PalArch's Journal of Archaeology of Egypt / Egyptology

STRENGTHENING NEEDED TO IMPROVE THE SEISMIC PERFORMANCE OF COLLAPSE AT THE SITES OF TEHRAN

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Alireza Kian Mehr: STRENGTHENING NEEDED TO IMPROVE THE SEISMIC PERFORMANCE OF COLLAPSE AT THE SITES OF TEHRAN -- Palarch's Journal Of Archaeology Of Egypt/Egyptology 18(10), 2027-2043. ISSN 1567-214x

Keywords: Performance levels, collapse, strength, site, nonlinear analysis

ABSTRACT

One of the performance levels of residential buildings is to avoid the global collapse of the building against maximum ground excitations. However, some common regulations for designing such structures, especially those that design such structures with force approaches instead of performance ones, do not require structural control at this level. Due to such issues, the need to increase the rigidity of the structure to control the performance levels mentioned in the structure is considered necessary. In this study, using the analytical approach, through incremental dynamic analysis, the nonlinear behavior of 4 moment frames in both intermediate and special ductility is estimated, then using fragility curves corresponding to the levels of collapse performance, fragility curves are plotted. Then, using the empirical approach and considering the difference in hardness of the four frames according to their pushover curve, the median and deviation changes from the criterion of nonlinear analysis response criteria, for each difficulty, are plotted. In the next step, for 6 different sites in terms of geotechnical and seismicity, probabilistic evaluation was performed to identify sites that have a higher probability of collapse than the acceptable level for these frames. For collapsible frames, the stiffness changes required to reduce the probability of structural collapse to an acceptable extent have been calculated according to the obtained curves. The results show that in the northern and southern constructions of Tehran, short rise intermaediate moment frames to provide protection against the possibility of collapse against maximum ground excitations should provide an increase in earthquake hardness of about 35% through permitted methods..

INTRODUCTION

According to the new loading regulations of different countries such as US ASCE7-16 [1] and the recent publication of Iranian regulations such as the fourth edition of Article 6 of the National Building Regulations of Iran(INBR-6) [2] for residential buildings, the possibility of collapse against maximum ground excitations should not be Exceed ten percent. However, according to the usual rules in regulations such as Iranian

Standard 2800, buildings (especially buildings of medium importance such as residential) are just designed according to the method and restrictions based on earthquakes with a return period of 475 years [3] .

The use of analytical methods to construct or reinforce structures against a phenomenon with completely indeterminate characteristics can inform structural designers about the reliability of the analysis methods of previous design regulations against earthquakes. Where the use of probabilistic analysis methods for the design and analysis of structures has recently been permitted, the methods used in this study will be applicable to actual projects under design. [4]

In seismic design of structures, it is necessary to specify the design objectives. Regulations are developed in accordance with the predetermined goals. In the design of buildings against non-seismic or conventional loads (dead load, live load, wind load, snow load and loads due to environmental conditions), Prevention of structural and non-structural damage is one of the main goals of the design. If in seismic design, these targets are considered uneconomical for strong earthquakes; Because to meet this goal, the cost of construction of structures is such that it will diminish this incentive. The expression of seismic design objectives depends precisely on the seismic loading characteristics. Due to the problems in accurate prediction of seismic load, the science of probability is usually used in their definition and the amount of seismic force is defined in accordance with the seismicity conditions and the history of earthquakes in the region. In order to express the goals of seismic design, it is necessary to first pay attention to the characteristics of earthquakes and its different levels in order to better define the goals of seismic design [5].

Because linear analyzes cannot model the actual behavior of structures, nonlinear analyzes are generally known as more accurate analyzes. In this regard, we can name the nonlinear static analysis method (pushover). This analysis is a good method for predicting deformation and force needs and presents all the important features of nonlinear response. In this analytical method, lateral loads are used as predetermined patterns and approximate the relative inertial forces at the position of generalized real masses. Then, using the existing methods, they determine the displacement of the target in the desired structure and increase the lateral force under a specific pattern until the displacement of the target is achieved. Deformations and internal forces are calculated at each level of displacement. In nonlinear dynamic analysis of structures, deformations, internal forces and in general the response of the structure under one or more specific acceleration maps are calculated. Also in this analysis, the structural response is calculated by considering the nonlinear behavior of the materials and the geometric nonlinear behavior of the structure. In this method, it is assumed that the stiffness and damping matrices can be changed from one step to the next, but the time intervals of the steps are constant and the response of the structure under acceleration mapping is considered for each time step and using the methods Numerical can be calculated.

These are estimates of the need for deformation and strength of the structure that are comparable to the capacity of existence. This method provides a complete picture of the behavior of the structure from the elastic stage to the collapse. In this study, the effect of frame stiffness on the probability of its collapse and the amount of changes required to reduce these probabilities has been discussed. Push curves have been used to evaluate the stiffness.

One of the types of nonlinear dynamic analysis of structures is spectral analysis, which is better to use the u uniform hazard spectra of the site to obtain more accurate lateral forces. These spectra are obtained from the hazard curve obtained at the site using different reduction relationships and in terms of geotechnical and seismic characteristics of the site and according to the desired return period for the structures. Therefore, a structural frame can be considered safe in terms of seismicity in one building and unsafe in another building. In this research, a number of buildings with different conditions in Tehran have been used.

