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EFFECT OF FIRE-INDUCED PROGRESSIVE FAILURE ON THE PERFORMANCE OF 5-STORY STEEL BENDING FRAME BUILDINGS USING INCREMENTAL NONLINEAR DYNAMIC ANALYSIS

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ABSTRACT

This research has studied the fire-induced progressive failure in steel structures, which has been less studied in Iran up to now. To this aim, three-dimensional and two-dimensional models statistical tools such as mean and standard deviation as well as engineering software were applied in the analysis of data to analyze the time history of the structure. To make the results reliable, computational software such as SiesmoStruct and Etabs were applied, and to extract the probability of some cases from the IDA analysis outputs, diagrams called fragility curves were used, and finally, effective solutions were suggested. In this research, the vulnerability of the structure under progressive failure was studied in two steps in a 5-story steel model and using fragility curves, the probability of failure of each of them were determined in the form of fragility curves for CP performance levels. Finally, comparing the fragility curves, it can be realized that the probability of structural failure is lower for a more intact structure with a lower height than for tall structures. Therefore, structures with high ductility compared to other cases, have less annual damage and structures with low ductility can be considered as the next option. Of course, it is essential to explain that the cost of construction of each structure should also be considered by contractors as the cost of construction of a structure with high ductility is much higher than other structures.

INTRODUCTION

Earthquake as a destructive phenomenon in most parts of the world threatens the safety of constructions and their inhabitants, so that decreasing the irreversible damage of this phenomenon at the lowest cost, has always been the final aim of earthquake engineers, researchers and scientists. With the development of applied sciences, particularly computer sciences, earthquake engineering ideas and perspectives have also upgraded significantly. Extensive research is now underway on the behavior of existing systems and new tools and systems.

Earthquake resistance has long been considered in the design of constructions. For this aim, regulations have been made around the world with the main objective of protecting human lives and decreasing casualties, as well as reducing damage and financial costs. In other words, the purpose of these regulations is to make a proper safety margin for people's lives and property by monitoring the design, construction, quality of materials, service, application, location and maintenance of constructions. These regulations include a combination of simplified theories as well as some empirical views based on engineering judgment. Generally, all these regulations were formed with the following three objectives: first, to maintain the normal function of the construction under normal loads and service in the elastic range; second, not to collapse under load limits and ensure the safety of residents; and third, not to collapse under accidental loads such as fire, human error, etc. In other words, the construction must have a high degree of redundancy. For example, one of these regulations is the 2800 regulation of Iran. In this regulation, generally, three special levels are provided for designing the final conditions of the building. In each of these levels, for each building system, a coefficient is considered as a behavior coefficient to take into account the nonlinear behavior of structural members. Besides, criteria for ensuring the ductility of beams, columns, and other lateral load-bearing members are provided in such a way that, as far as possible, plastic joints are made first in braces and beams and finally at the junction of columns to the foundation. This mechanism is equivalent to no damage and destruction of roofs and providing vertical load by structural members at the time of the earthquake and after that with a return period of 475 years. In these regulations, not only is the employer not able to decide on the performance of its construction against different levels of seismic risk, but also due to the generalization of the conditions required by the regulations, sometimes the safety margin is such that it is not economical for the employer. On the other hand, the devastating earthquakes such as Northridge, Kobe, Lumapriteria had unpleasant and unpredictable results.

With the advancement of engineering science, numerous and new topics entered the design principles, all of which aim to optimize the design, increase the level of use and comfort, as well as maintain the safety of residents and constructions.

To achieve the above objectives, it is essential to predict the performance of the structure in a specific event that is itself affected by uncertainties. Earthquakes, explosions and fire are among the events that can be considered in the design of the structure as well as the prediction of its performance.

In performance-based earthquake engineering, or PBEE, it seems that seismic risk decision-making can be improved through assessment and design methods that state criteria for decision-making. Reports on the application of basic performance-based earthquake engineering approaches in the United States of America (FEMA-273, ATC 40) make this significant advancement in recognizing these ideas, which was a dream for professional earthquake engineers Earthquake pushovered by a society of action for many years. When the construction is loaded with earthquake lateral loads, the rate of failure and its nonlinear responses will behave according to the shape. Relationships between structural response indices (between classes, deformations of inelastic models and forces of members) and the described performance indices are the same as the levels of different failures.

