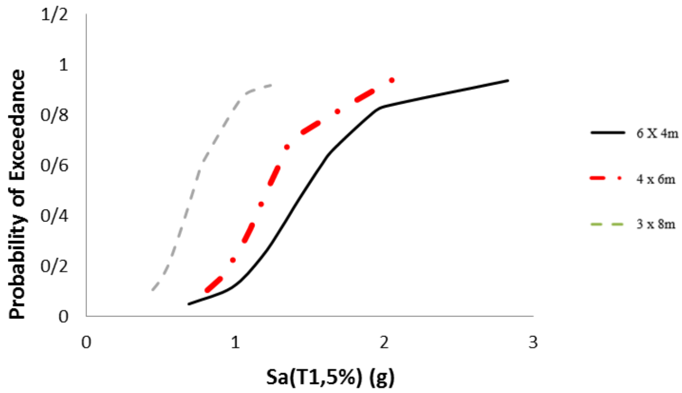


Figure 34. Fragility curves for frames with 4 floor structure with compressive strength of 23.42 MPa in LS performance level.

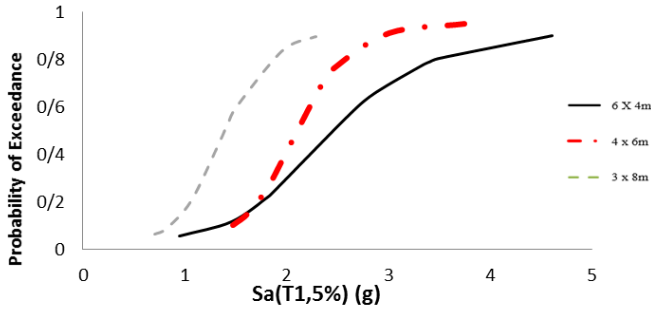


Figure 35. Fragility curves for frames with 4 floor structure with compressive strength of 23.42 MPa in CP performance level.

Table 7. S_a corresponding to various probability of occurrence in IO performance level.

Span size	Comp. strength f_c (MPa)	$(S_a(T_1,5\%))$ IO (g)		
		16%	50%	84%
6×4	18.58	0.735	0.947	1.234
	21	0.698	0.958	1.215
	23.42	0.685	0.875	1.089
4×6	18.58	0.539	0.723	1.025
	21	0.430	0.600	0.808
	23.42	0.522	0.705	0.951
8×3	18.58	0.415	0.567	0.754
	21	0.335	0.459	0.604
	23.42	0.319	0.434	0.567

Table 8. S_a corresponding to various probability of occurrence in LS performance level.

Span size	Comp. strength f_c (MPa)	$(S_a(T_1,5\%))$ LS (g)		
		16%	50%	84%
6×4	18.58	1.062	1.553	2.279
	21	1.136	1.566	2.241
	23.42	1.049	1.465	2.051
4×6	18.58	0.891	1.245	1.685
	21	0.832	1.076	1.375
	23.42	0.918	1.216	1.773
8×3	18.58	0.657	0.964	1.333
	21	0.549	0.769	1.094
	23.42	0.514	0.721	1.017

As shown in charts and values presented in Tables 7-10, collapse probability and/or not meeting the IO, LS, CP performance level in a constant level of seismic intensity increases by increasing the span dimension in structures. Values in Tables 5-10 can be used to determine earthquakes of plan with a specified collapse probability, and/or to assess the adequacy of codes for structure design vs. earthquake loads.

Table 9. S_a corresponding to various probability of occurrence in CP performance level.

Span size	Comp. strength f_c (MPa)	($S_a(T_1$,5%)) CP (g)		
		16%	50%	84%
6×4	18.58	1.577	2.471	3.777
	21	1.628	2.463	3.766
	23.42	1.610	2.460	3.901
4×6	18.58	1.783	2.157	2.598
	21	1.661	2.049	2.519
	23.42	1.636	2.107	2.742
8×3	18.58	1.169	1.687	2.474
	21	1.016	1.461	2.122
	23.42	0.965	1.387	1.967

Table 10. S_a capacity of studied structures.

Comp. strength f_c (MPa)	Span size (m)		
	6 × 4	4 × 6	3 × 8
18.58	2.471	2.157	1.687
21	2.463	2.049	1.461
23.42	2.460	2.107	1.387

Regarding Table 10, the deduction amount of *Sa* capacity for four floor structures with 6 spans of 4 m in the states of f_c =21 and 23.42 MPa compared to structure with compressive strength f_c = 18.58 MPa are 0.3 and 0.4%, respectively, the deduction amount of S_a capacity for four floor structures with 4 spans of 6 m in states of f_c = 21 and 23.42 MPa compared to structure with f_c = 18.58 MPa are 2.3 and 5%, respectively, and the deduction amount of S_a capacity for 4 floor structures with 3 spans of 8 m in states of f_c = 21 and 23.42 MPa compared to structure with f_c = 18.58 MPa are 13.4 and 17.8%, in respect. Moreover, these results express that the deduction of S_a capacity of structure increases by using f_c = 23.42 MPa compared to other states.

CONCLUSION

Considering the conducted study on concrete frames with various compressive strength in research, some achievements of this study are as the following.

Concrete frames with compressive strength f_c = 21 MPa compared to frames with f_c = 23.42 MPa have higher *Sa* capacity. Also, the performed research and experiments express that they have a deformation recovery capacity almost similar to the frames with f_c = 23.42 MPa.

Comparison of frame behaviour with different height indicates that *Sa* capacity of structures decreases by enhancing plan dimensions, in the other words, structures with bigger spans reach to a same level of seismic demand under effect of less amount of spectral acceleration, that it can somehow arise from reduction of structure formability by increasing the length of span.

Comparison of frame behaviour with different compressive strength shows that S_a capacity of frames are almost the same in 4 floor frames with 6 spans of 4 m, but the S_a capacity of frames with f_c = 18.58 MPa is higher in frames with 4 spans of 6 m and 3 spans of 8 m.

Comparison among fragility curves demonstrates that per a certain and constant spectral acceleration, exceedance probability of structure enhances by increasing the span length of each of the limit states (IO, LS, CP).

DATA AVAILABILITY

Some or all data, models, or code generated or used in the study are available in a repository online in accordance with funder data retention policies.

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