On the other hand, increasing the processing power of computers has made it possible to perform such analyzes with great accuracy. One of the newest methods of structural analysis is the incremental nonlinear dynamic analysis (IDA) method. This parametric analysis is based on nonlinear dynamic analysis and has been developed to investigate the performance of structures under earthquake impact. In this method, one or more accelerometer analyzes are scaled to several levels of intensity and applied to the structure. Hence one or more curves of parametric responses are generated against the intensity of the shock. [6]

The aim is to scale more accurately the entire behavioral range of the structure. At each scaling step, the structural model is analyzed under the desired acceleration maps and one or more curves of the damage response are obtained in terms of intensity. Using this curve, defining limit states, finally combining the results with the probabilistic analysis curve, structures can be evaluated. The unique information provided by this curve about the response of multi-degree-of-freedom structures can be a justification for the widespread use of this method despite its time-consuming and difficult process. [7]

In recent years, the failures observed due to multiple earthquakes have highlighted the weaknesses and shortcomings of the existing structural design method. This led to the creation of new concepts and methods in the design and performance evaluation of structures. The main issue in these methods is to consider the effect of uncertainties in the loading and complex behavior of structures in the nonlinear range in the design and evaluation process, and usually control the performance of structures at two levels consistent with the first failure and collapse. These methods are based on probabilities and in them, incremental nonlinear dynamic analysis is used to evaluate the performance of structures and more accurately estimate the demand and capacity of structures. [8]

The method is described in FEMA 350 and FEMA 351 regulations and gives the designer the ability to estimate the level of confidence or in other words the probability that the designed structure will be able to achieve the desired performance goal.

To draw fragility curves and estimate the probability of collapse, we are faced with two main categories of uncertainty, each of which can give these results a probabilistic form. Some of these uncertainties, such as record-to-record (RTR), are intrinsic and some are non-intrinsic. FEMA-

P695 uses a fuzzy approach to qualitatively evaluate non-intrinsic (cognitive) uncertainties. The rating is excellent, good, average or poor in terms of reliability, so the values of each can be 0.1, 0.2, 0.35, 0.5, respectively. [9]

Methodology

The PEER framework according to Figure (1) is based on risk analysis, structural analysis, failure analysis and damage analysis, which in this study, the first two steps have been used.

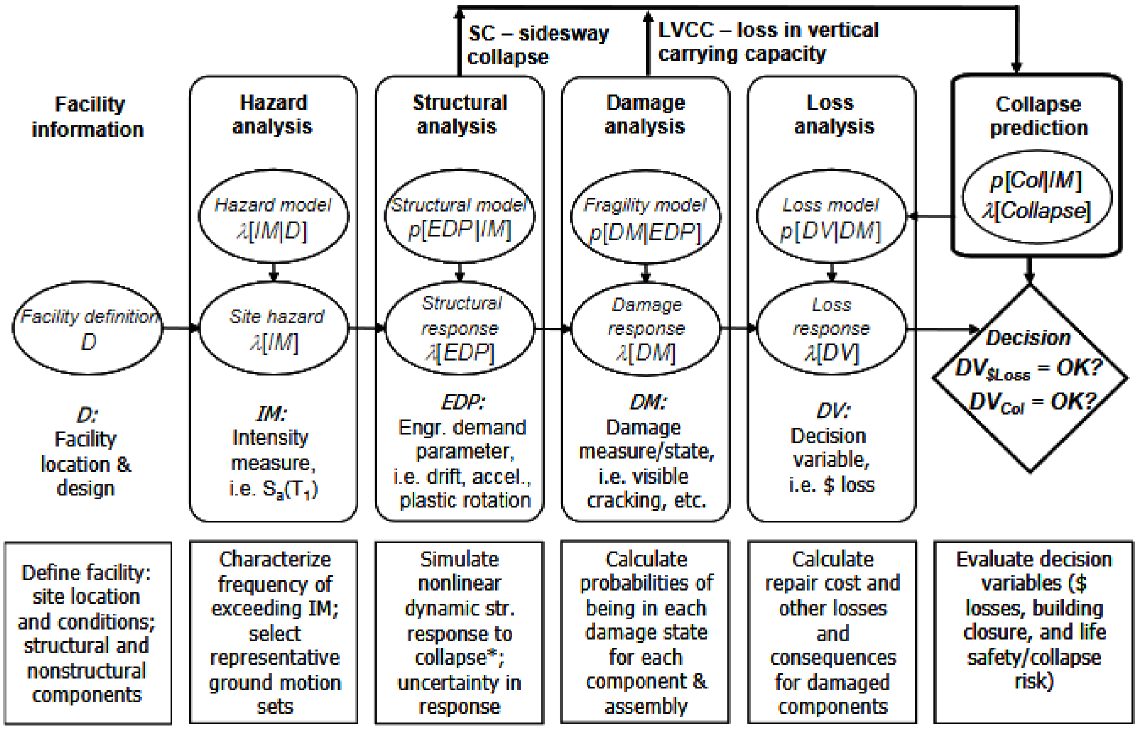


Figure 1:Provided PBEE framework for design based on probabilistic performance and quantification of collapse probability [10]

According to the mentioned process, risk analysis is the first step of the framework. In this project, using the data of Iran Seismic Hazard Analysis Database and soil classification of Tehran Geotechnical Studies Center for 6 sites according to Figure (2) from Tehran according to Table (1) The required spectral accelerations have been considered to obtain the uniform hazard spectra of all 6 sites as shown in Figure (4) through the method in ASCE7-16 as shown in Figure (3).

