Research Article

Collapse Probability of Immediate and Special Moment Frames in Tehran under MCE

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Residential buildings in most cities, which make up the most significant percentage of buildings, generally contain the most financial and human losses in the face of strong earthquakes. The purpose of this study is to investigate the possibility of the collapse of intermediate and unique steel moment frames against maximum ground excitations. In this study, through the first two steps of PEER methodology, using four steel structural frames with intermediate and unique moment frames, after designing according to the codes of national building regulations of Iran and standard 2800, this probabilistic evaluation was used to ensure their safety against collapse. In the next step, to deepen the results, 7 other sites from Tehran were selected. Their hazard spectrum was used to calculate the probability of collapse. In the end, it was observed that, with the reduction of the number of structural floors, the IDA curves at the lower IM level become horizontal in this project. The results showed that some of the 5-story steel structures under study in some parts of Tehran have a higher probability of collapse than acceptable.

1. Introduction

The earthquake phenomenon has long been considered as one of the sources of damage to structures. On the other hand, risk management of construction projects is one of the primary and essential goals and aspects of civil engineering, which has received less attention in recent years. Therefore, in this regard, for the structures of Iran, due to the high seismicity that we are witnessing, new probabilistic methods that can achieve more accurate and understandable results and have grown in the last decade as a turning point are used to test existing Iran national regulations [1]. Using these analytical methods for the construction or retrofitting of structures against all phenomena with indeterminate characteristics can inform structural designers about the degree of reliability of the methods of analysis of previous design regulations against earthquakes [2]. The safety of structures against collapse has always been an issue in earthquake engineering. The collapse of the structure means the loss of the ability of the structure or part of it to withstand the vertical forces acting on the structure [3]. Sun et al. researched the effect of gravitational force on the dynamic behavior of a system of one degree of freedom and change in the periodic content of the structure. They showed that the maximum displacement that subsequently causes the structure to collapse is directly related. With a coefficient of stability, displacement is like the point of submission of the structure [4]. Zareian and Krawinkler for a frame-shaped structure showed that, considering the reducing behavior, especially for the case where the P-Δ behavior does not govern the characteristic strain hardening of members, has a significant effect on seismic intensity such as the collapse threshold of the structure [5].

Lee and Foutch, in 2002, also examined the performance of steel moment frames as part of the FEMA/SAC project [6]. Jalayer also used the IDA concept in 2003 to estimate the capacity of the overall dynamic collapse threshold for concrete frame structures [7]. In his model, he considered that the reduction in column strength was due to shear failure based on the model proposed by Song and Pincheira [8]. In 2009, Sezen performed a full-scale experiment on structures with shear-sensitive concrete columns under cyclic lateral load to the point of collapse (to the extent that the other column could no longer withstand the applied axial...
The collapse of the columns does not occur immediately after the loss of lateral strength [9]. In this regard, it should be said that the calculations related to the collapse can be based on what happened after the failure of the column. For example, in this case, if we consider the collapse of the structure as a function of column failure and other unbearable gravitational loads by the structure after it, the concept is called progressive collapses. It refers to the spread of direct local damage within a structure. Following local damage due to removing one or more load-bearing members, the failure spreads in a chain in the structure. It causes the failure of part of the structure or the whole structure. Parisi et al. by studying several reinforced concrete flexural frames analyzed the sensitivity of single-column removal in several different scenarios concerning the effects of final steel strength parameters and single-column removal location on maximum bearing capacity, drift, and drift residue. They are vertically performed by IDA with 24 earthquake records. In models with the removed column on the ground floor and the corner, the highest demand was observed among all scenarios. Also, assuming $e_{su} = 20\%$ (the influence of ultimate steel strain) allows the catenary action of beams above the removed column to be effectively mobilized, providing a more realistic assessment of structural response and progressive collapse capacity. In that case, analytical estimates of vertical drift capacity become consistent with those derived by experimental tests [10]. Brunesei et al. studied two categories of flexural frames of short-reinforced concrete in a method using the theory of large deformation and independent of failure due to the removal of single columns, in the first category of load-bearing frames and the second category of load-bearing frames earthquakes; they concluded that secondary beams (included in 3D models) significantly increased the robustness of the case-study structures as they effectively contribute to controlling the overall structural response and provide an alternative load path for column loss-induced overloads [11]. Vian and Bruneau (2003) concluded in a study that the stability coefficient has the most excellent effect on the behavior of the structure, and by increasing this coefficient, the maximum tolerable drift for the frame and spectral acceleration such as collapse threshold decreases [12]. In 2000, Shinozuka et al. and fellow researchers developed fragility curves for steel, concrete, and wooden buildings in Kobe, Japan, using the damage function and various PGA values. Examination of the presented diagrams shows that reinforced concrete structures made in Kobe city have the lowest fragility and steel structures have the highest fragility [13]. In their joint paper, Zareian et al. proposed two methods based on seismic intensity and engineering demand to draw fragility curves from structural elements. These functions can be obtained for structural and nonstructural elements. By combining these two categories of failure curves with seismic hazard curves and IDA curves, it is possible to estimate the damage to the structure in the event of a possible earthquake [14]. In 2010, Vamvatsikos et al. used incremental dynamic analysis to estimate the sensitivity and seismic performance uncertainties of structures; they found that ductility, negative strength, and negative resistance are highly sensitive in estimating the performance of structures [15]. The present study is based on calculating the probability of collapse in residential steel structures designed according to the national building regulations of Iran under maximum ground excitations. Therefore the purpose of this study is to investigate the probability of collapse in the behavior of medium and unique moment structure frames.