The PBEE evaluation method addresses this issue that one way to describe it is through the idealization of pushover curves, which is mostly associated with the axes of the imposed shear base of the earthquake in the vertical axis and the drift between the floors in the horizontal axis. The idea of a static pushover can be related to the intensity of the incoming earthquake, which leads to performance values. Structural engineers have traditionally calculated performance by defining engineering response parameters such as deformation and structural forces. The goal of PBEE is to extend performance values directly. The first way of PBEE assessment, the same as FEMA273, attempts to relate structural response indices, which are drift between the floors, member forces, and non-elastic member deformations, to directional functional descriptions such as IO, LS, and CP. In any case, the estimated relationship between structural indicators and performance values in large sections is determined by calibration to produce the expected performance, which can have a great influence on the preparation of the current building regulation. Thus, competition remains for the development of these methodologies and models and inputs used to link the accuracy of engineering response parameters such as drift to performance values (damage, loss rate, and friction, etc.).

Tavakoli and Afrapoli (2014) have also discussed this issue in their article entitled "The effect of irregularity in the plan on the potential progressive failure of steel buildings with bending frame and dual bracing system". According to them, one of the challenges that threaten buildings today is the issue of progressive damage due to earthquakes, explosions and fires. Progressive failure is defined as the spread of local failure from one member to another that eventually results in the rupture of the entire structure or most of it. The purpose of this paper is to investigate the effects of irregularities in the plan on the potential for progressive failure. In this research, a five-story steel building with 5, 10 and 15% eccentricity with two types of bending frame and dual bracing systems has been investigated. In this study, which was performed using OpenSees software, after performing nonlinear static and dynamic analyzes, the potential of progressive failure was investigated. The results show that irregularity in the plan has a significant effect on the potential for progressive failure [1].

In another paper, Jiang and Qiang (2016) investigated the progressive failure of three-dimensional steel structures with concrete slabs exposed to fire. This paper numerically investigated the progressive failure resistance of three-dimensional steel structures with reinforced concrete slabs, dynamic analysis and assessment of computational costs [2].

In 2010, Vamvatsikos et al. used incremental dynamic analysis to estimate the sensitivity and uncertainties of seismic performance of structures [3]. Since several accelerometers are required to perform incremental dynamic analysis, Azarbakht and Dolsek have adequately addressed the number of records and the minimum number of records required [4]. Also, in FEMA P695 regulations, several records in the form of tables for the near and far fault areas are recommended [5].

Since Iran is located on the Alpine-Himalayan earthquake belt and has experienced devastating earthquakes, it has been significant to address this issue, which is why much research has been done on the impact of earthquakes on structural design. Following that, the probability of possible damages caused by the earthquake has been predicted.

American Institute of Standards and Technology's Committee of Experts, as standard 05-SEI 7/ASCE suggests the following definition: Progressive failure is the development of local failure from one element to another, causing sudden failure of the whole or incompatibility is a big part of that structure. This failure is also known as improper failure.

European codes do not have a specific standard for progressive failure, but do include standards for accidental phenomena such as fire, explosion, and impact.

In other words, whenever one or more members of a structure suddenly break, and then the building progressively collapses, in this case, each load distribution causes the failure of the other elements of the structures one after the other, until that a new state of interaction arises so that part or all of the structure is destroyed.

On the other hand, whenever one or more members of a structure suddenly break, and then the building progressively collapses, in this case, each load distribution causes the failure of other structural elements one after the other, until a new state of interaction arises, so that part or all of the structure is destroyed. The definition of disproportionate failure implies that progressive failure will be disproportionate when the total damage exceeds the allowable standard according to the given standard. Thus, the definition of disproportionate failure varies based on plausible risk prediction. Different design codes, guides, articles, or references usually select progressive failure when they refer to disproportionate failure. In the present study, in the same way, where the word progressive failure is mentioned, it implicitly expresses disproportion.