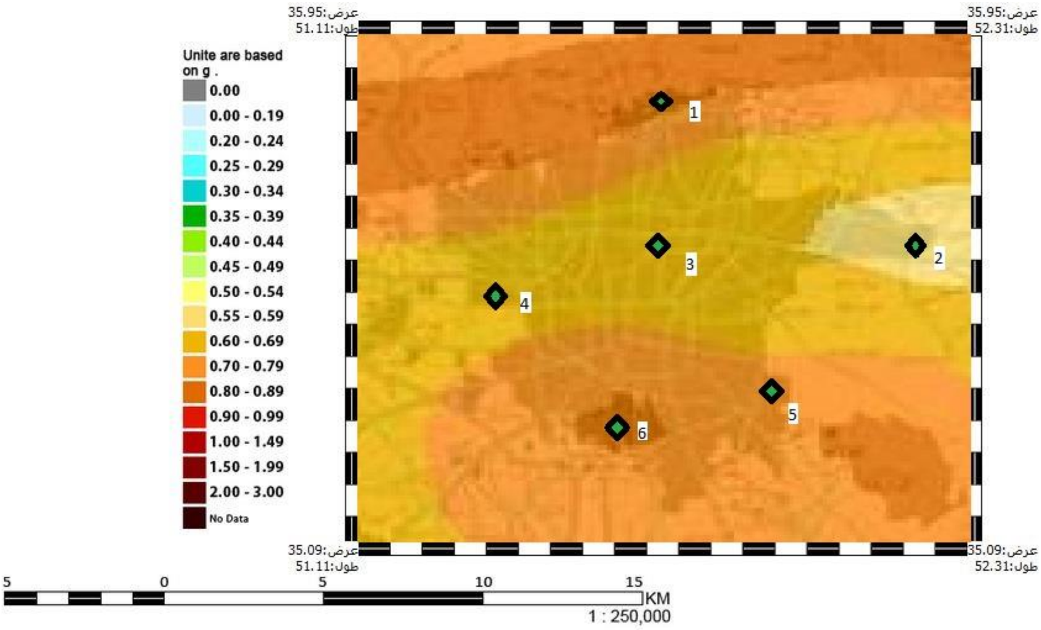


Figure 2:Zonation of Tehran in terms of pga for earthquakes with a return period of 2475 years

Table 1: Location and seismic characteristics of sites

Site	langitude	latitude	soil type	SA(0.0)	SA(0.2)	SA(1.0)	Ts
1	51.42	35.8	C	0.85	2.06	0.993	0.482039
2	51.38	35.72	C	0.576	1.4	0.518	0.37
3	51.35	35.71	C	0.67	1.621	0.574	0.354102
4	51.31	35.71	C	0.676	1.636	0.6	0.366748
5	51.49	35.63	D	0.773	1.852	0.758	0.409287
6	51.38	35.63	D	0.8	1.924	0.782	0.406445

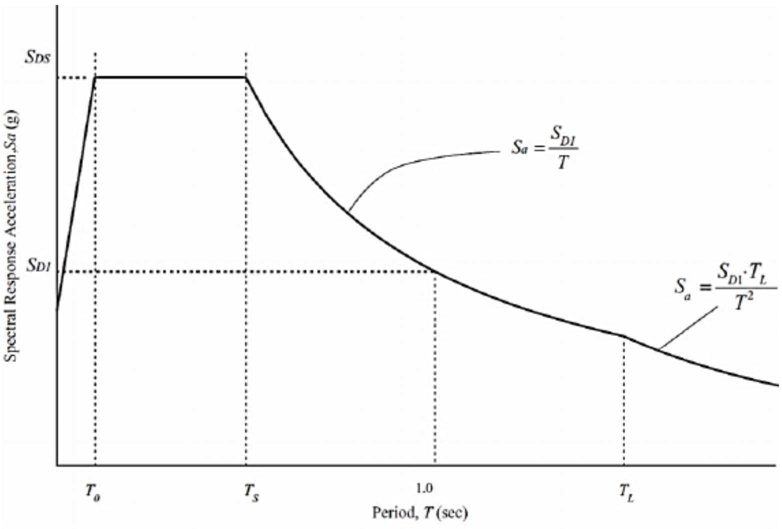


Figure 3: Parameters for finding the MCE spectrum according to the required spectral accelerations

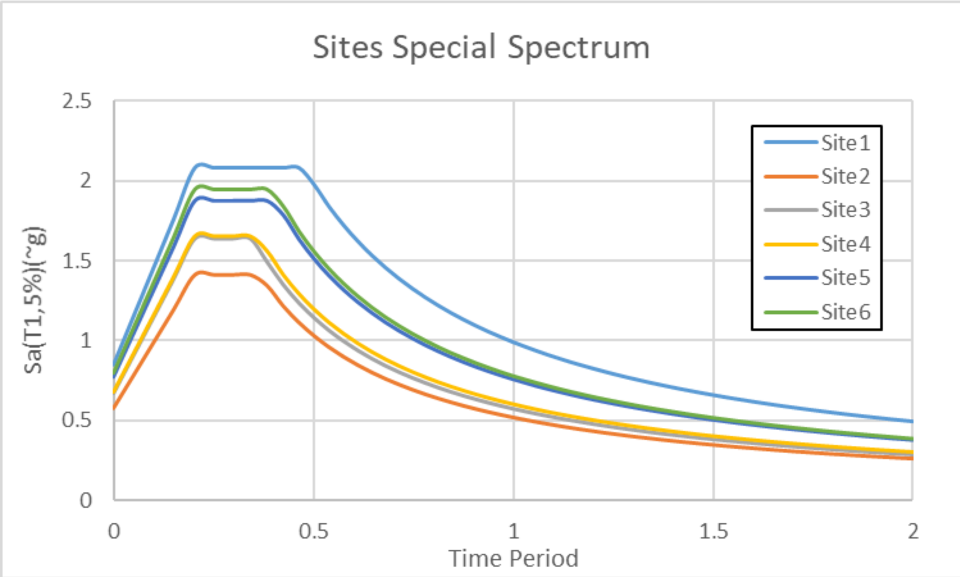


Figure 4:2475-year uniform hazard spectra of each site

The periodicity obtained in OPENSEES software for each of the frames and the spectral acceleration obtained for the frames in each site according to their uniform hazard spectrum can be seen in Table (2), in case of change in frame stiffness. The square of the fundamental period of the building changes as much as the inverse of the change in stiffness ratio, ie $T^2 \propto \frac{1}{k}$. This relationship can be used in later sections to calculate the strength required to secure the frames against collapse.