2. Methodology

In order to achieve the objectives of this study, first, steel structures located in an area with high seismic hazard and located on type III soil category of standard 2800 are modeled in ETABS software. For each of the mentioned structures, a code in the form of XXX-X is considered, the first part of which is related to the type of lateral bearing system and the second part specifies the number of floors (each floor height 3 meters). Then, according to the tenth topic of the national building regulations of Iran version 92, after this referred to as INBR 10 and standard 2800, version 4, the models are designed. Thus, in this project, we will find the required sections for the beams and columns of the frames, which can be used in an incremental dynamic analysis called IDA. The software used to perform and program nonlinear analysis is Opens. In this study, the maximum relative interstory drift of frames is considered a criterion and engineering parameter of damage. Evaluation of buildings after performing nonlinear dynamic analyses for 20 selected earthquake records has been determined, and finally, the fragility curves of the structures have been obtained for the collapse level (CP) at the functional level. Because models are designed for residential use, the probability of the collapse of the frame according to ASCE7-16 regulations should be less than 10%.

The plan of the structures, according to Figure 1, has six bays in the X direction and five bays in the Y direction. The plan of the structures, the dimensions of the bays, and the direction of applying load from each shell to beams are following the reference hypotheses [16]. Also, as can be seen in this figure, one-sided alternating slabs are used for the roof of each floor, which is a common idea in ancient buildings. Move in one direction. This bearing system works reverse for the other span to distribute the load in parallel (evenly spaced) to the other spans. According to the subject of the present study, the composite slab is one of the most widely used types of roofs in steel buildings, which transfers its load one-way due to the placement of side beams. The side beams are of IPE160 type and are located at a distance of 50 cm from each other. Also, the thickness of the slab is considered to be 12 cm according to Article 9 of the National Building Regulations of Iran.

Figure 2 shows the perspective of one of the five-floor models and the frame’s location under study. Also, the loading of structures is based on the sixth issue of national regulations of Iran (INBR 6) according to Table 1, which is distributed linearly on two-dimensional frames in proportion to their load-bearing surface, that is, 3.6 meters from the length of the load-bearing floor. The legs of the columns are naturally assumed to be stuck, there are no various
irregularities, including stiffness distribution irregularities in the 3D model, and they are placed in two categories of immediate and special ductility against earthquakes. The specifications of steel materials related to structural members are also mentioned in Table 2. Table 3 also shows the seismic performance factors “R” used for frame as the 1st dynamic linear spectral analysis for design based on INBR that has force-based design approach; this factor involves the nonlinear performance of each system due to reducing the spectral acceleration that finally results in “C” factor of seismic effective weight and from there base shear of the frame. At the last step of this procedure, Table 4 shows sections that were designed.