METHODS

Preparation of earthquake records related to the study area

The first step in the process of drawing fragility curves is to prepare a set of seismic maps that show the seismicity of the area. In the current study, brittleness curves have not been obtained for a specific region, so it is possible to select acceleration maps with almost similar conditions in terms of fault mechanism, distance to the desired site, earthquake magnitude and frequency content, and accordingly obtained brittleness curves.

Modeling and determining the amount of demand

Structural models are prepared based on the type of analysis, for example, for nonlinear analysis, the model must also comprise the properties of plastic. Then, for each earthquake record, the maximum response of the members during the earthquake is calculated, so that after all the analyzes, the demand values of diverse members are determined based on the different magnitudes of the earthquake. In the next steps, a relationship must be found between these discrete values that describe how diverse members of the structural change in response to different earthquake intensities.

Definition of structural failure states

Since in diverse regulations and instructions for different types of structures, failures are defined qualitatively, and based on each of these failure modes, brittle curves have been defined for the structure in question. Also, limit states for structural failure should be defined and to these limit states, small values should be attributed as relative displacement of the roof floor, and then brittle curves for the structure under study should be defined and introduced for passing through each of these failure states.

Failure criteria

Definition of limit states plays an important role in assessing the vulnerability of structures because they have a direct influence on the assessment of structural performance levels. In assessing vulnerability, choosing a simple and appropriate scale for injuries is very important to assess the demand and capacity system. There are two quantitative and qualitative approaches to describing performance levels or limit states.

Qualitative approach

This method has traditionally been used in building codes. FEMA273 and its newer version FEMA356 provide comprehensive information on performance levels that are qualitatively defined [6].

Quantitative approach

This method involves providing mathematically based damage indices based on the responses of designed structures. The description of the injury index is given in the previous sections. In 2004, Ghobarah introduced limit modes and performance levels as inter-story drifts (ISD) in the form of percentages [7].

ATC58 method for preparing fragility curves

In the definition of ATC58, fragility functions are functions in which the probability that one of the components of a system or element is affected by one or more states of damage is defined as the amount of seismic demand which is expressed as relative displacement or floor acceleration or another parameter that indicates the magnitude of the earthquake. Fragility functions are expressed as functions with a normal log distribution that has a mathematical form in the form of equation 1:

$$F_i(D) = \emptyset\left(\frac{\ln(D/\theta_i)}{\beta_i}\right) \qquad \text{equation 1}$$

where θ_i is the mean of the random variable values of the earthquake intensity parameter and β_i is the natural logarithm value of the standard deviation or distribution of this data. $F_i(D)$ is the conditional probability that expresses the probability that the structure will be damaged equal to or greater than the state of failure i according to the seismic demand parameter D. ϕ is the standard normal cumulative distribution function. The values of θ and β are calculated according to the type of component and the state of failure in one of the following methods.

Equation 2 is used to calculate the probability of structural damage in a particular failure condition and not more or less than that condition:

$$P[i/D] = F_{i+1}(D) - F_i(D) \qquad \text{Equation 2}$$

Where $F_{i+1}(D)$ represents the probability of the condition that the structure is likely to suffer damage equal to or greater than the failure state of i+1.

The peak ground acceleration (PGA) and the peak ground velocity (PGV) are usually considered as indicators to describe the intensity of earthquake movements. As higher acceleration does not always mean more severe structural damage, particularly in high-period structures, similarly, higher velocity do not always lead to more severe structural damage. Other indicators of strong ground movements such as peak ground displacement (PGD), duration of movements (T_d), spectral intensity (SI) and spectral properties can also be considered in estimating damage.

In this research, the peak ground acceleration parameter was used due to the extent of its application in earthquake engineering to consider the effect of strong earth movements. Another solution used in this research is the use of dynamic analysis of time history [8].

The process of preparing analytical fragility curves by the normal log distribution technique

Damage to a specific structure is divided into different and predefined states, and the fragility function is defined as the probability of increasing the amount of physical damage from the specified state. Based on the use of the failure index, its equation is written as equation 3:

$$Fi(IM)=P(di>DIi/IM) \qquad \text{equation 3}$$

Where $F_i(IM)$ is the conditional probability of increasing the failure from the i-failure state in the magnitude of the IM earthquake, di is the failure index in the magnitude of the earthquake (IM) and DI is the failure index threshold of the failure state i.