Table 2: Spectral accelerations and periodicity of analysis of each frame

Structural specifications		Spectral accelerations each site					
Frame	Time Period	SA(1)	SA(2)	SA(3)	SA(4)	SA(5)	SA(6)
IMF5	0.88	1.13	0.59	0.65	0.68	0.86	0.89
IMF10	1.66	0.60	0.31	0.35	0.36	0.46	0.47
SMF5	0.84	1.18	0.62	0.68	0.71	0.90	0.93
SMF10	1.58	0.63	0.33	0.36	0.38	0.48	0.49

The second step in the framework is structural analysis. The frames used in this study include 2 intermediate moment frames of 5 and 10 floors and 2 special moment frames of 5 and 10 floors with two bays of 5 meters on both sides and three middle bays of 7 meters. Which is designed according to the regulations of the tenth section of the National Building Regulations of Iran(INBR-10) and the 2800 standard (these regulations design the structure with the aim of LS performance level), with sections according to table (3). In this table, IMF stands for Intermediate moment Frames and SMF stands for Special moment Frames.

Table 3: Specifications of sections designed for each frame:

IMF-5		
Beam Section(mm)	Column section(mm)	Floors
IPE 450	BOX 450*450*15	5
IMF-10		
Beam Section	Column section	Floors
IPE 550	BOX 450*450*20	5
IPE 450	BOX 400*400*15	10
SMF-5		
Beam Section	Column section	Floors
IPE 400	BOX 450*450*20	5
SMF-10		
Beam Section	Column section	Floors
IPE 550	BOX 450*450*25	5
IPE 400	BOX 400*400*20	10

The performance level of the collapse threshold under the maximum ground excitations has been done by nonlinear static analysis and incremental dynamics in OPENSYS software with a concentrated plasticity approach.

INCREASING DYNAMIC ANALYSIS AND FRAGILITY CURVE

Incremental dynamic analysis was performed by 20 records as recommended by Shome and Cornell. Records are from the series of recommended records of FEMA-p695 according to Table (4).

Table 4: Specifications of earthquake records

No	Earthquake			Station data			
	Name	Station	Magnitude	VS_30(m/s)	PGA(g)	site-source dist	Field distance
1	Northridge	Beverly Hills	6.7	356	0.52	17.2	Far Field
2	Northridge	Canyon Country-WLC	6.7	309	0.48	12.4	Far Field
3	Duzce	Turkey Bolu	7.1	326	0.82	12	Far Field
4	Imperial Valley	Bonds	6.5	223	0.76	2.7	Near Field
5	Imperial Valley	Delta	6.5	275	0.35	22	Far Field
6	Imperial Valley	El Centro	6.5	196	0.38	12.5	Far Field
7	Imperial Valley	Chihuahua	6.5	275	0.28	7.3	Near Field
8	Kobe	Shin-Osaka	6.9	256	0.24	19.2	Far Field
9	Kocaeli	Duzce	7.5	276	0.36	15.4	Far Field
10	Northridge	Saticoy	6.7	281	0.42	12.1	Near Field
11	Landers	Yermo Fire	7.3	354	0.24	23.6	Far Field
12	Landers	Coolwater SCE	7.3	271	0.42	19.7	Far Field
13	Loma Prieta	Capitola	6.9	289	0.53	15.2	Far Field
14	Loma Prieta	Gilroy	6.9	350	0.56	12.8	Far Field
15	Kocaeli, Turkey	Yarimca	7.5	297	0.31	4.8	Near Field
16	Superstition Hills	El Centro	6.5	192	0.36	18.2	Far Field
17	Superstition Hills	Poe Road	6.5	208	0.45	11.2	Far Field
18	Cape Mendocino	Rio	7	312	0.55	14.3	Far Field
19	Chi-Chi	CHY101	7.6	259	0.44	10	Far Field
20	San Fernando	Hollywood Stor	6.6	316	0.21	22.8	Far Field

The results of IDA analysis obtained from this method along with 16,50,84% percentiles and the log normal density function of the data are plotted in Figures (5) and (6), in addition to the points due to the reduction of hardness to 20% hardness. The elastic of each diagram is selected for further consideration in these diagrams.