Then, the allowable amount of drift is controlled according to the standard 2800, which is described in

Table 1: Load amount of models.

| Load type   | Dead load | Live load |
|-------------|-----------|-----------|
| Floor + slab| 500 kg/m² | 200 kg/m² |

Table 2: Specifications of steel materials used.

| Steel material type ST-37 |
|---------------------------|
| Elasticity module, E       | $2.1 \times 10^7$ ton/m³ |
| Yield stress, $F_y$        | 24000 ton/m²             |
| Ult. strength, $F_u$       | 37000 ton/m²             |

Table 3: Seismic factors.

| Frame | Seismic resistant system         | Pref. factor | $c$       |
|-------|---------------------------------|--------------|-----------|
| IMF-5 | Intermediate moment frame       | 5            | 0.1796    |
| IMF-10| Intermediate moment frame       | 5            | 0.0788    |
| SMF-5 | Special moment frame            | 7.5          | 0.1197    |
| SMF-10| Special moment frame            | 7.5          | 0.0788    |

Table 5. $C_d$ factor is a parameter that multiplies the drift of frames to reach the actual amount of drift, as the forces decreased with the $R$ parameter in the lateral loading step.
In this section, first, nonlinear static analysis is presented to determine the general form of the structural behavior of the studied frames in the static field. The lateral loading pattern is from the base shear splitting method with a triangular pattern. The target displacement is from the relationship of issue 360 is assumed. More than the relationship of issue 360 is assumed.

According to research, spectral acceleration in the first mode of structure with 5% damping (Sa (T1,5%) (g)) is a suitable choice for intensity (IM) and considering that the discussion here is on the general behavior of the structure and structural components, the maximum relative displacement of the floors is also selected as the Engineering Demand Parameter (EDP). In order to model the mentioned frames in OpenSees, first, the nodes were introduced, each with coordinates related to the beginning and end of beams, columns, and braces. These points, as mentioned, will behave in a two-dimensional ambiance with three degrees of freedom, including two degrees of transition and one degree of rotation.

Because in this study, seismic solid excitations are used for lateral loading of frames, many structure members enter into their nonlinear behavior, so a secure method should be used to consider such behavior. In this research, the concentrated plasticity behavior model, as shown in Figure 5, is used to consider the nonlinear behavior. In modeling this study, what is considered according to Figure 6 of the software, in general, is a uniform behavioral curve in the loading mode, and if the effects of cyclic loading are considered, using the Landa parameter and bilin material materials for each member according to it can be done according to its geometric characteristics [17].

Since the structural members have become two parts, the end springs and the middle member, in the approach of concentrated plasticity, and the energy absorption in these two parts is not the same, modifications must be made for several arrays of the stiffness matrix of each member. The “modellasticbeam2d” member has been used for structural members between plastic joints [18]. This member changes the stiffness matrix at each moment of loading by changing the arrays of the stiffness matrix corresponding to each member and according to the degree of division of the total stiffness of each member between the plastic joints and the middle member [19]. According to previous research in the modeling under study, if we assume that the total stiffness of each member is 11, 10 parts of it are allocated to springs [20]. Finally, a general comparison between the periods obtained from the ETABS and OpenSees models can be seen in Table 6. As can be seen, the maximum difference between the periods obtained from the two software is less than 13%.