Estimation of the probability of failure index of the desired threshold is as follows:

$$Fi(IM)=P(di>DIi/IM)=\int_{DI}^{\infty} f_{IM}(DI)dDI \qquad \text{equation 4}$$

Where $f_{IM}(DI)$ is the density function of the probability of failure index in magnitude of the earthquake of IM. Equation 3-7 will be a function of the cumulative probability of the failure index as Equation 5:

$$Fi(IM)=P(di>DIi/IM)=1-FIM(DIi)=1-\int_{-\infty}^{DI} f_{IM}(DI)dDI \qquad \text{equation 5}$$

Assuming a normal logarithm distribution for the failure index, equation it is written as equation 6:

$$Fi(IM)=P(di>DIi/IM)=1-P(d\leq Di/IM)=1-\Phi\left(\frac{1}{\beta_{IM}}\ln\left(\frac{ID}{ID_{IM}}\right)\right) \qquad \text{equation 6}$$

In the first approach, the cumulative distribution function of the normal log can be fitted to the numerical results of equation 7:

$$p(D > d_i|IM) = \Phi\left(\frac{1}{\beta_i}\ln\left(\frac{IM}{IM_i}\right)\right) \qquad \text{equation 7}$$

According to Equation 6, $p(D > d_i|IM)$ is the probability of failure increase from the i^{th} threshold, IM is the parameter for measuring the intensity or magnitude of the movement of earth, and IM_i is the mean value for the failure state in which the structure experiences i^{th} damage, and β_i is the standard deviation of the normal logarithm function according to equation 8.

$$Fragility = \Phi\left(\frac{b\,Ln(IM)+Ln(a)-Ln(d)}{\beta_{D/IM}}\right) \qquad \text{equation 8}$$

Equation 8 is a standard normal cumulative distribution function. $\beta_{D|IM}$ is the conditional logarithmic standard deviation for the earthquake magnitude of IM, which indicates the distribution of data in a given IM. Parameter d also expresses the limit state.

The calculation of $\beta_{D|IM}$ will be such that at each point of IM_i the value of Ln($SD_i \times \sigma_i$) is plotted (σ_i is standard deviation corresponding to SD_i) resulting in several points above the current points. From the new points, a line passes parallel to the previous line that has the best fit of the points and the vertical distance between these two lines gives the value of $\beta_{D|IM}$ [14].

In the second approach, for each earthquake magnitude, the probability of increasing structural failure is obtained from the desired failure level and a curve is fitted on these discrete points. Therefore, if the mean values and standard deviation for each earthquake magnitude are known, equation 9 can also be used to calculate fragility curves:

$$Fi(D)=\emptyset\left(\frac{\ln(D)-\theta_i}{\beta_i}\right) \qquad \text{equation 9}$$

Where θ_i is the mean of the natural logarithm values of the maximum structural response at each earthquake magnitude, β_i is the standard deviation or the natural logarithm distibution of the maximum structural response values at each earthquake magnitude, and D is the limit state of member failure. Parameter ϕ is the standard normal cumulative distribution function. $F_i(D)$ is the conditional probability of member survival, or in other words, less damage than the limit state D in the earthquake magnitude of i. Equation 10 is used to obtain the probability of reaching or exceeding the limit state D [9]:

$$Fi(D)=1-\emptyset\left(\frac{\ln(D)-\theta_i}{\beta_i}\right) \qquad \text{equation 10}$$