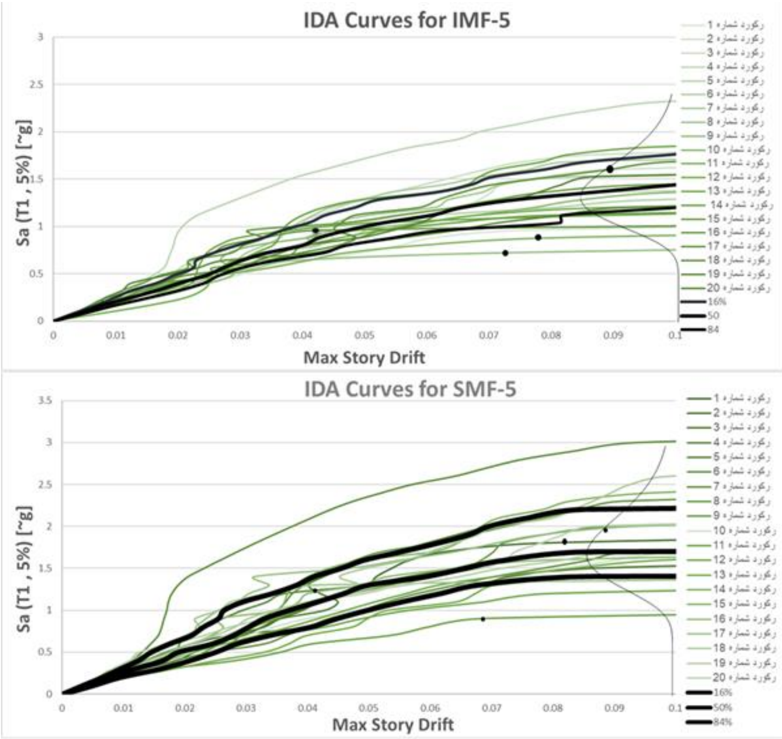


Figure 5: The obtained IDA curves are for 5-story moment frames

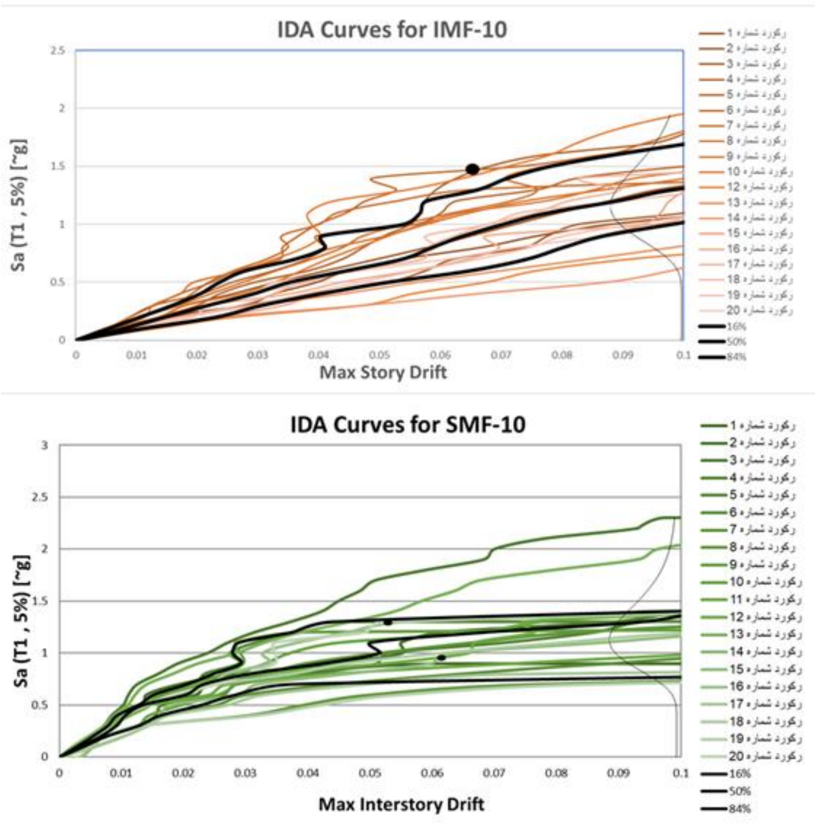


Figure 6: The obtained IDA curves are for 10-story moment frames

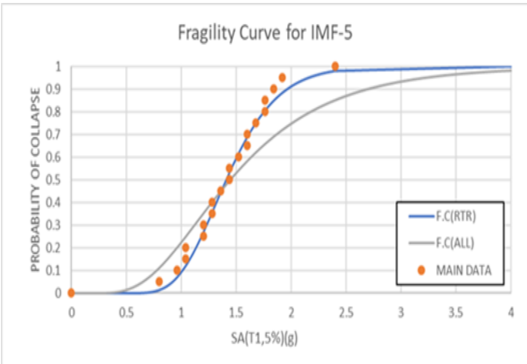
Collapse Threshold Performance Level refers to the performance level that is predicted to cause extensive damage to the structure due to an earthquake, but the building collapses and lateral losses are minimized. [11] In this case, we see a significant reduction in stiffness and strength. We will be a robust lateral force system, a large stable lateral displacement in the structure. Accordingly, in FEMA350, in steel moment frames, this limit is set for the IDA curve equal to 20% of the initial elastic slope or θ_max = 10%. [12]

In order to extract the probability of occurrence of limit states from IDA analysis outputs, diagrams called fragility curves are used. Using ordered values, the probability of occurrence of a limit state in the structure is calculated for values less than or equal to a desired IM value, which is a cumulative probability, and its graph is plotted against the IM value. Using this chart, we can say that for each IM level, the probability of a limit state occurs, provided that the IM value is limited to the desired level. EASYFIT software has been used to draw and fit fragility curves, which has the ability to fit and draw statistical curves in a specialized way with different methods.

After finding the fragility curves by the above method, the parameters corresponding to the cognitive uncertainties, which are considered here as the average of the recommended FAMA 0.35, with the variance used to plot the initial fragility curves by taking the square root of the sum of all 4 squares. Uncertainty is estimated according to Table (5) and is used to draw the final fragility curves according to Figures (7) and (8) while maintaining the middle of the curves.

Table 5: Uncertainty parameters

Frame	Inv(ln(μ)) [~g]	βRTR	βDR	βMDL	βTD	βTOT
IMF-5	0.34	0.26	0.35	0.35	0.35	0.66
SMF-5	0.62	0.25	0.35	0.35	0.35	0.66
IMF-10	0.25	0.27	0.35	0.35	0.35	0.66
SMF-10	0.33	0.28	0.35	0.35	0.35	0.66



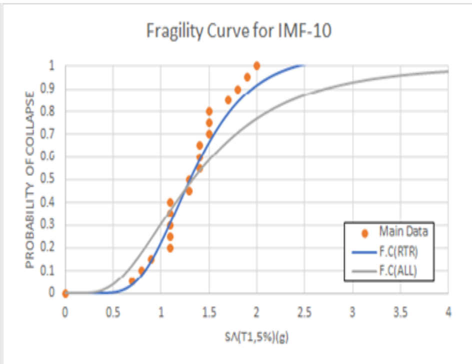


Figure 7:Fragility curves for 5and 10story intermediate momment frames Despite all the uncertainties

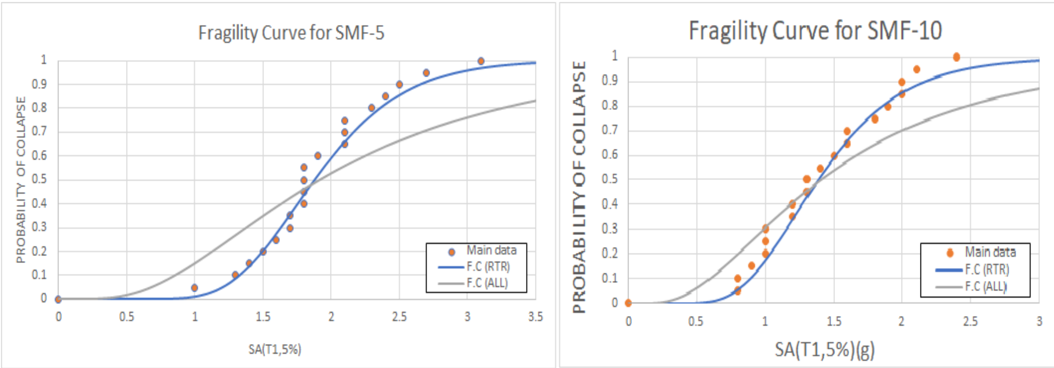


Figure 8:Fragility curves for 5 and 10 story special momment frames Despite all the uncertainties

The probability of collapse of moment frames is obtained by using the probability corresponding to their spectral acceleration in each site according to the final fragility curves in Table (6).