FEMA-P695 introduces two categories of far-field and near-field records for seismic design of structures, suitable for seismic design of structures against MCE earthquake regardless of the scale factor. In order to meet the site requirements of the structures, only the records have been selected with different geotechnical conditions of the seismic station and site conditions, such as average shear wave velocity to a depth of 30 meters (VS30). It should be noted that this standard specifies the desired speed range of 175 to 375 meters per second. According to the

### Table 4: Designed sections.

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

### Table 5: Frame drift control.

| No. | Frame | Max-drift | C_d | Allowable drift |
|-----|-------|-----------|-----|-----------------|
| 1   | IMF-5 | 0.0059    | 4   | 0.00625         | Accept          |
| 2   | IMF-10| 0.0048    | 4   | 0.005           | Accept          |
| 3   | SMF-5 | 0.0043    | 6   | 0.0045          | Accept          |
| 4   | SMF-10| 0.0033    | 6   | 0.0036          | Accept          |

\[ d_f = \frac{C_d C_1 C_2 S_a(T_e)}{4p^2 g} \] (1)

The push curves of moment frames can be seen in Figure 3; the force required to be submitted to average first yielding intermediate moment frame is always lower than the particular moment frame; it could be due to the fact that this model covers the INBR like the less seismic coefficient of special frame rules which is more stringent and also weak beam and firm column control. For incremental dynamic analysis in this research, Figure 4 presents a two-dimensional side frame of the structures modeled in OpenSees software and analyzed. It should be noted that since incremental dynamic analysis is a very time-consuming and costly nonlinear analysis with a very high volume of operations, a two-dimensional frame has been used according to Figure 4, which is drawn for a 5-story frame (the dimensions of the bays and the height of each story in a 10-story frame are similar to 5 floors) because the need for supercomputers seems to be necessary.

\[ \text{Demand Parameter (EDP). In order to model the mentioned frames in OpenSees, first, the nodes were introduced, each with coordinates related to the beginning and end of beams, columns, and braces. These points, as mentioned, will behave in a two-dimensional ambiance with three degrees of freedom, including two degrees of transition and one degree of rotation.} \]
consideration of these conditions, a total of 16 records of far-field circumference and four records of near field without impact have been selected according to Table 7 for nonlinear analysis [21].

The results of incremental dynamic analysis (IDA) obtained from this method with plots of 16, 50, and 84% and the normal log density of the data are plotted in Figures 7 and 8, in addition to the points due to the reduction of hardness by 20%. The elastic stiffness of each diagram is selected for further consideration in these diagrams.

Collapse threshold performance level refers to the performance level that is predicted to cause extensive damage to the structure due to a lateral load, but the building collapses and lateral losses are minimized [22]. In this case, we will see a significant reduction in the stiffness and strength of the lateral-resistant force system, a large stable lateral displacement in the structure. Accordingly, in FEMA350, in moment frames, this limit is set for the IDA curve equal to 20% of the initial elastic slope or \( \theta_{\text{max}} = 10\% \) [23]. In this study, the same limits have been used to estimate the collapse for the drift criterion. In order to extract the probability of limit states from IDA analysis outputs, diagrams called fragility curves are used. To draw these graphs, the IM

Figure 3: Push curves for 5-story and 10-story frames from (b) to (a), respectively.

Figure 4: Overview of the 5-story moment frame modeled in OpenSees.

Figure 5: Overview of ZeroLength springs and their placement in a single moment frame.
seismic intensity corresponding to the occurrence of the desired limit states is arranged in descending order for all accelerometers; after that the main data are calculated cumulatively. Log-normal fit curve using mean and standard deviation and PDF curve can be obtained. Because only record-to-record uncertainties are used in this curve, it is abbreviated to FC (RTR) in the diagrams. Figures 9 and 10 show the resulting curves for the analyzed models.

In order to draw fragility curves and estimate the probability of collapse, we are faced with two main categories of uncertainty, each of which can give probabilistic results. Some of these uncertainties are inherent, and some are cognitive. Since the mentioned uncertainties have different sources, each of them is a random variable independent of each other and can be combined statistically. In this study, as in other studies for this type of modeling,
[16, 24] have been used. The final values of the uncertainty parameters (variance of the collapse probability cumulative distribution function) for the studied models are obtained according to Table 8.