Introduction of the studied models

To achieve the objectives of this project, first, a 5-story steel model with a supposedly three-dimensional cross brace, in an area with a high risk of seismicity and on type IV soil is modeled in the 2015 ETABS software. The reason for selecting this number of floors is to evaluate a high-rise structure and a mid-rise structure during failure. Then, this structure is designed based on the topics of steel (No. 10) and concrete (No. 9), edition 2013, and the 2800 Seismic Guide, edition 4 (the latest edition of the National Building Regulations). In this way, the essential sections are designed for columns and beams. Then a side frame of this structure, which seems to be a more critical frame, is selected and used in SismoStruct nonlinear software, and nonlinear analysis is done on them. In the next step, since the project aims to investigate the progressive failure in this structure, it is assumed that due to a fire or explosion or a special impact that occurs on the fourth floor of the 5-story structure, a beam and a column in the side frame of this structure are destroyed and lost their load and transfer of force and are also removed in modeling (step 1). In the next step, a beam and a column in the third floor of the 5-story structure in the side frame of this structure are destroyed and lost their load and transfer of force and are also removed in modeling (step 2) and removed from the structural elements. In this way, the behavior of the structure due to progressive failure can be observed and the probability of failure can be investigated for each step. In fact, in this project, one intact structural state and two damaged structural states for both structures and a total of 6 states can be observed and their behavior can be compared with each other in the face of a fire or explosion or a specific shock or different earthquakes. In the final step, nonlinear analysis is performed for each of these four modes and IDA curves and fragility curves are obtained for all cases. The nonlinear software used in this research is SiesmoStruct. In SiesmoStruct, increasing nonlinear dynamic analysis is performed on these frames under 10 selected earthquake records based on FEMA P695. In this study, the relative displacement of structures is considered as a criterion and indicator of damage. In each analysis, the desired levels of failure are determined. Evaluation of buildings after performing nonlinear dynamic analyzes for 15 selected earthquake records, using peak ground acceleration versus peak relative drift diagrams are performed, and finally, the fragility curves of the structures for the collapse potential (CP) functional level are determined.

Table 1: Characteristics of consumables in the project

ST37 steel materials	
Mass of a unit volume, M	0.8 ton/m^3
Weight of a unit volume, W	7.85 ton/m^3
Modulus of elasticity, E	2.1*10^7 ton/m^3
yield strength of steel, F_y	24000 ton/m^2
Final strength of steel, F_u	37000 ton/m^2

The characteristics of consumables in the project are given in Table 1. The thickness of the concrete slab is an important parameter that must be determined in concrete roofs before loading. According to the tenth topic, the thickness of the concrete slab in the combined beams is determined by the following equation:

$$t \geq \max(h+25mm, l/20, 80mm) \sim 150 \text{ mm}$$

The load of the structures is according to Table 2:

Table 2. Loading structures in the project

Type of load	Dead load	Live load
Floors + slabs	600 kg/m^2	200 kg/m^2

Sideloading

Depending on the layout of the shear wall, the distance between the center of mass and the center of stiffness in the X direction may be greater than 20% of the building dimension. In this case, the building will be irregular and since the number of floors is more than 5 floors, the equivalent static method can not be used to calculate and apply the lateral force of the earthquake and the use of dynamic analysis is obligatory.

According to the sixth topic, the dynamic base shear should be equated with the equivalent static base shear. Thus, it is essential to define a statically equivalent seismic force. Because the structural system is the same in two directions, the earthquake coefficient will be equal in both directions. The earthquake coefficient for determining the equivalent static base shear based on the sixth section is calculated as follows:

1. According to Table 2 of the 2800 regulations, new edition, and given the location of the building in a relatively high-risk zone, the acceleration basis of the plan is:

A = 0.35

2. Because the building is residential, the building is in the group of buildings of medium importance and the coefficient of importance is:

I = 1

3. Since the building system in both directions is a dual or combined system of special steel bending frame with special convergent steel bracing, the behavior coefficient of the structure in both directions is equal to:

Cd =5.2, R_{ux}=R_{uy}= Ru=5.5, Ω0 = 2.5

3. The reflection coefficient of the building in 2800 regulations, new edition, is calculated by the following equation:

B=B1*N

Since soil type is type IV, we have:

T0=0.15, TS=1.0, S=1.75, S0=1.1

To calculate T in cases where the interframe separators do not obstruct the movement of the frames, we have:

T=0.08H0.75

T= 0.76 sec 5-story structure with a height of 20 meters

Thus, the earthquake coefficient is:

$C = \frac{ABI}{R_u} = 0.1375$ For a 5-story structure

Characteristics of frames of primary structures

In this project, although incremental dynamic nonlinear analysis will be performed on only one side frame of both structures, so in all models, the focus is on these frames. The sections of beams and columns for the existing structures are described in Table 3, and also a general picture of the frame designed in the software environment will be provided.