Table 6: Fragility curves for 5and 10story intermediate momment frames Despite all the uncertainties

Frame	Collapse probability					
	P(1)	P(2)	P(3)	P(4)	P(5)	P(6)
IMF5	0.22	0.06	0.08	0.08	0.13	0.14
IMF10	0.08	0.04	0.04	0.04	0.06	0.06
SMF5	0.2	0.05	0.06	0.06	0.07	0.07
SMF10	0.07	0.03	0.04	0.04	0.05	0.05

The results of the probability of collapse show that the intermediate and special 5-story moment frames in the first site and the 5-story intermediate moment frame in the 5th site are too unsafe for the probability of collapse. 10-story frames are safe in any of the sites in Tehran and do not need to be reinforced further. Therefore, in case of the need to build the mentioned insecure frames in sites 1 and 5, one of the ways is to increase the stiffness of the structure so that the probability of collapse for newer structures falls below 10%.

Nonlinear static analysis

In this section, nonlinear static analysis is presented to determine the general form of structural behavior of the studied frames in the static field in Figures (9) to (10). The lateral loading pattern is obtained by dividing the total shear lateral force by a triangular pattern and the displacement of 50 cm is obtained as a representative of the structural stiffness index in Table (7) for further calculations.

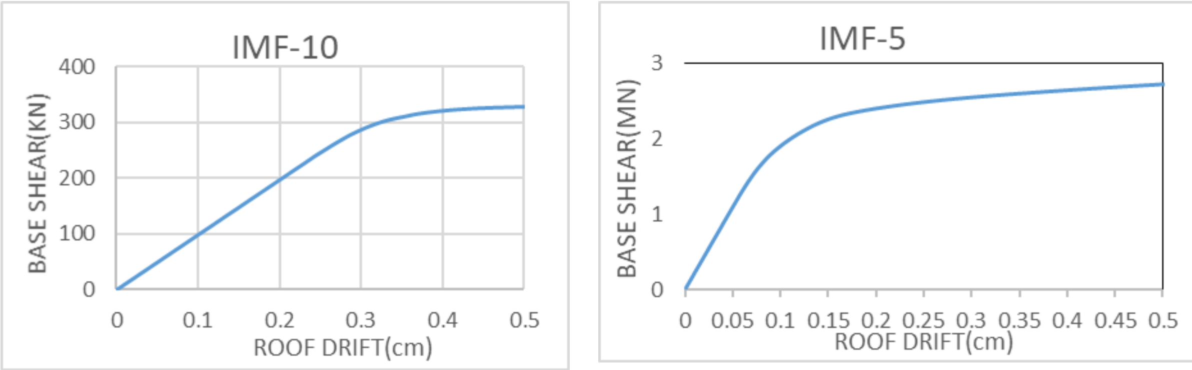


Figure 9:Intermediate moment frame pushover curve

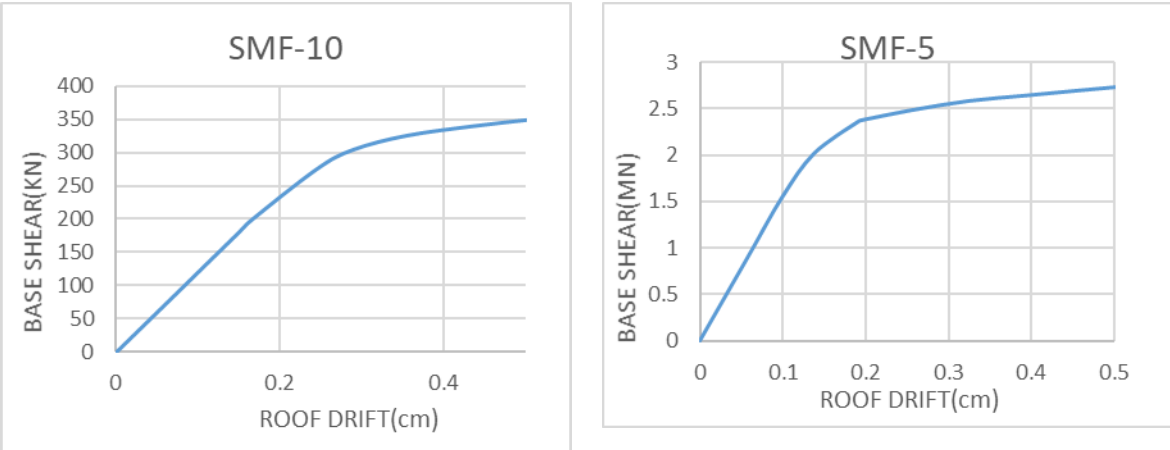


Figure 10:Special moment frame pushover curve

Table 7: The force required to make a displacement of 50 cm in the frames to millions of Newtons

Frame	F(dr=50cm)
IMF5	2.71
IMF10	0.32
SMF5	2.85
SMF10	0.35

Preparing to estimate the required difficulty

To find the probability of fragility in the mentioned methodology, it is necessary to draw the fragility curve through incremental dynamic analysis. To draw the mentioned fragility curves, the variance of spectral accelerations at the place of reaching the failure criteria of structures and their averages plays the main role. In this study, according to the studied frames, the force required to change the static position of the frames by 50 cm is summarized as K and B_{RTR} as a parameter of record to record uncertainty or variance of the main points of fragility and AVG as In the middle of them in the fitted diagrams, the mentioned data have been used in the probabilistic analysis of the frames in Figures (11) to (13). In this research, in addition to the data obtained for the mentioned parameters, the results of other researches have been used to provide a more complete diagram for the next tasks.