Regarding other uncertainties on the fragility curve, it is necessary to change the original variance of the data to the variance obtained from the combination of uncertainty parameters. Using this combination, the final fragility curves (FC (RTR)) take a more dormant form according to Figures 11 and 12.

Then, the probability of collapse is obtained with spectral acceleration with different spectrums being done. According to Figure 13, it is noteworthy that the periodicity used is the periodicity of the first mode of the frame in the modal analysis performed by OPENSEES software. In Figure 13, a comparison of the uniform hazard spectrums of the site (IHA Project) shows that the design spectrum of standard 2800 (St2800-DBE) is the product of the acceleration of the design basis in the building reflectance.

The factor for soil type 3 according to this standard and to measure the safety of buildings in it has been prepared for 475-year-old earthquakes and also the maximum range of motions of earthquake, called St2800-MCE, which is 50% larger than the range of the 2800 design base earthquake, and is based on earthquakes with a return period of 2475 years. Since all the models under study have a periodicity of more than 0.42 seconds (the intersection point of the 2475-year standard design spectrum of the 2800 standard and the uniform hazard spectrum of the Iran Hazard Analysis...
The probability of collapse obtained in Table 9 showed that the probability of collapse obtained for 10-story frames is less than that of 5-story structures for the structural systems studied. The probability of collapse obtained for a particular 5-story moment frame is less than the intermediate 5-story moment frame. The probability of collapse for unique 10-story frames is almost close to the intermediate 10-story moment frame. This indicates that choosing the safest structural system for each height system or choosing the most optimal one will not lead to a specific system.

3.1. Investigation of Models in Several Different Sites in Tehran.

To further investigate the results, sites from the city of Tehran were selected, and the models of the structures under study, assuming that the models are in seismic conditions of each of those points, were analyzed for the possibility of their collapse.

Table 10 shows the data received from data sources. Source [25] was used to give data of site S2. As Figure 14 shows, other sites were chosen from different urban areas with different seismicity of Tehran. To observe the behavior of frames in each seismic zone from source [26], soil types have been taken from the soil zoning map of the Center for Geotechnical Studies and Material Strength of Tehran that shows the soil type of each coordination of Tehran according to the standard soil classification of standard 2800. According to the method of ASCE7-16, the 2475-year uniform hazard spectrum, which
is the maximum ground motion, can be plotted concerning the corresponding spectral acceleration of 0, 0.2, and 1 seconds, so that the spectral acceleration in the homographic part of the spectrum is divided from the spectral acceleration of second 1 over the desired periodicity. The one-second spectral acceleration in Table 10 is given for this purpose. It is noteworthy that, with the calculations performed for the models concerning the spectral acceleration of 0 and 0.2 seconds, the periodicity of none of the frames outside the homographic part of the spectrum function is not endangered. Therefore, to calculate the frames' spectral acceleration, the mentioned homographic relationship has been used, and the results can be seen according to Table 11.
The acceleration corresponding to the periodicity of each frame in Table 11 shows that considering a design range for the city of Tehran according to what is mentioned in the standard 2800 does not include sufficient accuracy to design structures in all buildings in this city and the multizonation method can be used to determine the spectrum for different parts of the city. Considering that the models’ fragility curves have been obtained in the previous section, the final results can be presented by finding the probability of collapse corresponding to the spectral acceleration obtained for each model in any part of Tehran. Table 12 shows the final analysis results of the probability of collapse of structures for the maximum ground movements in each site.

As mentioned before, the regulations set an acceptable limit for the probability of collapse in structures with residential use for maximum credible excitations which is 0.1. Therefore, according to the results obtained in Table 12, it can be said that some steel moment frames with residential use in some parts of Tehran have deviated from the above limitations and are therefore unsafe against collapse.

4. Conclusions

This study aims to investigate the probability of intermediate and unique steel moment structures designed according to the national building regulations of Iran under the effect of a maximum earthquake. The most important results of this research can be mentioned as follows:

(1) The results of the nonlinear static analysis show that the force required to deliver the intermediate moment frame has always been less than the unique moment frames; this can be due to the stricter rules of unique moment frames in Article 10 of the National Building Code of Iran.