Table 3. Sections used in selected structural frames of existing structures.

5-story structure			
Floor	**column**	**number**	**bracing**
First to third floor	Box 300*300*12	IPE 330	Tubo 100*100*8
Third to fifth floor	Box 250*250*12	IPE 330	Tubo 100*100*8

Selective structural frames for IDA analysis

The selected side frame is modeled in ETABS in SiesmoStruct software. It is necessary to explain that to decrease the volume of incremental dynamic analysis operations, which sometimes lasts for several days, two-dimensional frame modeling of the structure is used, which is in a more critical condition in terms of load level. For this purpose, it will be necessary to select several accelerometers.

Characteristics of accelerometers and their correction

The most important features of any accelerometer from the standpoint of being effective in creating structural response should be the amplitude, frequency content, spectral response and duration of survival. Amplitude is usually expressed by the maximum amount of acceleration or in some cases by the number of maximum accelerations that exceed a certain level.

The accelerometers used in the dynamic analysis of time history should be similar to the intense ground motions that can occur in the study area in terms of the effective factors mentioned in the structural response. In general, accelerometers can be divided into two categories:

A: Numerical accelerometers produced based on the spectrum of the design that are called artificial earthquake accelerometers.

B: Accelerated recordings of real earthquakes, which generally belong to the famous and destructive earthquakes that have occurred in different parts of the world.

The current study has used several scaled accelerometers to perform nonlinear analyzes using time history technique according to the desired functional purpose of the structure, which is related to the vertical component of the earthquake and represent earthquakes with a probability of occurrence of 2% in 50 years. All accelerometers are retrieved from the Pacific Earthquake Engineering Research Center (PEER; http://peer.berkeley.edu/products/strong_ground_motion_db.html PEER) website.

In increasing nonlinear dynamic analysis, when the peak ground acceleration (PGA) value and proportionally all the accelerations of a consecutive mapping are increased, the elastic response spectrum of that mapping increases in the same proportion. As the mapping spectrum gradually increases, this spectrum covers all parts of the design spectrum that are within the vibrational frequency range of different vibration modes of the structure.

Increasing nonlinear dynamic analysis curves

Incremental nonlinear analysis curves include several IDA curves, each of which is obtained using several nonlinear dynamic analyzes.

Selection of accelerograms

The first step in the process of evaluating the performance of IDA curves is to prepare a set of seismic maps, so that this set indicates the seismicity of the area.

Modeling frames in SeismoStruct software

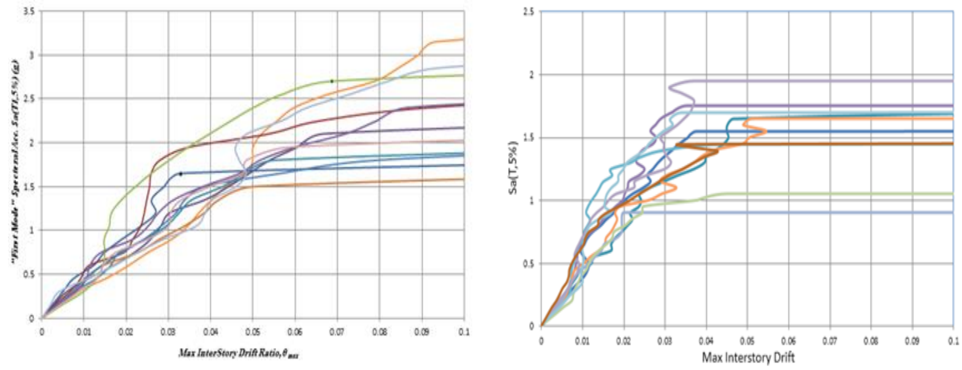
In this project, the lateral bending frames are modeled and analyzed in the form of two-dimensional structures modeled in SiesmoStruct software. It is notable that since incremental dynamic analysis is a very time-consuming non-linear analysis with a very high cost and volume, a two-dimensional frame is applied. After performing the analyzes, the parameter curves related to the response are drawn. These curves, which are usually plotted in the space of the two variables IM and DM, are known as IDA curves.

Based on the observations, modeling of structures was performed in ETABS and SiesmoStruct software and the outputs and results will be discussed below.