Table 8: Putting together the parameters of fragility and pushover analysis

Frame	F(dr=50cm)	BRTR	AVG	p(col)
CBF5	6	0.19	2.2	0.13
EBF5	3.5	0.22	1.9	0.015
SMF5	2.8	0.25	1.8	0.07
IMF5	2.7	0.26	1.4	0.13
OBF5	1	0.27	1.39	0.14
CBF10	0.7	0.28	1.38	0.004
EBF10	0.45	0.28	1.37	0.06
SMF10	0.35	0.28	1.35	0.05
IMF10	0.32	0.28	1.3	0.06

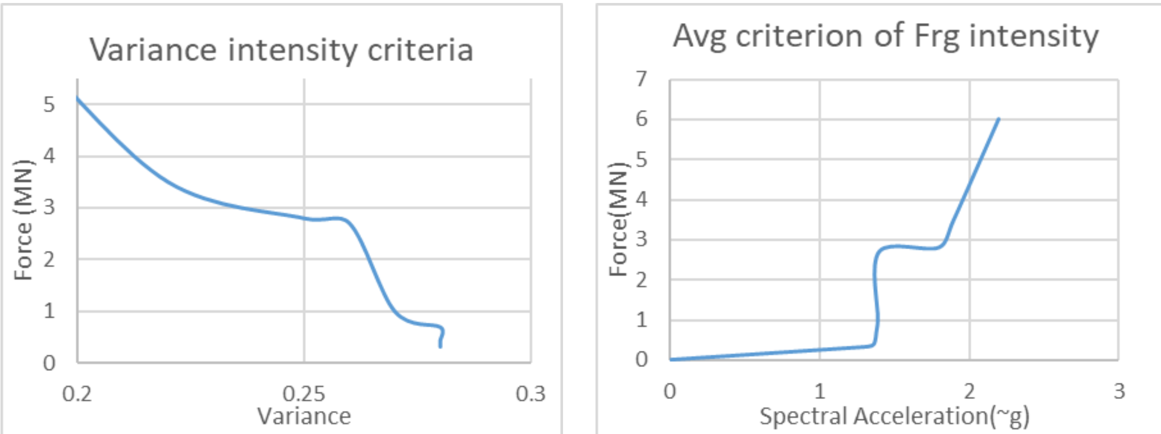


Figure 11:Relationship between fragility parameters and force corresponding to 50 cm drift

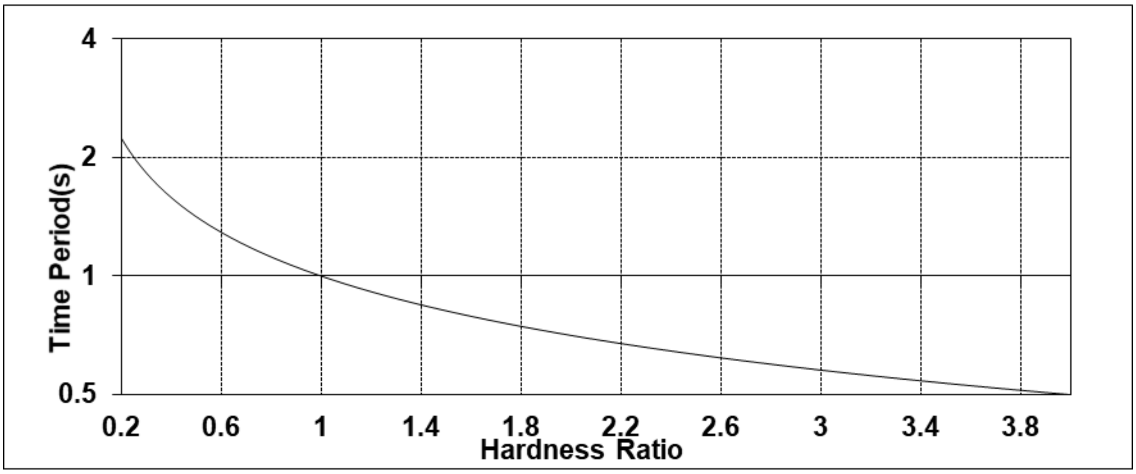


Figure 12: The relationship between periodicity theory and hardness ratio has changed

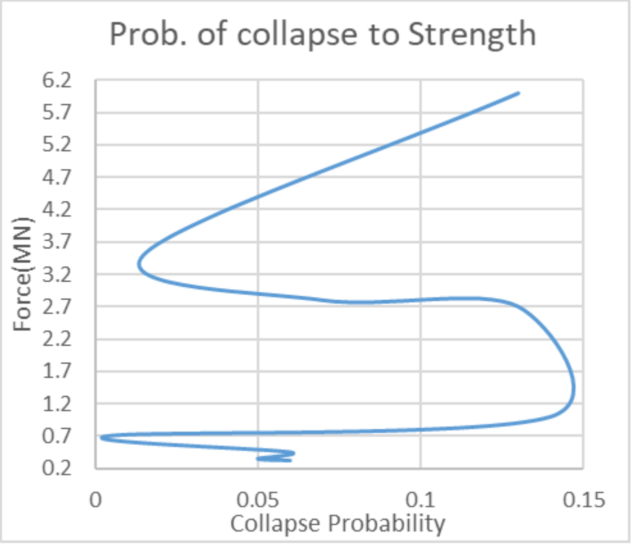


Figure 13: force of 50 cm drift corresponding to the probability of collapse

As can be seen in Figure (10), by increasing the stiffness required to bring the force index corresponding to the 50 cm drift to about 3 to 5.2, it can be assumed that the probability of the structure collapsing is less than 10%. As an optimal proposal, index 3.2 is proposed for the average stiffness of a 5-degree bending frame. And the periodicity of 10% is predicted to decrease due to the increase in the existing hardness, and with the increase of the mentioned hardness, for the final control of more accurate answers with guessed guesses, again increasing the dynamic analysis and fragility according to Figures (14) to (15) The necessary comparisons with each other are shown in Table (9).