(2) Since all the models under study have a periodicity period of more than 0.42 seconds (the intersection point of the 2475-year standard design spectrum of 2800 standard and the uniform hazard spectrum of Iran Hazard Analysis Project for Site No. 2 (building particular spectrum)), the collapse probabilities calculated by the spectral acceleration of the MCE spectrum (the return period of 2475 years) and even the DBE (475-year return period) of the 2800 standard are higher than the probability

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**Table 11: Spectral acceleration.**

| Structural specifications | Spectral acceleration of each site |
|---------------------------|-----------------------------------|
| Frame | Time period | SA(S2) | SA(1) | SA(2) | SA(3) | SA(4) | SA(5) | SA(6) | SA(7) |
| IMF5 | 0.88 | 0.72 | 0.58 | 0.66 | 0.89 | 0.77 | 0.66 | 0.61 | 0.66 |
| IMF10 | 1.66 | 0.38 | 0.31 | 0.35 | 0.47 | 0.4 | 0.35 | 0.32 | 0.35 |
| SMF5 | 0.84 | 0.75 | 0.61 | 0.69 | 0.93 | 0.8 | 0.69 | 0.63 | 0.69 |
| SMF10 | 1.58 | 0.4 | 0.32 | 0.37 | 0.49 | 0.42 | 0.37 | 0.34 | 0.37 |

**Table 12: Collapse probability of each site.**

| Frame | Collapse probability |
|-------|----------------------|
| IMF5  | P(S2) | P(1) | P(2) | P(3) | P(4) | P(5) | P(6) | P(7) |
| IMF10 | 0.092 | 0.04 | 0.06 | 0.17 | 0.1 | 0.06 | 0.05 | 0.09 |
| SMF5  | 0.04 | 0.01 | 0.01 | 0.02 | 0.01 | 0.01 | 0.01 | 0.01 |
| SMF10 | 0.04 | 0.02 | 0.03 | 0.09 | 0.06 | 0.03 | 0.02 | 0.05 |

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Figure 14: Location of surveyed sites on the seismic hazard zone of Tehran and its suburbs.
calculated with the uniform hazard spectrum of the project.

3. The results showed that the superiority of different structural systems in terms of safety against collapse compared to each other at all heights is not the same. In short-rise or high-rise buildings, whether for optimal design or safe design, it is different. For example, in this study, for 5-story frames, the particular moment frame reports a lower probability of collapse. It is, therefore, safer in this respect. However, unique or intermediate moment frames in a 10-story elevation system report the same probability of collapse.

4. Since the uniform hazard spectrum required for probabilistic analysis was collected from a site in Tehran and with the data provided by the Tehran Hazard Analysis Project, seven other sites were collected from this city with different seismicity and to draw their hazard spectrum. The need case data were used from the Iranian hazard analysis database, which showed that Tehran could not have a hazard spectrum to examine all structures. The multizonation method can be used to determine the spectrum for different parts of the city. Also, the possibility of the collapse of all structural models in 7 other sites was investigated. The results showed that, for some 5-story steel structures under study in sites of Tehran, the probability of collapse would be beyond acceptable, and also they are unsafe. That is an example of a violation of the methods mentioned in some regulations, such as version 4 of the 2800 Iranian standard.

This study aims to review and compare the new evaluation methods of structural design with the old methods proposed in the codes and provide examples of violations in the design of structures in terms of the regulations at the national or international level that they cannot prove their safety using new methods. Nevertheless, this is still a significant challenge because many types of structures can be studied. Future research developments can also study further uncertainties to achieve the more accurate size of related parameters, assess vulnerability and resilience in different structural systems, and compare it with acceptable and reasonable values against different earthquakes. On the other hand, obtaining zones for important cities where, due to the soil and seismicity of the region, there is a greater possibility of damage to buildings with different uses can bring valuable results.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The author declares no conflicts of interest.

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