RESULTS

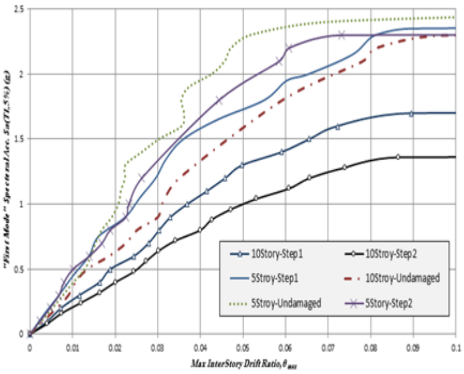
Obtaining curves from nonlinear dynamic analysis called IDA

To extract the probability of occurrence of limit states from IDA analysis outputs, fragility curves are used. To draw these graphs, the intensity of the IM earthquakes corresponding to the occurrence of the desired limit states is arranged in descending order for all accelerograms. As we will see in the following figures, the IDA curves for a 5-story structure in all three modes (intact structure[1], partially damaged frame[2] (by removing one beam and one column on the fourth floor), partially more damaged frame[3] (by removing one beam and a column on the third floor in addition to the removal of a beam and a column on the fourth floor)) are drawn. Summarized curves of 16%, 50%, 84% of each are given below to compare the behavior of the structures (figure 1).

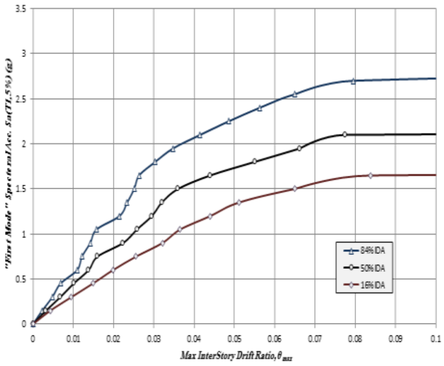


A B

[1] Undamaged Frame-5Story
[2] Step 1 Damaged Frame-5Story
[3] Step 2 Damaged Frame-10Story



C



D

Figure 1- a) IDA curves obtained for an undamaged 5-story structural frame, b) IDA curves obtained for a partialy damaged 5-story frame (by removing a beam and a column on the fourth floor), c) 16 %, 50% and 84 curves for the IDA curves obtained for the partially damaged 5-story frame (by removing a beam and a column on the fourth floor), d) Comparison of the 50% curves of the IDA curves obtained for all 6 structural frames.

Obtaining fragility curves

To obtain fragility curves using ordered values, the probability of occurrence of a limit state in the structure is calculated for values less than or equal to an IM value, which is a cumulative probability, and its graph is drawn versus the IM value. Using this chart, we can say what is the probability of occurrence of a limit state for each IM level, provided that the IM value is limited to the desired level. Considering the limit state of the CP destruction threshold, the fragility curves of the structures can be plotted. It should be noted that based on the FEMA350 guideline, the limit state of point destruction is equivalent to 20% of the initial mean slope corresponding to the starting point of the horizontalization of IDA curves. Fragility curves were obtained and their comparison were made for a 5-story structure in all three cases (intact structure[4], partially damaged frame[5] (by removing one beam and one column on the fourth floor), partially more damaged frame[6] (by removing one beam and a column on the third floor in addition to the removal of a beam and a column on the fourth floor)) (Figure 2).

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4 Undamaged Frame-5Story
5 Step 1 Damaged Frame-5Story
6 Step 2 Damaged Frame-10Story