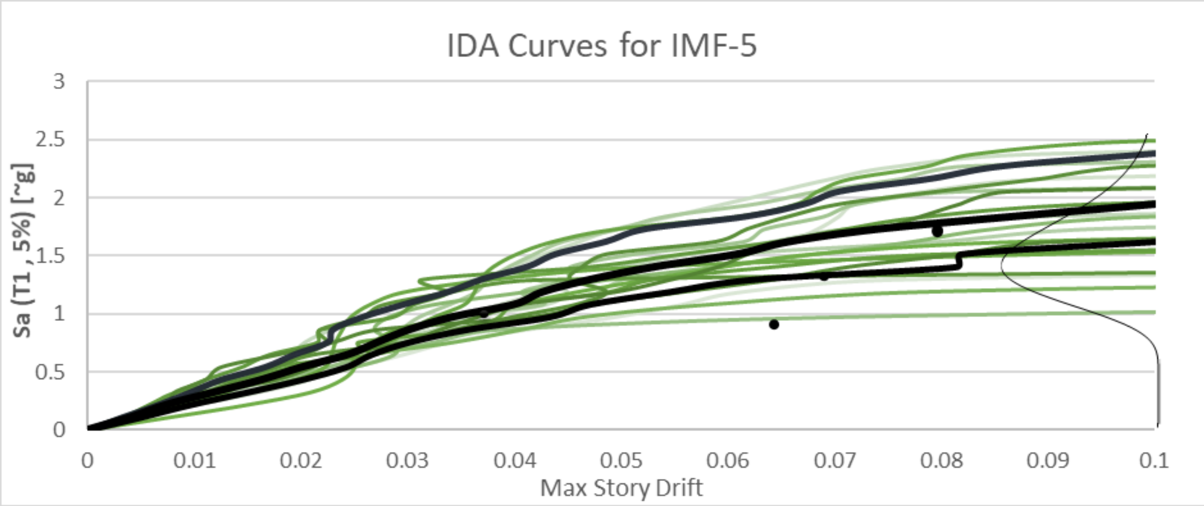


Figure 14: Intermediate moment frame IDA curve after increasing stiffness

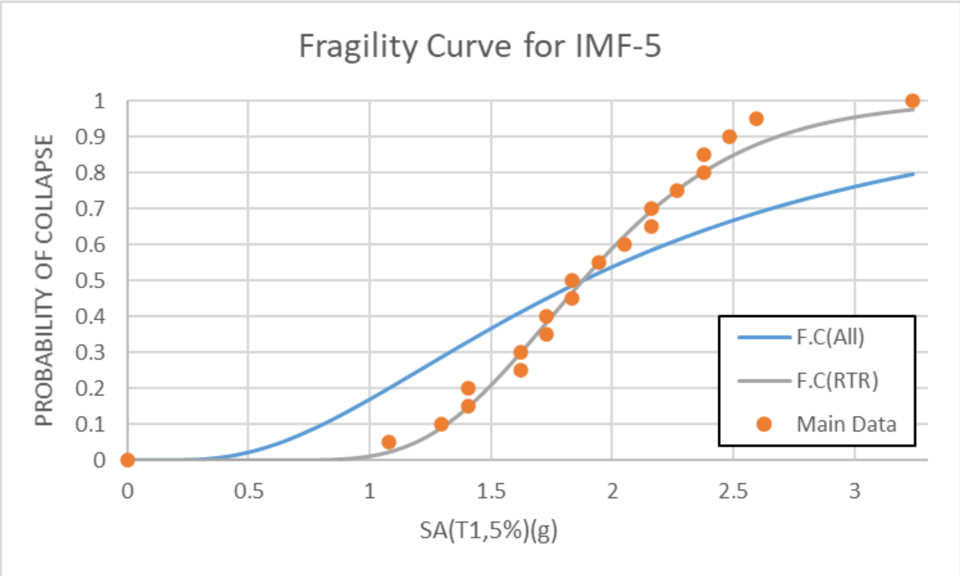


Figure 15:Fragility curves after increasing hardness

Table 9: Comparison of initial experimental parameters with final analyzed values

Parametr	Empirical	Analytical	%Eror
T	0.8	0.78	2.56
BRTR	0.23	0.25	8.00
AVG	1.9	1.94	2.06
Max(p(col))	0.1	0.104	3.85

CONCLUSION

Collapse probability analysis has been used as one of the tools to measure the safety of buildings with different uses, especially against maximum ground excitations. The dependence of different parameters of fragility analysis with structural analysis has complicated the possibility of obtaining the optimal hardness to find the probability of acceptable collapse and requires multiple analyzes. In this study, incremental dynamic and fragility analyzes were performed using 4 intermediate and special moment frames and its parameters were obtained. Then, assuming that each of them was located in 6 different sites in Tehran, their safety against collapse in different places was measured. Intermediate and special 5-story moment frames in two different sites out of 6 sites did not have the necessary safety. 5 Floors Intermediate moment frame to increase the strength required by the experimental method to achieve the probability of collapse below 10% in site 5 and using collapse probability diagrams against the selected force index to achieve displacement of 50 cm, variance and median spectral acceleration corresponding to the collapse and The fundamental periodicity of the structure against the selected index was tested with incremental dynamic analysis and fragility due to increasing stiffness and the results showed an error below 8% of the empirical and analytical values of the parameters of fragility. Using surveys, it can be said that some short-rise buildings in Tehran need to increase the stiffness and strength of the structure up to 35% against earthquakes.

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