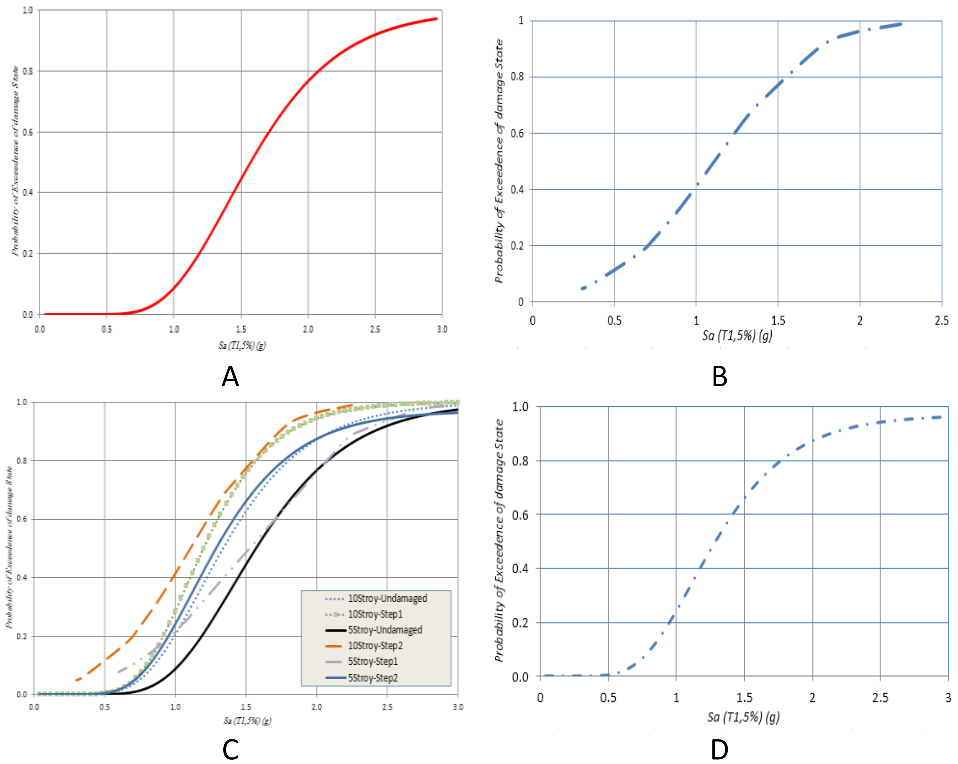

Figure 2 - a) Fragility curves from IDA curves for the level of failure corresponding to a undamaged 5-story structural frame for CP failure level, b) Fragility curves from IDA curves for the level of failure corresponding to a slightly damaged 5-story frame (By removing a beam and a column on the fourth floor) for CP failure level, c) Fragility curves resulting from IDA curves for the corresponding failure level for a slightly damaged 5-story frame (by removing a beam and a column on the third floor in addition to the removal of a beam and a column on the fourth floor) for the CP failure level, d) Comparison of the fragility curves obtained from the IDA curves for the failure level corresponding to the CP and IO failure level for all 6 structural frames.

CONCLUSION

1. In this study, the vulnerability of the structure under progressive failure in two steps was studied in a 5-story steel model and by drawing fragility curves, and the probability of failure of each of them was determined in the form of fragility curves for CP functional levels.

2. As the ductility of the structure declines, the structural failure increases and as a result, the IDA curves fall horizontally at the lower IM level.

3. Comparing the fragility curves, it can be realized that the probability of failure of structures at a certain and constant IM level is lower for a more undamaged structure that has experienced less initial failure due to fire.

4. In the performance-based design method, more realistic behavior of the structures can be obtained than before, in case of a fire. The most significant reason for the importance of discussion on performance-based design is to encourage the use of initiative in the development of methods to improve structural performance.

5. This study was performed for the development of suitable fragility curves with bending frame steel structures, and of course, it should be noted that these fragility curves only result from the possibility of failure of structural elements of some common structures in the country. The fragility curves for non-structural elements are obtained from their own methodology, which cannot be described in this study.

6. By applying the introduced method which is a part of ATC58 project to estimate the possible damages of structures and its application in the insurance industry of the country, it is possible to obtain damage curves easily to be able to reach a common language for decision making and information transfer between the employer and the engineers. Annual damage estimation is a useful and valuable tool for estimating the amount of building insurance.

7. For the structures studied in this research, we see that the structure with high ductility compared to other cases studied probably has less annual damage and the structure with low ductility can be considered as the next option. Of course, it is essential to explain that the cost of construction of each structure should also be considered by contractors because the cost of construction of a structure with high ductility is much higher than other structures.

8. Using the results of this research, it is likely to implement a major part of the new regulations such as ATC58 in Iran, which is now being approved as a new regulation in prestigious universities around the world.

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